DESIGN OF
OPEN CHANNEL TRANSITIONS

A Dissertation
submitted in partial fulfilment of
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of
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in
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CERTIFICATE

Certified that the dissertation entitled DESIGN OF OPEN CHANNEL TRANSITIONS, which is being submitted by Mr.A.Aini in partial fulfilment of the requirements for the award of degree of Master of Engineering in Water Resources Development of the University of Roorkee, is a record of the candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for award of any other degree or diploma.

Certified further that he has worked for a period exceeding six months from the Ist Oct. 1975 for preparation of this dissertation.

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LIST OF SYMBOLS

Symbol	Description	Dimension
1	2	3
A	Cross-sectional area	L^2
A_o	Cross-sectional area at a specific point	L^2
a	half width of the flume	L
a_y	Lateral acceleration	LT^{-2}
B	Bottom width of the channel	L
B_a	Average width of the channel	L
B_c	Width of the channel	L
B_f	Width of the flume	L
B_s	Width of the channel at water surface	L
B_x	Width of the channel at distance x	L
b	Half width of the channel	L
C	Constant	-
C_1, C_2, C_3, C_4	Coefficient	-
C_L	Coefficient of head loss $= H_L / hv_2$	-
Cumec	Cubic metre per second	$L^3 T^{-1}$
d	Depth of the flow	L
d_m	Mean depth	L
E	Specific energy head of flow at any point	L
E_o	Specific energy head of flow at position L_a in approach channel	L
E_s	Specific energy head assumed as constant	L
e	Energy correction factor	-

Contd.

1	2	3
F	Froude number	–
G	Constant = $\frac{F^{2/3}}{1 + \frac{1}{2} F^2}$	–
g	Acceleration due to gravity	LT^{-2}
H	Elevation of total energy line	L
H_1 & H_2	Energy head of upstream and downstream flow respectively	L
H_o	Elevation of total energy line above bed and energy of flow	L
H_L	Head loss	L
ΔH	Energy loss in the jump	L
h	Height of lining	L
h_f	Friction head	L
h_v	Veloeith head	L
Δh_v	Change in velocity head	L
h_{LS}	Head loss upto a particular section in transition	L
K	Constant defined by $\frac{gk}{c}$	–
k	Constant	–
L	Length of transition	L
L_a	Distance upstream to a point of measurement	L
M	Constant defined by $\frac{C}{kn}$	–
m	Momentum correction factor	–
N	Constant of integration	–
n	Constant and integral or frictional	–

Contd.

1	2	3
p	Pressure	$ML^{-1}T^{-2}$
Q	Discharge	$L^3 T^{-1}$
q	Discharge intensity or discharge per unit width	L^3T^{-1}/L
R	Hydraulic mean depth	L
R_o	Radius of curve for the recommended inlet	L
R_e	Reynold number of flow	-
r	Expansion ratio	-
S_o	Channel floor slope	-
S_f	Friction slope	-
T	Tangent distance for recommended inlet	L
T.E.L.	Total energy line	L
u	Coordinate of velocity in the downstream direction	LT^{-1}
V	Velocity of flow	LT^{-1}
$\bar{V}$	Dimensionless velocity of flow	-
V_m	Mean velocity of flow	LT^{-1}
V_n & V_t	Normal and tangential velocity of flow	LT^{-1}
V_c	Velocity of canal	LT^{-1}
V_f	Velocity of flume	CT^{-1}
v	Coordinate of velocity in lateral direction	LT^{-1}
W	Top width	L
WS	Water surface	-
Δ WS	Change in water surface	L
X	Axial distance from throat of flume or end of approach channel	L

Contd.

1	2	3
X_1	Half length of transition	L
y_1	Half of the drop in water surface	L
z	Elevation	L
z_b	Elevation of bed	L
Δz_b	Change in floor elevation	L
α & β	The coefficients for non-uniform velocity and pressure distribution	
α_1 & α_2	Constants	-
α opt.	Optimum rate of divergence for highest recovery head	-
β_1 & β_2	Mach angle formed by inclination of first/last disturbance line to the streamline	L^0
γ	Unit weight	$ML^{-2}T^{-2}$
θ	Angle of deflection of the wall	L^0
θ_1	Constant of integration	-
ϕ	Divergence angle	-
η	Efficiency of transition	-

S Y N O P S I S

Transitions are important features of open channels. Their design and functioning determine the stability of channels to a great extent. Taking this fact into consideration, an attempt has been made in this dissertation to critically and systematically review the available literature on basic theory of flow through channel transitions, their functions and procedures of design based on analytical approach as well as model tests.

The design procedures of transitions with subcritical flows have been elaborately described. The design of high velocity(supercritical) transition has assumed importance recently and has also been discussed. Critical comments, whereever possible, have been made on past and present practices. Some typical examples have also been mentioned.

Recommendations have been made regarding design of inlet and expansions transitions, energy losses, and their lengths based on economics and efficiency.

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CHAPTER-1

INTRODUCTION

1.1. GENERAL:

Nearly all hydraulic structures involve cross-sectional changes in artificial water courses whereby the uniformity of flow is disturbed and hence suitable transitions become necessary in streams and rivers as well, either manmade or natural, transitions may greatly influence the depth and velocity of flow.

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.

In open channels, there are several situations when the normal section of flow is constricted or flumed for reasons of economy. In bridges, aquaducts, syphons, superpassages, falls, head regulators and in many similar hydraulic structures the original cross-section of flow is reduced so as to economise the construction costs. Fluming of channels offers an expedient device for discharge measurement such as in Venturi Flumes.

Under normal design and installation conditions, practically all canal and flumes require some type of transition structure to and from the usual waterways.

In an open channel fluming can be attained in two ways:-

(a) By reducing the width of the channel without varying the depth. The discharge per unit width is consequently increased.

(b) By reducing the depth of the channel with or without varying the width. General usage would class such a work as a weir, but when the width is also reduced, it falls under category(a) and is known as a flume.

1.2. HISTORICAL BACKGROUND:

The antiquity of the conception of fluming may not be accurately found out from history but the usage of flumed works seems to have been let down to us from sufficiently early ages. In connection with constructional engineering, whenever it was found so necessary, the eary designers provided adequate means for effective fluming. The Roman Aqueducts and the ancient flumed works in France, Spain, North Africa and Mexico still have ample evidence in support of this statement. The constructional details and the designs of these structures conclusively prove that the basic theory of fluming was possibly not known in those days but their designers were acquainted with the application and effectiveness of the method of fluming.

Since the historic ages ' Fluming ' had been subjected to an uninterrupted train of experimentation. During this period flumes and flumed works appeared in various stages of development but it was not until the first decade of the present century that the standard flume of today took its shape. Alongside with this constructional progress of flumes, our knowledge about the underlying theory of fluming considerably advanced. The theoretical knowledge which is now available may not be complete as yet but fluming as a problem of Hydraulics, now stands a better chance for a more comprehensive study.

The studies were made in the past by various authors on the problem of sub-critical flow in both contraction and expansion transitions. Most of them were pertaining to closed conduits and relatively less work was done in open channel transitions.

Gibson[1] (1912) tried to find out the minimum head loss for his conical diffuser by measuring the values of K in the head loss formula

$$H_L = \frac{K\,(V_1 - V_2)^2}{2g}$$

The minimum loss was found to occur at a splay of 1 in 16.

Hinds[2] (1928) presented a summary of rules which were established for the design of transition structures by the United State Bureau of Reclamation (U.S.B.R.). The method of design for warped type transition is based on several assumptions, most important one of them being regarding the length of transition which is fixed by an angle of $12\frac{1}{2}^{\circ}$ between the axis of channel and the line joining the points where the water surface meets the side wall at entry and exit of transition.

The Central Board of Irrigation and Power, India[3] (C.B.I.P), (1934) made some analytical studies on open-channel expansions with a view to arriving at an optimum rate of divergence, The final expression for the divergence angle ϕ , was found to be

$$\operatorname{Tan}\ \phi = \frac{C\,(R/B_c)^{n} + 0.018\,(R/B_c)^{-1}}{R_e^{1.5} \times \frac{e}{1-e\,F} + m\left(2 - \frac{r}{r-1} + eF\right)}$$

Where B_c is the width of the channel, e the energy-correction factor, r the expansion ratio, and m the momentum correction factor.

All the parameters are for downstream channel; where the flow is least stable. The model studies become essential to determine the more significant parameters which influence the transition's performance predominantly.

It also outlines the simplified procedure for transition's design, based on specific energy principles. But in all the designs, the illustrated lengths of transition have been arbitrarily fixed by splays varying between 1 in 2 to 1 in 3.

The works done by A.C. Mitra [4] (1940), The Poondi Research Station, Madras[6] (India) in 1949, Chaturvedi[10], (1963), were all directed towards finding an efficient shape of expansion (the length being assumed arbitrarily).

Kalinskie [5] (1946), Chaturvedi [9] (1963) and some others studied the flow characteristics in an expansion.

Ippen [7] (1950) adopted similar procedure as the C.B.I.P. and made some interesting study of specific energy curves.

Tults [8] (1956) conducted a series of experiments in rectangular closed conduit with straight linear boundary. From the result of his experiments he derived the expression :-

$$\alpha_{opt.} = \alpha_1 + \alpha_2 \eta \left(1 - \frac{1}{r^2}\right)$$

Where $\alpha_{opt.}$ is the optimum rate of divergence for highest recovery of head, η the maximum pressure efficiency, r the expansion ratio, and α_1 and α_2 are the constants. The optimum angle for infinite expansion was found as 8^0 to the axis.

Tult did not consider non-uniform velocity distribution and ignored the downstream kinetic energy in determining the value of efficiency. Moreover, the results of his experiments conducted in a semi-model closed conduit may not tally with those of a double sided open-channel expansion, where resistance varies from point to point in cross-section.

1.3. PURPOSE AND DESCRIPTION :

Large numbers of transition structures are needed in irrigation distribution systems, drains and other water conveyance to direct canal flows into pipelines, syphon barrel, and aqueduct, and back into canal section again. Transitions are generally used both at the inlet and outlet of structures. An accelerating water velocity usually occurs at the inlet of a structure and a decelerating velocity of the outlet.

Transitions usually produce gradual changes in water prism cross-sections and are used at structure inlet and outlets and at changes in canal sections to :

(1) Provide smooth water flow,

(2) avoid excessive energy losses,

(3) minimize canal erosion,

(4) reduce ponded water surface elevation at cross drainage structures,

(5) provide aditional stability to adjacent structures because of the added resistance to percolation,

(6) retain earthfill at the ends of structures,

(7) provide safety for the structure and the channel on the downstream, and

(8) to eliminate cross wave and other turbulence.

The form of such transitioning may range from straight vertical headwalls normal to the water flow to very elaborately designed streamlined warped transition.

1.4. GENERAL REQUIREMENTS:

Where head is not valuable, or for small structures, the straight head wall type may be satisfactory, but for the usual structure of any size, a somewhat more elaborate transition is desirable. In those cases where head is extremely limited or valuable, the transitions should be very carefully designed for streamline proportioning. Transitions may also be required between adjacent earth and lined canal sections. In general, acceleration of velocity occurs in inlet transitions, and deceleration in outlet transitions. They should never be designed so as to constitute undesirable " choke " sections, and transition floors should normally not be set on steep slope that would destroy the effectiveness of the inlet cut-off in performing its hydraulic functions. Properly designed transitions may provide some control of flow and eliminate much of the danger from percolation, erosion, scour, particularly at the outlet.

1.5. TYPES :

Transitions are either open (no top) or closed. Open transitions may be either of concrete or earth. Earth transitions are used to transition base width, invert elevation, and side slopes from a canal structure or concrete transition to that of the waterway section. The water surface is opened to atmosphere, and the pressure on the surface is then constant (atmospheric).

The most common concrete transitions for inline canal,structures are:

(1) streamlined warp, (2) straight warp, and (3) broken-back.

Closed transitions (sealed) are used to further reduce energy losses for pipe structures by providing an additional gradual change of water prism cross-section from rectangular to round. The pressure on the top surface may thus vary from zero to any practicable value.

Transitions of class I (open) may be further subdivided as follows:-

Sub-Class I (i):

In which the velocity never rises above the critical. Practical examples of this sub-class are (a) flumed bridges in which the fluming ratio depends largely on the nature and cost of the overwork, (b) venturi flumes in which the depression of the surface level is used to measure the discharge passing; (c) aqueducts and superpassages in which also, fluming is resorted to in order to reduce the capital cost of the work.

Sub-Class I (ii):

In which the velocity rises above the critical but the design in such that the stream returns to sub-critical flow without interposition of a standing wave, This type is rather rare although a few has been built of which the dimensions constrain a hyper-critical velocity but for which no provision has been made to reduce the velocity to sub-critical without the interposition of a standing wave. This sub-class postulates a loss of head much higher than that in sub-class I (i) but it is, in general economical owing to the decreased waterways required.

Sub-Class I(iii):

In which the velocity rises above the critical and in which provision is made for the generation of a standing wave at the point where the stream resumes subcritical velocity.

Transition of Class II (sealed) can also be subdivided on similar lines:

Sub-Class II (i):

In which the velocity never rises above the critical. This type is seldom met with in practice, but easy to design and the loss of head is relatively small owing to the low velocity.

Sub-Class II (ii):

In which the velocity is above the critical. This is the type ordinarily met with in practice. The barrel is designed to be of as small cross-sectional area as is compatible with a reasonable velocity through it. The possibility of varying the pressure energy renders it possible to design the transition from hyper-critical velocity to sub-critical with the greatest simplicity. The loss of head in this type is, however, higher than in the former due to higher velocity.

Sub-class II (iii):

In which the velocity rises above the critical and in which a standing wave forms at the point of return from hyper-critical velocity to sub-critical. It is seldom that this type is designed deliberately. Usually it is found that a syphon of the sub-class II(i) turns out to be of this class due to faulty designing. As a matter of fact, this type is difficult to design and in practice the loss of head is usually transferred to a fall of appropriate type.

1.6. PURPOSE AND SCOPE OF DISSERTATION:

Since the transition is desirable to be used at many places along the canal alignment, a suitable design to render it economical and the energy loss minimum is a necessity.

The design of transition is an important problem in hydraulics and many investigations and model tests have been done by the institutions in different countries for finding the nature of the flow in the transition portion of the canal, and also the most efficient hydraulic performance of the transitions. Still many of the problems in this subject remain unsolved.

In order to develop and find a definite method of design based upon available data, a review of all investigations for the design procedure of inlet and outlet transitions becomes necessary.

Taking materials from different books, journals, technical memoranda, and other publications the various methods of design have been critically discussed in this dissertation. Also optimum length of the transition that is the most important in taking care of the constructional cost is recommended here. Different controlling devices meant to prevent the flow separation in the downstream transition, which produces disturbance in the flow and causes erosion of the channel have been mentioned.

The design of high velocity transition which has been developed recently, is also discussed here, with illustrated example for design of inlet and outlet transitions.

CHAPTER-II

DESIGN PROCEDURE FOR SUB-CRITICAL FLOW

2.1. BASIC THEORY:

The equation of energy for irrotational fluid flow, commonly known as the Bernoulli's theorem, was first established by Daniel Bernoulli in 1738, nearly two decades before Leonhard Euler founded the science of hydromechanics.

Total energy head can be expressed in terms of mean velocity V_m as:

$$H = \frac{p}{\gamma} + Z + C \frac{V_m^2}{2g} \qquad (2.1)$$

which is the general practical equation for streamline flow.

In open channel, the surface level 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 + Zb, d being the depth above bed level and Zb, the elevation of the bed, Fig.2.1

In this case equation (2.1) can be written as :

$$H = d + Z_b + C \frac{V^2}{2g} \qquad (2.2)$$

where C is a constant.

The elevation of the total energy line above the bed level as the datum line is

$$H_o = d + C \frac{V^2}{2g} \qquad (2.3)$$

When C =1, $H_o = d + \frac{V^2}{2g}$ (2.4)

H_o is known as the energy of flow, E or specific energy.

In a rectangular channel with level bed, $V = q/d$ where q is the discharge per unit width. Equation (2.3) becomes

$$H_o = d + C \frac{q^2}{2gd^2} \qquad (2.5)$$

which when C = 1, reduce to

$$H_o = d + \frac{q^2}{2gd^2} \qquad (2.6)$$

which is illustrated in the adjoining Fig.2.2

These curve for various discharges are useful in the design of flumed structures. It will be seen that for each value of H_o a given discharge q per unit width can have two alternative depths of flow and that a minimum H_o exists for each value of q.

The depth and velocity at which the minimum value of depth occurs are known as ' critical depth ' and the ' critical velocity '. They are given by

$$C \frac{V^2}{gd} = 1 \quad \text{or} \quad d = \frac{CV^2}{g} \quad \text{or} \quad \frac{d}{2} = \frac{CV^2}{2g} \qquad (2.7)$$

substituting this value in eq.2.3 and putting ~~value~~ the value of H_o from equation 2.5 we will get :

$$d = 2/3\, H_o = \left(\frac{C\,q^2}{g}\right)^{1/3} = \frac{C\,V^2}{g} \qquad (2.8)$$

$$\text{or} \qquad V = \sqrt{\frac{2gH_o}{3\,C}} = \left(\frac{gq}{C}\right)^{1/3} = \sqrt{\frac{gd}{C}} \qquad (2.9)$$

In non-rectangular uniform channels the corresponding equations are :

$$H_o = d + C \frac{Q^2}{2gA^2} \qquad (2.10)$$

and $$H_o = d + \frac{Q^2}{2gA^2} \qquad (2.11)$$

where datum for H_o and d is the lowest point of the bed. Since $dA = B_s \times dd$ where B_s is the surface width, the critical velocity and the critical depth are given by

$$C \frac{V^2}{g\ dm} = 1 \qquad (2.12)$$

and $$V = \sqrt{\frac{g\ A}{C \times B_s}} = \sqrt{\frac{g\ dm}{C}} \qquad (2.13)$$

where dm is the mean depth $\frac{A}{B_s}$

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

Referring back to equation (2,7), we have for uniform flow (C = 1), the critical depth and velocity are given by

$$\frac{V^2}{gd} = 1$$

where $\frac{V}{\sqrt{gd}}$ is the Froude number, F of the flow. Thus F serves as an useful indication of the flow characteristic. When $F > 1$, the flow is supercritical and for $F < 1$, it is sub-critical.

A difference between sub-critical and supercritical flow is that in the former a disturbance or afflux affects the upstream conditions whereas in the latter no such effect is produced unless the disturbance is sufficient to form a standing wave which advancing upstream changes the flow from supercritical to sub-critical. 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 sub-critical and the super-critical flow is that in the former a contraction producing a converging flow where on the sides or bed or both of the channel, results in reduced depth of flow provided, the depth is comparatively small; whereas in supercritical flow it results in an increased depth. The effect of convergence due to a rising bed is shown in Fig.2,3, width

A contraction in width increases the discharge per unit width q while the total energy head H_0 remains unchanged. If any horizontal line is followed in Fig.2.3 it will be seen that with increasing q, d decreases in sub-critical region and increases in the supercritical. On the other hand, arise in bed decreases the total head H_0 and q remains unchanged. If any line of constant q is followed, it will be seen that with decreasing H_0, d decreases in the sub-critical region and increases in the super-critical one. Converse is the case for expansion.

2.2.1. ANALYTICAL CONCEPTS FOR DESIGN OF TRANSITIONS:

The flow through a transition is non-uniform: Depending on the degree of constriction and rate of flaring, the flow may either be rapidly, varied or gradually varied in nature. Due to curved surface profile in a transition the pressure distribution is not hydrostatic. In the contracting zone the pressure gradient is negative and in expanding zone the pressure gradient is positive, as the flow is sub-critical. These pressure gradients affect particularly the layers of liquid in the immediate proximity of the boundary which are deficient in momentum because of boundary resistance. For negative pressure gradients (i.e. in zones of acceleration) the momentum of these deficient layers will tend to be augmented because the corresponding accelerative forces are generally larger than those of boundary shear. For positive pressure gradients (i.e. zones of deceleration) on the contrary, the pressure forces join with boundary shear in tending to reduce even further the momentum of boundary layer. Thus, where as accelerative boundary flow is inherently stable, decelerative boundary flow almost invariably unstable and results in flow separation from the boundary leading to turbulence and eddies(11).

Although surface of separation are sometimes considered to enclose region of dead water, the high intensity of shear along separation surface actually produces appreciable circulation in the eddy or roller thus formed. These rollers are the source of much of the head loss associated with such flow, since the main flow must infuse the energy to sustain them. Substantial portion of kinetic energy, which could have been transformed to pressure head, gets eaten away through production of turbulence which is converted, diffused and dissipated by the main stream subsequently.

The theoretical treatment of transition flow is restricted severely by basic assumptions and boundary conditions which can be handled analytically. In most cases, the stream behaviour must be anticipated from basic experimental information or from factors not included in the analytical treatment or not considered in adequate form. When both analysis and sound extrapolation of experimental evidence prove inadequate, recourse must be had to specific model studies.

The exact laws governing the complex expansive flow not being understood so far, simplifications have necessarily been assumed to arrive at approximate workable solutions.

The general trend is towards the one-dimensional approach which is described as below:

2.2.2. ONE-DIMENSIONAL APPROACH:

The flow is actually three dimensional and flow parameters change from section to section; with the number of variables involved it is very difficult to analyse them theoretically. For simplifying analysis one dimensional characteristics of flow with a free surface are considered. This is accomplished by presuming that the zone of flow under consideration consists of single stream tube characterized at each section by a mean velocity of flow, a mean pressure and mean elevation. Variations of flow characteristics across any section are thus ignored and only the changes of mean values in the direction of flow are taken into consideration.

To explain it briefly, taking the x-direction along the axis of the transition, and d as the depth of flow on a horizontal bed, the fundamental equation of energy for a channel with vertical sides, ignoring friction and other losses is given by

$$E_s = \frac{\alpha Q^2}{2g A^2} + \beta d \qquad (2.14)$$

where E_s is the specific energy assumed as constant,

Q is the constant discharge,

A is the variable area at any point in the transition, and α & β are the coefficients for non-uniform velocity and pressure distribution.

Differentiating and assuming α & β equal to 1,

$$-\frac{dd}{dx} = -\frac{Q^2}{g} \cdot \frac{1}{A^3} \quad \frac{dA}{dx} \qquad (2.15)$$

Evidently equation 2.15 can be solved either by (i) assuming the water surface profile dd/dx, or (ii) the variation of the area dA/dx. By the way, simple illustrations are given below :-

(i) Let dd/dx = C, i.e. the water surface is assumed, to be a straight line. Then from equation 2.15, $dA/A^3 = - Cg/Q^2\ dx = C\ 1\ dx$ and $dA/dx = C\ 1A^3$ (2.16)

(ii) Similarly, if the cross-sectional area is assumed as a linear function of X, i.e. $A = A_0 + a_1\ x$

$$\frac{dA}{dx} = a_1$$

Hence, from equation (2.15)

$$\frac{dd}{dx} = \frac{a_1\ Q^2}{g\ A^3} \qquad (2.17)$$

The solutions based on either of the assumptions (i) and (ii) have the likely disadvantage of abrupt discontinuities in the values of dv/dx and dd/dx, more specially at each end of the transition reflected as kinks in the water surface or boundary line. Several trials may be necessary till the final step by step computation eliminates all discontinuities as it may sometimes require a different assumption of water surface slope or area variation.

Correction is finally made in the water surface profile by allowing for friction losses from Chezy or Manning's formula which is another arbitrary approach to adjust the overall losses in the proposed design.

In varied flow the friction losses vary from point to point in proportion to V^2 which itself is variable, while the losses due to form effect are indeterminate. The one dimensional approach, thus, gives fairly satisfactory results when the constriction is very small[12].

For rapid transitions, it becomes very difficult to define accurately the magnitude of total head even when the depth is known. This difficulty is due to variation in the velocity distribution as a result of boundary resistance, local value of $V^2/2g$ within stream differing considerably from both $V^2/2g$ and the true average velocity head. The variation is reflected in the total head so that the one dimensional treatment in terms of mean velocity can be continued only if correction factors α & β for the increased kinetic energy and momentum of stream are introduced. The value of α and β change at successive sections of a transition in a fashion not readily predictable which makes it difficult to work out these values. In rapidly accelerated motion, the values of α and β decrease towards unity, whereas for decelerated motion in expanding streams their values may greatly increase at sections adjacent to transition structure. In evaluating experimental measurements, therefore, it is seen that careful determination of the velocity distribution is important in assigning the proper rate of head loss to a transition.

2.2.3. TWO-DIMENSIONAL APPROACH[12]

This approach attempts at evolving a shape of transition which is capable of imparting positive lateral acceleration to eliminate separation. According to Bakhmeteff[37]. "In case of a divergent flow, when streamlines are subtantially inclined towards the cross-sectional area, the acceleration Oa may have a noticeable component Oa' in the cross-sectional plane (see Fig.2.6), the effect of which will be to modify the distribution of pressure as caused by gravity alone. These local stresses due to pressure variation and lateral accelerative forces would necessarily dominate the flow to the extent the curvature is impressed and constriction imposed."

R.S. Chaturvedi(12) analysed the flow in subcritical expansions with dimensional approach. Dominating forces coming into play in case of expansive transitions are, (i) lateral influx of momentum from main stream, (ii) the adverse pressure gradient acting upstream, (iii) boundary drag opposing the convective transverse mixing, and (iv) the frictional resistance of side walls and bed acting against the flow whether axial or lateral. It has not been possible so far, to derive an expression for the lateral transfer of momentum. When strong enough, it can successfully outbalance the tendency towards separation and eddy formation at the sides.

We may better assume lateral velocity dy/dt to be any function of y, obtain the curves of the transitions more generally. However, the most appropriate form of f (y) has to be guessed and should be sufficiently general such as :

$$f(y) = \frac{dy}{dt} = ky^n \qquad (2.18)$$

where n can be integral or fractional, positive or negative and k be any constant. Similar other curves can now be evolved from equation 2.18 whose hydraulic performance can be analysed, compared and examined on a model.

In fact, equation(2.18) is quite general and covers even the earlier of equation(2.17) for which the approach was quite different

(Julian Hinds approach). To illustrate the same, assuming frictionless flow, we have from equation (2.15)

$$dd/dx = Q^2/g \quad 1/A^3 \quad dA/dx = K \quad (\text{ say })$$

(For expansion flow dd/dx increasing and is positive).

Then, $dA/dx = g\,K/Q^2 \, . \, A^3$

or $$\frac{d}{dx} \cdot \frac{Q}{V_x} = \frac{gK}{Q^2} \cdot \frac{Q^3}{V_x^3}$$

Since $V_x y$ = constant, (continuity equation) or $V_x = C/y$, we have as before

$$Q \cdot \frac{d}{dx} \cdot \frac{y}{C} = Qgk\,(y/c)^3$$

and $$\frac{dy}{dx} = \frac{gk}{C^2} \quad y^3$$

So that,

$$\frac{dy/dt}{dx/dt} = \frac{gk}{C^2}\, y^3$$

and $$\frac{dy}{dt} = \frac{gk}{C^2} \quad y^3 \quad \frac{C}{y} = \frac{gk}{C} \quad y^2$$

which reduces to the general form of equation (2.18)

i.e. $$\frac{dy}{dt} = K\,y^n$$

Where $K = \dfrac{gK}{C}$

2.2.4. <u>DERIVATION AND ANALYSIS OF SOME OTHER FORMS OF TRANSITION CURVES:</u>

Giving different values to n in the general equation, i.e. equation (2.18), -1, -1 1/2, 0, + 1/2, 1, 3/2, 2 etc., other forms of transitions are derived as below:

Since $dx/dt = C/y$ (continuity equation) and $dy/dt = K\, y^n$

Therefore, $dx/dy = \frac{dx/dt}{dy/dt} = \frac{C/y}{K\, y^n} = \frac{C}{K\, y^{(n+1)}}$

and $dx = C/K \cdot dy/y^{(n+1)}$ (2.19)

equation (2.19) is solved (i) when $n \neq 0$ as

$$X = \frac{C}{k} \cdot \frac{y^{-n}}{n} + N$$

or $X = M\, y^{-n} + N$ (2.19a)

Also (ii) when $n = 0$

$$X = C/K \log y + N$$

or $X = M \log y + N$ (2.19b)

By giving different values to n in equation (2.19 a) we can obtain a family of curves. The values of the constants can thus be evaluated with the help of the following boundary conditions :

$y = a$, when $x = 0$

$y = b$, when $x = L$

The curves thus obtained by taking $n = -1, -1/2, 0, 1/2, 1, 3/4$ and 2 are given in Table 2.1. Curve 3 is obtained for equation (2.19 b) and is a logarithmic curve. For n=1, the hyperbolic transitions evolved by Mitra are obtained.

On the basis of experiments conducted by Chaturvedi, he found that the best geometrical shape (Table 2.1 Curve-6) for subcritical expansion transitions is given by

$X = M/\, y^{3/2} + N$ (2.20)

If 2a is the width of flume channels 2 b width of channel downstream and L is the length of transition as shown in Fig.2.17. On elimination of the constants equation (2.20) reduces to,

$$X = \frac{L\, b^{3/2}}{b^{3/2} - a^{3/2}} \left[1 - (a/y)^{3/2} \right] \qquad (2.21)$$

The lateral accelerations for the different transition shapes have also been worked out. $a_y = d^2y/dt^2 = dV_y/dt = V_y\, dV_y/dy$ or, $a_y = n k y^{(n-1)} V_y$.

TABLE 2.1

DERIVED CURVES, THEIR EQUATIONS AND CHARACTERISTICS OF FLOW

Sl.No.	Equation of derived curve	Equation of transition for throat=2a, canal width =2 b and Length = L	Value of n in $dy/dt = Ky^n$	$V_y = dy/dt$	$a_y = d^2y/dt^2$
1	$X = My + N$	$X = \frac{L}{b-a} y - \frac{aL}{b-a}$	-1	K/y	$-K^3/y^3$
2	$X = My^{1/2} + N$	$X = \frac{L}{\sqrt{b}-\sqrt{a}} (\sqrt{y} - \sqrt{a})$	$-\frac{1}{2}$	$K/y^{1/2}$	$-\frac{1}{2} K/y^2$
3	$X = M \log y + N$	$X = \frac{L}{\log b - \log a} (\log y - \log a)$	0	K	0
4	$X = M/\sqrt{y} + N$	$X = \frac{L\sqrt{b}}{\sqrt{b}-\sqrt{a}} \left(1 - \sqrt{\frac{a}{y}}\right)$	1/2	$Ky^{1/2}$	$1/2\ K^2$
5	$X = M/y + N$	$X = \frac{L\,b}{(b-a)} - \frac{L\,ab}{(b-a)y}$	1	Ky	$K^2 y$
6	$X = M/y^{3/2} + N$	$X = \frac{L\,b^{3/2}}{b^{3/2} - a^{3/2}} \left[1 - (a/y)^{3/2}\right]$	3/2	$Ky^{3/2}$	$3/2\ K^2y^2$
7	$X = M/y^2 + N$	$X = \frac{Lb^2}{b^2-a^2} (1 - a^2/y^2)$	2	Ky^2	$2\ K^2y^3$

It can be seen from the table 2.1 that n = -1 gives a straight line transition with negative acceleration which may indicate a possibility of central jetting action or even one-side flow. Also positive acceleration generally increases as the value on n increases.

Negative acceleration persists in the second derived curve and is zero for the logarithmic curve 3. After this, the acceleration is positive for curves, 4,5,6 and 7. Curve 5 is Mitra's hyperbolic expansion, which will be discussed later. The comparative geometrical shape of the different curves can be seen in Figs.2.7,2.7(a) and 2.7(b).

In the above derivations the depth of flow has been assumed to be constant and effect of adverse pressure gradient has been ignored. These assumptions are of less serious nature than the assumption of ignoring deviation of hydrostatic distribution caused by the curvilinear flow along the expansion in the one dimensional approach.

2.3. VARIOUS METHODS OF DESIGN

2.3.1. HINDS METHOD:

The first major investigation for design of transitions was undertaken by Hinds[2]. From analysis of field data Hinds observed that these transitions which are gradual and smooth gave better performance in comparison to those having abrupt discontinuity in boundary and less gradual. He presented a number of empirical rules governing the design of transition based on the following assumptions:-

(i) Water surface profile is a compound curve made up of two reverse parabolas with the inflexion point at the middle and merging into the upstream and downstream water surface at either end of the transition tangentially.

(ii) Head loss coefficients C_1 and C_2 for inlet and outlet transitions respectively, remain constant throughout the length of transition.

(iii) Length of transition is given by an angle of 12°-30' between the axis of the channel and the line joining the flow line at two ends of the transition.

Design of transition by Hinds method is discussed under four main headings, as follows :-

(1) Unimportant, or low-velocity structures, not involving the hydraulic jump or flow at the critical depth.

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

(3) Structures, involving the hydraulic jump or flow at the critical depth.

(4) Experimental data (described in para 2.7).

2.3.1.1. UNIMPORTANT TRANSITIONS:

Where velocities are low, may be designed arbitrarily, by adaptation from successful structures operating under similar conditions.

The U.S. Bureau of Reclamation has accumulated a large variety of detailed designs for transition structures of secondary importance. 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. A collection of possible types of simple transitions, sketched from some of these designs, is shown on

Figs.2.8,2.9,2.10 and 2.11. These structures differ widely in degree of perfection, and in the losses which they may be expected to produce.

The simplest forms of pipe inlets or outlets are illustrated by Types (1),(2) and (3), Fig.2.8. These types are used for culverts, and are satisfactory for small pipes. It is probably reasonable to assume entrance and outlet losses equal to 0.5 and 1.0 respectively, of the velocity head in the pipe for these structures. Types (4),(5) and (6), Fig.2.8, are somewhat more complicated structurally, and are perhaps slightly better hydraulically. Type (7), Fig.2.8, which is sometimes used for carrying small irrigation canals under roads and rail roads in cuts, is poor hydraulically, and is subject to stoppage by silt and weeds. Types (8) and (11), Fig.2.8, are slightly better, but more expensive to construct. Types (9) and (10), Fig.2.8 are fairly efficient, involving entrance and outlet losses, respectively, of perhaps 0.25 and 0.5 of the velocity head. Types (12) to (17), inclusive Fig.2.9, illustrate trash rack, weir and control gate arrangements for pipe inlets.

A type of structure suited to a given set of conditions may be selected from examples in Figs.2.8 to 2.11, the probable loss of head estimated by comparison with some structure for which the losses have been measured, and a reasonably good design prepared.

2.3.1.2. THE DESIGN OF IMPORTANT TRANSITION,NOT INVOLVING THE HYDRAULIC JUMP OR CRITICAL FLOW:

Experience indicates that the method of design outlined for simple structures is not adequate for important installations, especially where velocities are relatively high (excess of 1.8282 to 2.4384 metre per second). Under such conditions, the detailed dimensions and forms of the structures, throughout its entire length, become important. Careful detailed computations must be made. Extremely slender, and carefully constructed transition, if not proportioned in exact accordance with the hydraulic eequirements, may prove seriously defective. Sometimes the assumption is made that perfection can be approached by reducing the angle of divergence of the transition.

A study of many situation in U.S.A. showed the computed water-surface profile to be irrigular, or to contain at least one sharp angle, for all known faulty structures, except a few which are influenced by curvature in the channel. Accordingly, a new criterion for design was adopted namely, that the computed water-surface profile through the transition shall be a smooth, continuous curve, approximately tangent to the water surface curves in the channels above and below.

Undoubtedly, there is some particular form of surface curve best suited to any given set of conditions, but no data for the determination of the correct curve are available. Also; the length of the curve, or the slenderness of the structure, probably bears a definite relation to the efficiency of the transition, which relation is yet to be determined.

The application of the rule just given is illustrated by the two simple structures shown in Figs.2.12 and 2.13. Before computing the hydraulics, the inlet shown in Fig.2.12 might be considered good. However, if the hydraulics are computed at short intervals throughout the structure, the surface curve will be found to contain a sharp angle at the junction with the narrow channel, and marked disturbances in flow may be expected. By properly curving the walls of the inlet, as shown in Fig.2.13, this condition can be avoided, the rate of change in acceleration being changed in such a way that the water-surface profile becomes a smooth, continuous curve.

A plan based on the principles illustrated in Fig.2.13 is used by the U.S. Bureau of Reclamation for all important transitions. If desired, the dimension of the structure may be assumed and the surface curve computed from Bernoulli's theorem, the dimensions being subsequently changed and the curve recalculated until satisfactory results are secured. A more direct solution is obtained if the water-surface curve is first determined and the dimensions of the structure are computed to conform. The procedure recommended will be illustrated by example of actual designs.

2.3.1.3. THE INFLUENCE OF THE HYDRAULIC JUMP AND CRITICAL FLOW :

The preceding discussion is strictly applicable only where the hydraulic jump of flow at the critical depth are not involved. Where these factors are encountered additional precautions are required, although rules laid down for general proportions still hold. Numerous papers on the hydraulic jump have been published in the last few years. No important new data on this subject have been developed by the U.S. Bureau of Reclamation.

It may be well to emphasize again the fact that the unexpected and unnecessary introduction of critical flow conditions is a frequent source of trouble in transitions. This condition is illustrated in the outlet shown in Fig.2.14. Although the total length of this structure is 30.48 metre the velocity actually increases to a point near the lower end, where it suddenly " jumps " to approximately canal velocity. The normal depth in both the tunnel and the canal is above critical, and if proper attention had been given to the detailed dimensions, the jump would have been avoided and the efficiency of the outlet increased. In addition, heavy wave action in the canal would have been avoided. However, it is essential that any approach to the critical depth be carefully noted. Before attempting to plan an important transition the designer should be thoroughly familiar with the phenomenon of critical flow.

2.3.2. MITRA'S METHOD (13),(14)

Faced with the problem of large scale remoulding of irrigation works, A.C. Mitra, Chief Engineer, Irrigation Department, Uttar Pradesh(U.P.) India, initiated in 1940 a method of design known as hyperbolic transitions, for which a rational formula was also evolved by him. A large number of transitions were designed by the author and their performance was also studied under field conditions. They have been reported to be generally successful. The Froude number seldom exceeded 0.6.

Mitra's approach lies in determining a rational curve based on his formula assuming :-

(i) The rate of change of velocity along the length of transitions remains constant i.e. $dv/dx = \text{constant}$.

(ii) The depth of flow remains constant in the direction of flow. This assumption was sought to be achieved by gradually lowering the bed in the constriction transitions and raising it in the expansion.

If L is the length of transition (to be decided), B_c the width of the canal downstream (given), B_f the width of throat of the flume velocity throughout the length L. Checking the water surface profile, a satisfactory curve is obtained as compared in Hinds transition (Fig.2.4)

Theoretically, both the assumptions made by Mitra can be objected to. But, as mentioned earlier (Sec.2.2.2.), if dz/dx can be arbitrarily assumed to determine dA/dx, or vice versa, dv/dx = constant (as assumed by Mitra) is an equally good approach.

The second assumption of Mitra stipulates no recovery which, though theoretically incompatible, is again not so serious in practice. The constant specific energy and variable discharge diagram in Fig2.16 explains (approximately) the fundamental behaviour of the expanding transition as to how the combined transition loss and friction loss together with a rising bed would tend to keep the depth of flow nearly uniform. The figure is self-explanatory. There is necessarily some recovery howsoever small-more so in suitably designed transition and as such the no-recovery assumption for sub-critical flow is not fully justified.

Mitra's transition has been used quite frequently in U.P.(India) and elsewhere and its field performance has been very satisfactory. On attempting a rational solution for Mitra's equation with two dimensional approach (Table 2.1, curve 5), it was found that this curve was capable of importing a positive lateral acceleration to eliminate separation, which was not apparent otherwise from his original solution. However, the two basic assumptions i.e. constant depth & uniform rate of change of velocity limit the use of these transitions for constrictions easier than 50% (12).

(as proposed), B_x the width of flume to be determined at any point X (Fig.2.15), V_f and V_c are the mean velocities at B_f and B_c, i.e., within the throat and normal canal section, and X the axial distance from the throat where the velocity is V_x and width B_x, then

$$\frac{V_f - V_x}{X} = \frac{V_f - V_c}{L} \tag{2.22}$$

From the continuity equation and the first assumption,

$$B_f\ V_f = B_x\ V_x = B_c\ V_c = \frac{Q}{d} = K\ (\text{say}) \tag{2.23}$$

Therefore,

$$V_f = \frac{K}{B_f}\ ,\quad V_x = \frac{K}{B_x}\quad \text{and}\quad V_c = \frac{K}{B_c}$$

Substituting the above values in equation (2.22).

$$\frac{\frac{K}{B_f} - \frac{K}{B_x}}{X} = \frac{\frac{K}{B_f} - \frac{K}{B_c}}{L}$$

which reduces to

$$B_x = \frac{B_c \cdot B_f \cdot L}{L\ B_c - X\ (B_c - B_f)} \tag{2.24}$$

Mitra has justified his constant depth assumption by depressing the bed gradually towards the throat to the extent $\left(\frac{B_f^2}{2g} - \frac{B_c^2}{2g}\right)$, and then sloping it towards the exit. This is equivalent to designing for a closed conduit of constant depth, assuming a uniform variation of velocity throughout the length L. Checking the water surface profile, a satisfactory curve is obtained as compared in Hinds transition (Fig.2.4)

Theoretically, both the assumptions made by Mitra can be objected to. But, as mentioned earlier (Sec.2.2.2.), if dz/dx can be arbitrarily assumed to determine dA/dx, or vice versa, dv/dx = constant (as assumed by Mitra) is an equally good approach.

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2.3.3. CHATURVEDI'S METHOD:

The essentials of the method have been described in sections 2.2.3 and 2.2.4. For a value of n=1 the equation (5) Table -2.1 represents Mitra's hyperbolic transition. The plotting of different curves of Table 2.1 is shown in Fig.2.7. The experiments conducted by Chaturvedi[12] have shown that performance of the curve with $n = \frac{3}{2}$ is slightly better than Mitra's curve and was recommended. The transition curve suggested by Chaturvedi (Table-2.1, Curve-6) has been used at many places in India and their performance was found to be satisfactory.

2.4. WATER SURFACE:

2.4.1. WATER SURFACE PROFILE:-

The elevation of ends of the inlet transition may be determined from the drop in water surface. The length of the transition must be determined. Referring to Fig.2.23, between A & C the water surface theoretically may be made to follow any desired profile, within reasonable limits, the profile being controlled by the shape and size of the transition structure.

If the flow is to be smooth and the structure efficients, the theoretical water surface must be free from angles or sharp curves. The particular curve that fits the given conditions better than any other not being known, any smooth curve tangent to the normal water surface in the canal, at A, and in the flume, at C, may be used. It may be drawn arbitrarily or computed. In Fig.2.23, the water surface, neglecting friction, is taken as two equal parabolas, horizontal, respectively, at A & C & tangent at B.

A number of sections are next chosen across the transition at which to compute the elevations of the water surface and the structural dimensions. It is convenient to have these sections equally spaced. Let the length of contraction transition be 2 X_1 and the drop in water surface $2y_1$ as in (Fig.2.18). At the middle point of the transition (at B) the distance from section A will be X_1 and the drop in water surface y_1. Taking water surface at section A as the origin, the equation of the first parabola may be written as :-

$$y_1 = C\,X_1^2 \tag{2.25}$$

Substituting the known values of y_1 and X_1, the value of C can be obtained and hence the first parabolic curve plotted downstream and downwards from the origin at section A. The other half parabola is exactly similar, but for it the origin is on water surface at section C and the y values are to be measured upstream and upwards from the origin at the water surface at Section C.

In expansion transition the water surface is similarly determined. In this case the first half of the curve from flume throat will be concave upwards and the second half convex upwards.

For transitions which deliver water to or receive water from a pressure flow conduit or barrel, a single parabola is a more suitable curve.

2.4.2. WATER SURFACE ANGLE:

To obtain the most desirable hydraulic conditions, the angle between the water surface and the transition centre line should not exceed $27\frac{1}{2}°$ for inlet transitions and $22\frac{1}{2}°$ for outlet transitions. For some structure designs it may be economical to use an angle of $25°$ to allow the same concrete transition to be used for both inlet and outlet. For this angle the loss coefficients remain 0.5 for the inlet and 1.0 for the outlet.

2.5. OPTIMUM LENGTH OF TRANSITIONS:

G.Formica[33] conducted experiments on expansion transitions with different lengths viz. blunt (0:1), 1:1,2:1, 3:1, 4:1 and so on and B_c/B_f ratio as 2 and for varying discharge. He concluded that energy loss in a sudden expansion can be reduced by gradually enlarging the channel section or decreasing the angle of divergence. The length of gradual expansion has a limit beyond which the gain in efficiency becomes insignificant.

Smith and Yu[17] found that the rate of side wall flare required to avoid flow separation is 10:1 i.e. total central angle of $11°$-26'. Separation occured when central angle was increased to $19°$, except at values of B_c / B_f less than 2.

Rao and Seethuramaiah[19] found that the flow pattern in diverging channels are different for different angles of divergence. When angle of divergence is small i.e. less than $10°$ (total), the flow is symmetrical and pressure recovery is maximum and also pressure distribution is very uniform. If divergence angle exceeds $25°$(total), the flow clings to only one side of the channel and velocity head recovery is as low as 25%. If angle of divergence exceeds $60°$, the flow can be considered as jet flow leaving sides and velocity head recovery is negligible.

In open-channel expansion with sub-critical flow, as the length of transition is increased, the hydraulic efficiency increases very rapidly in the beginning. The rate of increment in efficiency, however, decreases with subsequent increment in length and the maximum efficiency is attained at a particular length of transition. If the length is increased further, the efficiency falls.

The maximum overall hydraulic efficiency for the type of curve tested (15) was 75% and was attained at a splay of 1 in 7, i.e., an angle of divergence of 8^o to the axis.

The hydraulic efficiency is influenced by flow characteristics, e.g., depth, discharge, etc. when the length of transition is small, efficiency increases with decrease in discharge, but at greater length of transition efficiency falls with decrease in discharge and rises with increasing discharge. Effect of Froude number, when high is not much; but when Froude number is low, there is a marked improvement in efficiency.

In deciding optimum length of transition in open channel expansions, comparative study should be made between the gain in hydraulic efficiency achieved by increment in length and the loss in revenue due to additional cost of construction arising from such increment in length. Optimum length of transition for higher discharges is greater than for lower ones; but it is independent of Froude number of flow.

The separation can be prevented at a very slow expansion rate which is not economical from points of view of energy's recovery and cost.

The velocity distribution after expansion becomes highly non-uniform with the result that although the average velocity of flow decreases, the local velocity remains considerably high, which if allowed to persist in erodible channels will scour away bed and sides of channel.

In deciding the efficiency of a transition, therefore, a second requirement beside energy recovery is that velocity after expansion should be distributed as uniformly as experienced in normal open-channel flow. In other words, the energy-correction factor for the exit should also be almost the same as at the entry of the transition.

2.6. SYPHON TRANSITION:

The design of syphon inlet and outlet transitions between canal and an inverted syphon is similar to that for the transitions between canal and flume. In the design of the inlet to a syphon, it is necessary to provide a water " seal " above the syphon opening at the inlet head wall. This "seal" is equal in height to the vertical drop from the inlet normal water surface to the top of the syphon opening, and it may be calculated hydraulically or set as an empirical minimum, depending upon design requirements and size of syphon.

Special considerations to hydraulic conditions must be given to the inlets on long syphons where, under certain conditions, the inlet will not become sealed & the hydraulic gradient will intersect the bottom of the conduit some distance away from the inlet. On long syphons, such condition may result from operation on partial flows, or at full flow when the actual coefficient of friction is less than assumed in the design. Under such conditions, a hydraulic jump occurs in the pipe and may cause very unsatisfactory operating conditions.

It is considered desirable to have the top of the syphon barrel at the head wall of the transition slightly submerged at full flow. This is thought to minimize reduction in capacity due to the introduction of air into the syphon. It is the practice of the U.S. Bureau of Reclamation, that the top of the syphon opening at the inlet head wall should be set below the approaching normal water surface a minimum distance of $1.1\Delta h_v$ to a maximum distance of $1.5\Delta h_v$ or 0.4572 metre. The amount of seal required should depend upon inclination and size of barrel, the larger seals being most applicable to the steeper inclinations and larger sizes. The seal makes it impracticable to obtain exact hydraulic computations just outside the syphon head wall. If the bottom grade is made regular and tangent to the invert of the closed transition, the channel will converge to a widthless than the syphon diameter, or if the plan is made nonconverging, the bottom grade will hump up and choke the entrance. The computed grade elevations for a short distance upstream from the inlet head wall may be arbitrarily changed to give a practicable transition structure. Therefore, the computation in Table 2.4 are carried to the end of the transition, but the computed dimensions at stations 0 + 12.19 and 0 + 13.72 are ignored. The conduit floor is drawn to a smooth connection between other computed points and the floor of thesyphon barrel with these exceptions the computations in Table 2.4 are similar to those in Table 2.2. The fact that the conduit is covered at points of high velocity makes it possible to take the liberties mentioned.

In the hydraulic design of the inlet, the seal having been determined, the velocity at the head wall is computed, the total drop in the water surface, neglecting friction, being taken as $1.1\Delta h_v$. Since the velocity is increasing at the head wall, therefore, it is not desirable to use two equal tangent parabolas for the surface profile through the inlet. A smooth flow profile is then assumed tangent to the water surface in the canal at the beginning of the transition and passing through the point at the head wall set by the computation above. There are no data available for determining the best form of the flow profile. Care must be taken in laying out the plan and profile of the transition in order that the hydraulic gradient will fit the assumed water surface curve. If they do not coincide, the plan, profile, or assumed surface should be altered as required.

As the practice (U.S.B.R.) has been established of assuming a single parabola, with a vertex at I as shown in Fig.2.25.

In the design of the syphon outlet structure, the water surface profile is taken as a single parabola, as recommended for a syphon inlet. The bottom slope of the outlet structure need not be tangent to the slope of the closed conduit at the head wall as it was in the case of the inlet, this procedure usually results in a vertical angle at the intersection of the syphon and the floor of the transition. An angle in the floor is less objectionable than an angle in the side walls, unless the syphon velocity is high and the transition slope is steep.

Recovery conditions at a syphon outlet are not the same as at a flume outlet. If a flume outlet fails to recover the head demanded of it, the water is backed up in the flume and the free-board is decreased.

It is desirable, therefore, to design for only a part of the probable recovery, as previously indicated. In the case of a syphon, failure to realize the computed recovery means a slight increase in pressure in the barrel, which is of no importance except as it affects the elevation of the water surface above the intake to the syphon. If the outlet is properly set any excess energy will be consumed at the inlet, or in the canal above the inlet. If the outlet is set too high the excess energy will be destroyed below the transition and may cause undesirable disturbances. It is advantageous, therefore, to set the outlet for the fullest recovery likely to be obtained, although in computing the overall losses through the syphon proper, allowance should be made for outlet loss.

2.7. TRANSITION LOSSES IN OPEN CHANNELS:

When a fluid has to be transported from one point to another it involves work and consequent expenditure of energy. The extent of this work depends on the nature of resistance the fluid has to over come during its motion. The energy required for this work may be available within the fluid itself. The losses in open channel flow, although similar to the losses in flow through pipes, are more complex.

Losses in pipes are classified as " skin friction loss " and ' form loss '. Skin friction loss is the resistance offered by the boundari.. in contact with the fluid. Form losses are caused by obstacles like valve, bends, expansions, contractions, etc.

Energy losses in open channels, as in pipes, are classified into two parts, namely, friction and form losses. Friction loss results from the resistance offered to the motion of the fluid by the roughness elements in the channel bed. This friction loss is a function of slope, hydraulic radius and the nature of bed surface. Form losses in open channel result from change in the flow condition caused by change in configuration in the channel, shape and slope. The change in flow condition may be in the form of :- (i) secondary flow, with circulation as in bends; (ii) stagnant or slack flow, with or without active eddies as in the case of sudden expansion or contraction; (iii) hydraulic jump with turbulent mixing of fluid particles; (iv) stratified flow, etc. In all these cases, the eddies dissipate considerable energy either in maintaining themselves in random motion or in colliding with one another (turbulent mixing). The energy is transformed into friction heat.

2.7.1. FLUME INLET LOSSES:

At the intake to the flume, the water surface must drop a distance sufficient to produce the necessary increase in velocity and to overcome friction and transition losses. Ordinarily, the friction loss is small and may be neglected, but it is included in computations in order to show how it may be handled, where necessary, and to give an idea of its actual amount. The drop in the water surface necessary to produce the required flume velocity is found by subtracting the velocity head in the canal from that in the flume (Δh_v).

It will be noted that of the twenty-nine sets of observations (by U.S.B.R.), only three observations show a loss greater than $0.1 \Delta h_v$

and only one greater than $0.14 \Delta h_v$. In a number of cases there appeared to be no loss, and the average loss is about $0.04 \Delta h_v$. In several cases the observed loss actually appeared to be negative, probably due to error or inaccuracy in measurements. If the negative values are included, the average observed loss is approximately zero.

2.7.2. FLUME OUTLET LOSSES:

The results shown for flume outlets are somewhat erratic[2]. Several apparently good outlets show high losses. The average loss for the sixteen observations on fourteen outlets is approximately $0.31 \Delta h_v$.

The observations made and data collected during the experiments,[16] indicate that the angle of expansion does not have much effect on the energy loss. In all cases for a given expansion ratio and a given discharge, the energy loss $H_L = H_2 - H_1$, was similar for varying angles of expansion at the transition. The angle of expansion, however, has considerable effect on the flow conditions at the transition. The slack water zone, covered with vortices and eddies, decreases in area and activity with the decrease in angle of expansion (Fig.2.19(a-e). The transitions with divergence angle of $12\frac{1}{2}°$ do not have slack water pocket and the flow follows close the boundary.

The energy loss increases with the increase in discharge and also increases with increase in expansion ratio (Fig.2.20). The dimensional plot reveals the following relationship between head loss and discharge for any given expansion ratio :-

Expansion ratio $B_c : B_f$	Head loss as a function of discharge	Coefficient (metric)
6.5	$H_L = C_1 Q^{0.6}$	$C_1 = 2.01$
4.0	$H_L = C_2 Q^{0.6}$	$C_2 = 1.61$
3.0	$H_L = C_3 Q^{0.6}$	$C_3 = 1.46$
2.0	$H_L = C_4 Q^{0.6}$	$C_4 = 0.50$

The dimensionless ration H_L/H_1 (the energy loss to the energy of upstream flow) decreases and approaches zero as H_2/H_1 (the ratio of energy downstream to energy of upstream flow) approaches unity (Fig.2.21). Similar result is exhibited in Fig.2.22, where H_L/H_1 approaches zero assymptotically as B_c/B_f approaches unity. There is no expansion loss when the channel width upstream and downstream are same. Skin friction losses in the short transition length are small and can be considered as of second order importance.

2.7.3. SYPHON INLET LOSSES:

Of the five tests recorded (by U.S.B.R.) for four syphon inlets, three show inlet losses of zero. One of the remaining two observations shows a loss of $0.33\Delta h_v$ and of $0.15 \Delta h_v$ making the average for all five approximately $0.10 \Delta h_v$. As previously explained, all the excess head in a syphon is concentrated at the inlet. This makes it difficult to secure a rating under normal operating conditions. Both the tests shown for syphon(2) were taken under adverse circumstances, a convas bag filled with water being suspended in the syphon barrel to cause the inlet to run full. The information shown indicates that properly designed syphon inlets, operating under full submergence, cause very little loss.

2.7.4. SYPHON OUTLET LOSSES:

The average loss for the eight tests recorded[2] on five syphon outlets is $0.07 \triangle h_v$. Five of the tests show no loss. No explanation is available for the apparently high loss through outlet. It is possible that this observation is in error. No important wave action is apparent. The data indicate that losses for well-proportioned syphon outlets may be expected to be smaller than $0.1 \triangle h_v$.

2.8. EXAMPLE OF A FLUME INLET:

Referring to Fig.2.23 let it be required to design an inlet from an earthen canal with a bottom width of 5.4864 metre, side slopes of 2:1 to a rectangular concrete flume, 3.8100 metre wide. The hydraulic properties of the canal and the flume are given in Fig.2.23. The design discharge is 8.9068 cumecs.

1. It is desirable first to determine approximately the length that will be required for the transition. The length of transition is determined so that a straight line joining the flow line at the two ends of the transition will make an angle of about 12.5^o with the axis of the structure. This length in the design is found to be 15.24 m.

2. Determination of the flow profile neglecting friction :-

For a structure of the type contemplated, the entrance loss may be safely assumed as 0.1 of the change in velocity head, or $0.1 \triangle h_v$.

The total drop to be provided is equal to 1.1 Δh_v plus the drop necessary to overcome friction. The change in velocity head from v = 0.8382 m/s. to 1.8227 m/s is equal to Δh_v = 0.1685-0.0356 = 0.1329. Neglecting the channel friction for the time being, the total drop in water surface is, therefore, 1.10 x 0.1329 = 0.1463 metre.

For a smooth and continuous flow, the theoretical flow profile may be assumed as two equal parabolas, tangent to each other at point B and horizontal, respectively, at A and C, as shown in Fig.2.23. Strictly, the parabolas should be tangent to the water-surface slopes in the canal and the flume, but the small divergence of these slopes from the horizontal is unimportant in the present example.

3. Computation of the flow profile including friction :

The computations are shown in Table 2.2 is as follows :

Number of stations equally spaced every 1.524 metre and measured in the direction of flow.

<u>Line 1:</u> Drop in water surface (Δ WS) from A to C, neglecting friction, is 0.1463 metre. The drop at mid point B is equal to 0.0731 m. The drops between A to B and B to C is computed as per paragraph 2.4.1.

<u>Line 2:</u> Change in velocity head. 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 values of Δ WS in line 1 by 1.1.

<u>Line 3:</u> The velocity head is obtained by adding Δh_v to the velocity head at A. Or the total velocity head, equal to the cumulative value of Δh_v entering the preceding column.

Line 4: The velocities corresponding to the assumed water surface curve are determined, or corresponding to the velocity head in the preceding column.

Line 5: The required area of cross-section is computed by dividing the discharge to the velocity in preceding column.

Line 6: Half the top width is obtained from the cross-sections of sketched plan (Fig.2.23). The plan may be chosen either arbitrarily or by trial until satisfactory results are obtained. The choice of a proper shape for the plan is a matter of judgement.

Line 7: Half the bottom width is obtained from the sketched plan.

Line 8: Average width = $0.5\ B + 0.5\ B_s$

Line 9: Average depth of flow, equal to Area /(.5 top width + 5 bottom width).

Line 10: The friction slope, computed by $S_f = \frac{n^2\ V^2}{2.22\ R^{4/3}}$ (Manning formula), where n = 0.014 for all sections in the transition.

Line 11: The friction head, equal to the distance between stations, or 1.524 metre, multiplied by the average of the friction slope of the section and that of the preceding section.

Line 12: Cumulative friction head.

Line 13: The water-surface elevation, including the effect of the channel friction, equal to 17.4987 (water surface elevation at 0 + 00) $- \Delta WS - \sum h_f$.

Line 14: The elevation of the channel bottom, equal to water surface elevation - depth of flow.

From line 15 to Line 20 related to structural dimensions of the transition, are self-explanatory.

2.9. EXAMPLE OF A FLUME OUTLET:

An outlet transition from a flume is designed in the same way as an inlet the only essential difference being that the conversion loss is subtructed from Δh_v to obtain Δ WS. A typical design for an outlet from a rectangular flume to an earthen canal is shown in Fig.2.24. In the example shown, the length of the outlet structure is determined on the same basis as the length of the inlet structure. It is now generally conceded that for equal efficiency an inlet may be made shorter than an outlet, for the same velocity change. The experience of the U.S. Bureau of Reclamation indicates that for properly designed outlets of the type shown in Fig.2.24, a length as determined by making $\phi = 12\ 1/2^{o}$ is sufficient for ordinary purposes. Inlets are generally made the same length because: (a) sharper warps are difficult to construct; (b) short transitions do not afford secure anchorage to the canal; (c) using the same length makes the forms interchangeable; (d) an average divergence of $12\ 1/2^{o}$ yields a structure of pleasing appearance and reasonable cost.

Table 2.3 is similar to Table 2.2. It will be noted that an allowance of $0.2\,\Delta h_v$ is made for outlet losses, which is somewhat more than might be expected from existing experimental data. Any excess in possible recovery of head over the computed recovery affords a small margin of safety against a reduction of free-board due to fouling of the canal below the flume. If there is any doubt as to the maintenance of a clean channel below the flume a greater allowance for outlet loss should be made. In extreme cases it may be desirable to assume no recovery at flume outlets, even for structures of careful design. The full actual recovering capacity of the outlet is thus made available for raising the canal water surface above normal without encroaching on the computed free-board in the flume.

Where an allownace is made for a greater outlet loss than is actually necessary, the destruction of the excess energy must be considered. If the flume velocity is relatively low, the depth being greater than the critical depth, a recovery will actually occur at the outlet whether provided for or not. The velocity in the flume will be increased because of the excess head and the depth decreased, making recovery necessary. If flow in the flume is at or below the critical depth no draw-down above the outlet is possible, and the excess energy must be dissipated in waves and eddies or by a hydraulic jump; in the canal or in the outlet. The resulting eddies may cause the canal to erode.

2.10. EXAMPLE OF A SYPHON INLET:

The design of a transition from an earth canal with a 3.6576 m. bottom width to a syphon 2.1336 metre in diameter is illustrated in Fig.2.25 and Table 2.4. At syphon inlet the design follows the same general principles as for flumes. The water surface profile is taken as one parabola as mention in design section for syphon.

2.11. EXAMPLE OF ASYPHON OUTLET:

The necessity for holding the top of the syphon barrel down at the head-wall does not exist at a syphon outlet, as in case of inlet. The computed dimensions may be followed throughout. Fig.2.26 represents a typical outlet design, the computations for which are shown in Table 2.5.

TABLE - 2.4 - COMPUTATIONS FOR SYPHON INLET

In Syphon, V = 1.2954 h_v = 0.0853
In canal, V = 0.7040 h_v = 0.0252
Δh_v = 0.0600

Elevation of water surface at 0+00 = 29.2062
Entrance loss = 0.1 Δh_v
Water Surface, single parabola

Line	Item	Station 0+00	0+1.52	0+3.05	0+4.57	0+6.10	0+7.62	0+9.14	0+10.67	0+12.19	0+13.72
1	Δ WS=Drop in WS*	—	0.0009	0.0033	0.0073	0.0131	0.0204	0.0292	0.0399	0.0524	0.0661
2	Δh_v=WS ÷ 1.1	—	0.0006	0.0030	0.0067	0.0118	0.0185	0.0268	0.0362	0.0475	0.0600
3	h_v= 0.0252+ Δh_v	0.0252	0.0259	0.0283	0.0320	0.0371	0.0435	0.0521	0.0615	0.0728	0.0853
4.	V	0.7040	0.7132	0.7437	0.7924	0.8534	0.9235	1.0119	1.0972	1.1948	1.2923
5	Area = Q ÷ V	8.5024	8.2683	7.9059	7.4414	6.8933	6.3730	5.8342	5.3604	4.9238	4.5522
6.	0.5 B_s= half width at WS	4.5110	4.3282	3.9776	3.5357	2.9870	2.3317	1.8135	1.3868	1.1582	1.0668
7.	0.5 B=Half bottom width	1.8288	1.8135	1.7526	1.6459	1.4935	1.3258	1.2192	1.0972	1.0668	1.0668
8.	0.5 B_s+0.5B=Ave. width	6.3398	6.1417	5.7302	5.1816	4.4805	3.6575	3.0327	2.4840	2.2250	2.1336
9.	d=Area ÷ Av.width	1.3411	1.3457	1.3853	1.4356	1.5377	1.7282	1.9233	2.1580	2.2128	2.1336
10.	S_f=Friction slope	.00011	.00012	.00013	.00014	.00016	.00018	.00022	.00030	.00045	.00074
11.	h_f=1.524 S_f(using ave.S_f)	--	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0006	0.0009
12.	Σh_f	--	0.0003	0.0006	0.0009	0.0012	0.0015	0.0018	0.0021	0.0027	0.0036
13.	WS Elev.=29.2062- ΔWS- Σh_f	29.2062	29.2047	29.2023	29.1980	29.1919	29.1846	29.1748	29.1642	29.1511	29.1364
14.	Grade=WS Elev.-d	27.8651	27.8590	27.8169	27.7624	27.6542	27.4561	27.2515	27.0062	26.7008	26.3960
15.	0.5 B_s-0.5 B	2.6822	2.5146	2.2250	1.8897	1.4935	1.0058	0.5943	0.2895	0.0914	-
16.	Side slopes	2.0000	1.8700	1.6000	1.3160	0.9700	0.5820	0.3090	0.1340	0.0330	-
17.	h= height of lining	1.7648	1.7678	1.8135	1.8684	1.9751	2.2464	2.5237	2.8407	3.2187	3.5966
18.	0.5W-0.5B=side slope x h	3.5296	3.3071	2.9017	2.4567	1.9141	1.3045	0.7802	0.3810	0.1225	-
19.	0.5W=0.5 top width	5.3584	5.1207	4.6543	4.1026	3.4077	2.6304	1.9995	1.4782	1.1887	1.0668
20.	0.5W to nearest 1.27 cm.	5.3594	5.1308	4.6609	4.1021	3.4036	2.6289	1.9939	1.4732	1,1937	1.0668

TABLE - 2.5 - COMPUTATIONS FOR SYPHON OUTLET

In syphon, $V = 1.1978$ $h_v = 0.0731$
In canal, $V = 0.7040$ $h_v = 0.0252$
$\Delta h_v = 0.0478$

Elevation of water surface at 0+13.72= 29.0447
Outlet loss neglected.

Water surface, single parabola

Line	Item	0+00	0+1.52	0+3.05	0+4.57	0+6.10	0+7.62	0+9.19	0+10.67	0+12.19	0+13.72
1.	$\Delta h_v = \Delta$WS=Rise in WS*	—	0.0100	0.0188	0.0265	0.0332	0.0384	0.0426	0.0454	0.0472	0.0478
2.	$h_v = 0.0731 - \Delta h_v$	0.0731	0.0630	0.0542	0.0466	0.0399	0.0347	0.0304	0.0277	0.0259	-
3.	V	1.1978	1.1125	1.0302	0.9570	0.8839	0.8260	0.7742	0.7376	0.7132	-
4.	Area = Q ÷ V	4.9191	5.2954	5.7060	6.1594	6.6518	7.1581	7.6179	7.9895	8.2683	8.5024
5.	$0.5\ B_s$= Half width at WS	1.0668	1.2496	1.5148	1.8623	2.2860	2.8194	3.3680	3.8801	4.2763	4.5110
6.	0.5B=Half bottom width	1.0668	1.1734	1.2649	1.3563	1.4630	1.5697	1.6764	1.7678	1.8135	1.8288
7.	$0.5B_s$+0.5B=Av. width	2.1336	2.4230	2.7797	3.2186	3.7490	4.3891	5.0444	5.6479	6.0898	6.3398
8.	d=Area - Av. width	2.3073	2.1884	2.0513	1.9141	1.7739	1.6337	1.5087	1.4142	1.3563	1.3411
9.	WS Elev.=28.9968+ ΔWS	28.9968	29.0069	29.0157	29.0234	29.0301	29.0352	29.0359	29.0423	29.0441	29.0447
10.	Grade=WS Elev.-d	26.6895	26.8184	26.9644	27.1092	27.2561	27.4045	27.5307	27.6280	27.6877	27.7036
11.	$0.5\ B_s - 0.5$ B	-	0.0762	0.2499	0.5059	0.8229	1.2496	1.6916	2.1122	2.4628	2.6822
12.	Side slopes	-	0.3480	0.1220	0.2640	0.4630	0.7660	1.1220	1.4930	1.8150	2.0000
13.	h=height of lining	3.1507	2.9547	2.7417	2.5298	2.3158	2.1004	1.9071	1.8099	1.7501	1.7343
14.	0.5W-0.5B=Side slope x h	-	0.1027	0.3346	0.6675	1.0729	1.6093	2.1412	2.7066	3.1791	3.4686
15.	0.5W=0.5 top width	1.0668	1.2762	1.5996	2.0238	2.5359	3.1791	3.8176	4.4745	4.9926	5.2974
16.	0.5W to nearest 1.27 cm	1.0668	1.2699	1.6002	2.0193	2.5400	3.1750	3.8100	4.4704	4.9784	5.2959

*Friction and outlet loss ignored in detailing outlet structure. In computing total drop through syphon allow for outlet loss = $0.2\ \Delta h_v + \Sigma h_f$

TABLE - 2.2 - COMPUTATIONS FOR FLUME INLET

In Flume, $V = 1.8227$ $h_v = 0.1685$

In Canal, $V = 0.8382$ $h_v = 0.0356$

$\Delta h_v = 0.1329$

Elevation of water surface at 0+00 = 17.4987

Entrance loss = $0.1 \Delta h_v$

Water Surface, reversed Parabola

Line	Item	STATION										
		0+00	0+1.52	0+3.05	0+4.57	0+6.10	0+7.62	0+9.14	0+10.67	0+12.19	0+13.72	0+15.24
1.	Δ WS=Drop in WS*	-	0.0030	0.0115	0.0262	0.0469	0.0731	0.0993	0.1200	0.1347	0.1432	0.1463
2.	$\Delta h_v = \Delta WS \div 1.1$	-	0.0027	0.0106	0.0240	0.0426	0.0664	0.0902	0.1088	0.1222	0.1301	0.1328
3.	$h_v = 0.0356 + \Delta h_v$	0.0356	0.0384	0.0463	0.0597	0.0783	0.1021	0.1258	0.1444	0.1578	0.1658	0.1685
4.	V	0.8382	0.8686	0.9540	1.0820	1 2405	1.4142	1.5697	1.6825	1.7587	1.8013	1.8196
5.	$A = Q \div V$	10.6280	10.2657	9.3459	8.2450	7.1906	6.3062	5.6856	5.3047	5.0724	4.9516	4.8959
6.	$0.5\ B_s$=Half width at WS	5.3645	5.1816	4.7022	4.1026	3.4223	2.7856	2.3521	2.0869	1.9684	1.9248	1.9050
7.	0.5B = Half Bottom Width	2.7432	2.6289	2.4131	2.2098	2.1208	2.0638	2.0321	2.0004	1.9684	1.9248	1.9050
8.	$0.5\ B_s + 0.5B$=Av. width	8.1077	7.8105	7.1153	6.3124	5.5431	4.8494	4.3842	4.0873	3.9368	3.8495	3.8100
9.	d=area ÷ av.width	1.3106	1.3133	1.3137	1.3045	1.2984	1.2996	1.2960	1.2963	1.2877	1.2862	1.2862
10.	S_f=Friction slope	0.00015	0.00017	0.00020	.00026	.00034	.00046	.00061	.00076	.00083	.00087	.00090
11.	$h_f = 1.524\ S_f$ (use Ave. S_f)	-	.0002	.0002	.0003	.0004	.0006	.0008	.0010	.0012	.0012	.0013
12.	Σh_f	-	.0002	.0010	.0013	.0017	.0023	.0031	.0041	.0053	.0065	.0078
13.	WS Elev.=17.4987 - $\Delta WS - \Sigma h_f$	17.4987	17.4954	17.4865	17.4716	17.4506	17.4237	17.3966	17.3747	17.3588	17.3491	17.3448
14.	Grade= WS Elev.-d	16.1881	16.1820	16.1728	16.1670	16.1521	16.1241	16.1006	16.0783	16.0710	16.0628	16.0585
15.	$0.5\ B_s - 0.5\ B$	2.6213	2.5527	2.2890	1.8928	1.3015	1.7217	0.3200	0.0865	-	-	-
16.	Side Slopes	2.000	1.945	1.744	1.447	1.000	0.554	0.247	0.067	-	-	-
17.	h = Height of lining	1.6246	1.6139	1.6063	1.5953	1.5935	1.6047	1.6114	1.6169	1.6075	1.5950	1.5865
18.	0.5 W-0.5B=Side Slope x h	3.2492	3.1425	2.8072	2.3088	1.5935	0.8900	0.3977	0.1079	-	-	-
19.	0.5W = 0.5 top width	5.9924	5.7714	5.2203	4.5187	3.7143	2.9538	2.4298	2.1083	1.9684	1.9248	1.9050
20.	0.5W to nearest 1.27 cm	5.9944	5.7658	5.2197	4.5212	3.7084	2.9591	2.4257	2.1082	1.9685	1.9304	1.9050

*Neglecting friction Temporarily

TABLE - 2.3 - COMPUTATIONS FOR FLUME OUTLET

In Flume, V = 1.8227 h_v = 0.1685 Elevation of water surface at 0+00 = 15.5998

In Canal, V = 0.9875 h_v = 0.0496 Outlet loss = 0.2 Δh_v

Δh_v = 0.1188 Water Surface, reversed parabola

Line	Item	STATION 0+00	0+1.52	0+3.05	0+4.57	0+6.10	0+7.62	0+9.14	0+10.67	0+12.19	0+13.72	0+15.24
1.	ΔWS=Rise in WS*	-	0.0018	0.0076	0.0170	0.0304	0.0475	0.0646	0.0780	0.0874	0.0932	0.0950
2.	Δh_v = ΔWS ÷ 0.80	-	0.0024	0.0094	0.0213	0.0381	0.0594	0.0807	0.0975	0.1094	0.1164	0.1188
3.	h_v=0.1685 - Δh_v	0.1685	0.1661	0.1591	0.1472	0.1304	0.1091	0.0694	0.0710	0.0591	0.0521	0.0496
4.	V	1.8227	1.8044	1.7678	1.6977	1.6002	1.4630	1.3106	1.1795	1.0759	1.0119	0.9875
5.	Area= Q ÷ V	4.9033	4.9470	5.0445	5.2536	5.5741	6.0962	6.8097	7.5622	8.2868	8.8164	9.0319
6.	0.5B_s=Half width at WS	1.9050	1.9178	1.9684	2.1000	2.3847	2.8346	3.4235	4.0264	4.5110	4.8799	5.0597
7.	0.5B=half bottom width	1.9050	1.9178	1.9684	2.0202	2.0702	2.1464	2.2226	2.3369	2.5018	2.6670	2.7432
8.	0.5B_s+0.5B=Ave. width	3.8100	3.8356	3.9368	4.1202	4.4549	4.9810	5.6461	6.3633	7.0128	7.5469	7.8029
9.	B=Area ÷ Av.width	1.2862	1.2899	1.2807	1.2746	1.2509	1.2247	1.2054	1.1887	1.1814	1.1689	1.1582
10.	S_f=Friction slope	.00090	.0008	.00085	.00075	.00063	.00051	.00041	.00033	.00027	.00024	.00023
11.	h_f=1.524 S_f(using ave.S_f)	-	0.0015	0.0012	0.0012	0.0009	0.0009	0.0006	0.0006	0.0006	0.0003	0.0003
12.	Σh_f	-	0.0015	0.0027	0.0039	0.0048	0.0057	0.0063	0.0069	0.0075	0.0078	0.0081
13.	WS Elev.=15.5989+ ΔWS-Σh_f	15.5998	15.6001	15.6047	15.6129	15.6254	15.6416	15.6580	15.6708	15.6797	15.6851	15.6867
14.	Grade = WS Elev. - d	14.3135	14.3102	14.3239	14.3382	14.3745	14.4169	14.4525	14.4821	14.4982	14.5162	14.5284
15.	0.5 B_s-0.5 B	-	-	-	0.0798	0.3145	0.6882	1.2009	1.6895	2.0397	2.2128	2.3165
16.	Side Slope	-	-	-	0.0625	0.2470	0.5530	0.9970	1.4220	1.7270	1.8920	2.0000
17.	h = Height of lining	1.5880	1.6017	1.5983	1.5944	1.5685	1.5365	1.5112	1.4920	1.4862	1.4481	1.4676
18.	0.5 W-0.5B=Side slope x h	-	-	-	0.0993	0.3871	0.8504	1.5057	2.1214	2.5644	2.7432	2.9535
19.	0.5W= 0.5 top width	1.9050	1.9178	1.9684	2.1199	2.4573	2.9968	3.7283	4.4583	5.0682	5.4101	5.6967
20.	0.5W to nearest 1.27 cm	1.9050	1.9177	1.9685	2.1209	2.4637	2.9972	3.7338	4.4577	5.0673	5.4102	5.7023

*Neglecting friction temporarily

2.12. SEPARATION CONTROL DEVICES:

In open channels, using gradual rate of flaring implies a great length of the walls which may prove prohibitively costly in comparison with the saving achieved throughout constriction. It is also seen that the performance of such expansions of long length is not upto the expectation. Not only is the head recovery poor but also the velocity distribution at the exit of the expansion is extremely non-uniform. Moreover, the flow is seen to separate from the boundary giving rise to undesirable eddies and rough flow in the tail channel. By giving some complicated shape of the expansion walls, very little improvement in performance takes place. Non-uniformity of the velocity distribution at exit results in erosion of tail channel for a considerable length beyond the exit of the expansion.

Usually the engineer is interested in minimising the length of the structure in order to minimise construction cost, which requires maximising the rate of increase of cross-section flow area. Thus a balance must be sought between economics and importance of minimising energy losses, as well as taking into account downstream erosion if an earthen outlet channel is to be used.

Adoption of appurtenances in an expansion so as to spread the flow and avoid scour in tail channel was initiated by Rao, in Poondi Research Station, Madras (India). Simons of United States Department of the Interior used wedges for spreading the flow. The head loss was considerably high. Smith and Yu [17] developed baffles of rectangular section for achieving uniform distribution of velocity after expansions. The head loss, in this case too, was considerable and wake developed behind each of the baffles. Mazumdar[38] adopted a streamlined expansion having a shape similar to the boundary of the eddy in a sudden expansion. The performance was tested for five different lengths and for several discharges and depths of flow.

Rouse[11] has suggested the use of splitter walls (Fig.2.27) in the outlet transitions of shorter length for reducing zone of separated flow and improving the velocity distribution. The number and thickness of walls should be decided on the basis of permissible head loss, as the increase in number of splitter walls will increase the head loss.

Kulandaiswamy[18] conducted experiments on expanding transitions with warped side walls and an adversely sloping bed. The side slope varied from vertical in the flume to 1/2 : 1 in the normal section. The side splay was 4:1. Studies were made for three cases : (i) expanding transition (Fig.2.28(A)), (ii) transition (i) with bed deflector (fig.2.28 (B)), (iii) transition (i) with vanes (Fig.2.28(C)). In the investigation by Kulandaiswamy measurements of velocity distribution and energy loss were made and it was observed that the transition with bed deflector or vane gave consistently better performance than the plane transition of the same splay. The vanes were found to be more efficient than the bed deflector.

Rao and Seetharamaiah[19] conducted experiments for controlling separation in wide angle diverging channels ($> 35^o$) for improving the velocity head recovery. Three types of devices (Fig.2.29) e.g. trihedral sill deflecting plates and splitter plates were tried. The main aim to use these devices was to divert the flow towards the boundaries to eliminate eddy region on sides.

Ramamurthy, Basak and Rao[20] conducted experiments in a flume of 1.2192 metre length. Humps of 0.0063 m and 0.0126 metre were tested in the model. It was found that use of simple hump set in a transitional structure was effective to reduce flow separation and limit the area in which the reversal of flow occurs. This is desirable as the decrease in flow reversal contributes to the reduction of possible damage to bed and sides of downstream channel.

Skogerboe, Austin and Bannet[21] suggested the use of triangular vanes in expansion transitions as shown in Fig.2.30.

Basic studies on expansion transition were carried at Irrigation Research Station, Poondi, Madras[22] (India) 1951. They have recommended a shorter length of exit transition of splay 4:1 built with warped sides straight in plan & providing either of the following separation control device.

(i) Two vanes commencing from the throat diverging downstream in plan leaving a small gap at the centre and having sloping crests (Fig.2.31) which effectively cause a greater obstruction to flow in centre and less on the sides so that high velocity flow is deflected towards the sides thereby distributing the flow evenly.

(ii) Bed Deflector Sill. It also functions on the same principles as vanes (Fig.2.32).

Both devices were found to function satisfactorily even for half F.S.L. (Full Supply Level) conditions. It may also be mentioned that while deflecting vanes will be convenient for adoption in cross drainage works, bed deflector will be particularly suited in problem of fluming of canal through rock cuttings.

An expansion may be designed for achieving any one or more of the following performances: (i) high recovery of head so that afflux is minimum, (ii) uniform distribution of velocity at exit so that there is no erosion in the tail channels and (iii) smooth flow in the tail channel free from eddies. Although such objectives may be partially fulfilled by providing a long length of expansion, the difficulties with such design have already been pointed out. It is felt, therefore, that some kind of appurtenances should be used in an effort to curtail the length, simplify the construction and at the same time improve the performance. The methods devised by several workers in this respect have already been described. From the comparative results of the preliminary experiments, it was found that the pair of triangular vanes converging downstream in plan and placed symmetrically near the commencement of expansion (Fig.2.28(C)) gave very encouraging results. Such vanes were initially used in Poondi Madras (India).

2.13. STRUCTURAL DESIGN :

2.13.1. WALL THICKNESS:-

The minimum thicknesses for walls are shown on Fig.2.33 and Table 2.6. These thicknesses are determined by the requirements of concrete cover and concrete placement. The dimensions shown are minimum only and should be increased where required by stress on other considerations. The minima shown are further restricted by the following requirements:-

(i) Walls having two-layer reinforcement shall be 0.2031 metre thick minimum.

(ii) Cantilever walls shall have a minimum thickness at the base equal to 8.3 cm. per metre of height upto 2.4384 metre; above 2.4384 metre the minimum thickness at the base shall be 0.2032 metre plus 6.25 cm. for each metre in height above 2.4384 metre. The walls may be tapered uniformly from bottom to top, but the top thickness should not be less than 0.2032 m. for two layer reinforcement. For architectural reasons a copying may be added to give the structure a substantial appearance.

2.13.2. SLAB THICKNESSES :

The thickness of slabs connected to walls carrying bending moment should normally be the same as the adjacent wall thickness at their junction (except as required for additional concrete cover) in order to carry design moments from wall to floor. The thickness of slabs connected to walls not carrying bending moment may be, but need not be, the same as adjacent wall thickness, and both may vary in thickness independently if all design requirements are met. Floors, walls, and slope slabs poured against ground or rock shall have a minimum protective cover for the reinforcement as shown in Table 2.7. Backformed walls may be of less thickness than adjacent slabs, by the amount of additional concrete cover required for the slabs.

For small transitions, or those of moderate width, the floor thickness may be held constant across the width; but for the wider floors it may be desirable to taper the floor or reduce the thickness of a

TABLE 2.6

MINIMUM THICKNESS FOR SLABS & PAVINGS

	Minimum total thicknesses as determined by clear distance requirements				
	TOP FACE		BOTTOM FACE		Total thickness
	Exposure condition	C.D.	Exposure condition	C.D.	
Single layer Reinforcement	Unexposed to weather	0.019 m	Placed against forms	0.025	0.076
			Placed against earth or rock	0.051	0.102
	Exposed to weather or backfill. Reinf. 5/8 Ø or less	0.038	Placed against forms	0.025	0.089
			Placed against earth or rock	0.051	0.114
	Top of footing	0.038	Placed against earth or rock	0.076	0.140
Double layer Reinforcement	Bridge slabs or unexposed to weather	0.025	Placed against forms	0.025 0.038*	0.152 0.165*
			Placed against earth or rock	0.076	0.203
	Exposed to weather or backfill. Reinf. 5/8 Ø or less	0.038	Placed against forms	0.025 0.038*	0.165 0.178*
			Placed against earth or rock	0.076	0.216
	Top of footing	0.038	Placed against earth or rock	0.076	0.216

NOTES:

C.D. denotes clear distance between face of concrete and nearest reinforcement bar.
Thickness based upon 1/2 Ø bars, and 0.051 m. clear distance between layers.

* Exposed to weather.

All dimensions are in metre.

Main Reinforcement otective covering reinforcement from tside of bar to rface of Monolithic ncrete	Pavings 0.229 m thick & less	0.229 m thick exclusive of wearing surface	phon rrels	Tunnels	Galleries in dams
1	2	3	0	11	12
exposed to weather, no fire hazard *	0.019	0.025	0.		0.127
re reistive		0.038	0.		
ructures exposed to ather, or backfill, submerged and can made accessible- r Nos.5 & less	0.038	0.038	0.038	0.038	
r ctures exposed to ather, or backfill, submerged and can be de accessible-Bar s.6 & 7 -		0.051	0.051	0.051	
ructures exposed to ather, or backfill submerged and can made accessible - r Nos.8 & over		0.051	0.064	0.064	
ructures submerged d can not be made accesible r Nos.7 & less		0.064	0.064	0.064	
ructures submerged and n not be made accessi- e- Bar Nos.8 & over		0.064	0.076	0.076	
ructures submerged and n not be made accessible ere failure due to rust- g would cause loss of life structure, such as water de of concrete scroll cases lrace and backfilled ls of power plants, etc.		0.076	0.076	0.076	
ncrete placed directly ainst ground, or rock, surfaces subjected to rrosive liquids **.	0.051	0.076	0.076	0.025 to A line	

The diameter of the bar shall be used as mi

Clear cover is to be tie or stir-up where

ES:

All stair slabs and landings in enclo

All underside of bridge slabs exposed d.

Special consideration shall be given to s scour, other agency requirements, etc.

All dimensions are in metres.

TABLE - 2.7 - PROTECTIVE COVER FOR REINFORCEMENT

Minimum clear cover of Main Reinforcement Protective covering of reinforcement from outside of bar to surface of Monolithic Concrete	Slabs & Pavings 0.229 m thick & less	Slabs over 0.229 m thick exclusive of wearing surface	Walls	Beams (Top & bottom)	Girders (Top & Bottom)	Beams & Girders (sides)	Columns	Footings	Syphon barrels	Tunnels	Galleries in dams
1	2	3	4	5	6	7	8	9	10	11	12
Unexposed to weather, or no fire hazard *	0.019	0.025	0.038	0.038	0.038	0.038	0.038	0.038			0.127
Fire reistive		0.038	0.038	0.051	0.064	0.038	0.064				
Structures exposed to weather, or backfill, or submerged and can be made accessible-Bar Nos.5 & less	0.038	0.038	0.038	0.038	0.038	0.038	0.038	0.038	0.038	0.038	
Structures exposed to weather, or backfill, or submerged and can be made accessible-Bar Nos.6 & 7		0.051	0.051	0.051	0.051	0.051	0.051	0.051	0.051	0.051	
Structures exposed to weather, or backfill or submerged and can be made accessible - Bar Nos.8 & over		0.051	0.051	0.051	0.064	0.051	0.064	0.051	0.064	0.064	
Structures submerged and can not be made accesible Bar Nos.7 & less		0.064	0.064	0.064	0.064	0.064	0.064	0.064	0.064	0.064	
Structures submerged and can not be made accessible- Bar Nos.8 & over		0.064	0.064	0.064	0.076	0.064	0.076	0.064	0.076	0.076	
Structures submerged and can not be made accessible where failure due to rusting would cause loss of life or structure, such as water side of concrete scroll cases tailrace and backfilled walls of power plants, etc.		0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	
Concrete placed directly against ground, or rock, or surfaces subjected to corrosive liquids **.	0.051	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.025 to A line	

* The diameter of the bar shall be used as minimum cover if greater than cover listed.

** Clear cover is to be tie or stir-up where surface is subjected to corrosive liquids.

NOTES:

All stair slabs and landings in enclosed wells; use 0.0189 M. clear cover.
All underside of bridge slabs exposed to weather, 0.025 m clear cover shall be provided.
Special consideration shall be given to any design condition not included herein such as scour, other agency requirements, etc.

All dimensions are in metres.

centré portion for the sake of economy, provided design requirements are met. There are no rigid rules for the design of transition floors, and their proportioning must depend upon judgement and experience applied within known or assumed design conditions. Floor designs may thus vary from simple slabs of constant thickness, both transversely and longitudinally, to slabs tapered or reduced in thickness and with ribs and cut-off walls both ways to fulfill structural and hydraulic requirements.

2.13.3. FLUME SECTION:

The considerations in the design of a concrete bench flume section must include the water load inside of the flume; backfill against one or both sidewalls, with and without water in the flume; and the type of foundation supporting the structure. The floor should normally be the same thickness as the side walls at its junction. Not less than one-third of the reinforcement required at the base of the side walls for water load should be carried across the floor of the flume, the remaining reinforcement ending at partial distance in the floor and sides as may be required by the balance of moments. The minimum reinforcement required for moment should not be less than the requirement for temperature reinforcement.

Concrete elevated flumes must be designed to carry the weight of structure and water between supports. This may be accomplished by designing the walls to carry the load, or by supporting the flume section on beams between supports. The substructure of elevated concrete flumes consists of piers supported on footings. Where the piers consist of slender reinforced concrete columns above the bases, which are framed together rigidly at the top, the structure takes on the character of a rigid frame and should be designed as such. Design considerations for piers include wind load acting on the piers and superstructure, dead and live load weight of structure, forces resulting from expansion or contraction of the superstructure, and the reaction of the foundation. The piers must be stable against overturning or sliding. The maximum bearing pressure on the footing should not exceed the allowable limit for the type of foundation material encountered.

2.13.4. STRUCTURAL DETAILS:

The reinforcing and structural details will depend upon design and installation conditions or local conditions. If the structure is to be completely and carefully backfilled with compacted materials, no provision need be made for inside hydrostatic pressure. However, provision should be made for any anticipated hydrostatic uplift for any expected combination of outside loading. Where necessary, counterforts or buttresses should be provided to support the weight of walls during backfilling. In general, the structural reinforcement should follow conventional design theory of cantilever and sloped walls, or, with the use of counterforts and ribs, may involve the design of slabs of various types of support and degrees of fixity.

2.13.5. CUT-OFF WALLS :

Cut-off walls should in general, be a minimum of 0.6096 metre deep and 0.1524 metre thick for water depth of 0 to 0.9144 metre over the

2.13.7. STRUCTURAL DESIGN FOR SYPHON TRANSITIONS:

The structural design of any particular syphon will, of course, vary with the shape and material. In general, recognized textbook design discussions of dead and live loading, lateral loading, foundation assumptions, frame, beam, and slab design, etc. are adopted. Some structural details and dimensions are not specifically govern by pure design, & must necessarily be determined upon the basis of experience and good practice. The structural design of open transitions are discussed above. The discussion is necessarily of a general nature applicable to the usual conditions met in practice. No rigid rules of structural design can be prescribed, and conditions of a special nature must be specifically met at the time of design.

2.13.8. FREE BOARD RULES, OPEN TRANSITIONS:

In order that designs shall be consistent as regards free boards provided in open transitions, it is desired that the following general rules be observed.

(i) In general, the free board provided will be related to the water depth in the canal. For syphons, thefreeboards at the cut-offs in unlined canals should be 0.3048 metre for water depths of 1.524 metre or less, 0.381 metre for depths from 1.524 to 2.1336 m, 0.4572 m for depths from 2.1336 to 2.7432 metre and 0.5334 metre for depths, from 2.7432 to 3.6576 metre. For these depths the freeboard at the headwalls should be twice that at the cut-off neglecting the fall or rise of the water surface in the transition. Transitions for depths over 3.6576 metre must be given special consideration.

(ii) Freeboards provided at the cut-offs of transitions to or from tunnels should be the same as specified for syphons. The top of the cut-offs; and 0.7620 metre deep and 0.2032 metre thick for water depths of 0.9144 to 1.8288 metre. For some small structures, cut-offs 0.4572 metre deep and 12.7 or 15.24 cm. thick may be satisfactory under some conditions, and obviously, dimensions greater than the above minima may be advisable under many conditions. The cut-offs have the three main functions of reducing percolation around the structure, preventing movement of the structure, and making the transitions more rigid in design and construction.

2.13.6. MISCELLANEOUS STRUCTURAL CONSIDERATIONS:

Some miscellaneous structural considerations related to transition design are as follows :-

(i) Transition should not be placed on curves if such can be avoided.

(ii) For best construction economy, inlet and outlet transitions should be designed in such a way as to allow interchange of forms. This is, however, seldom possible except on small structures, if lengths are governed by maximum water line angles.

(iii) Lengths of transitions are mainly governed by hydraulic considerations. In some cases, lengths are stipulated for certain types of structures. In other instances, the lengths are nominal or minimum, such as for small pipe structures. The designs should be produced which are well proportioned and which satisfy all known requirements and stated minimum.

2.13.7. STRUCTURAL DESIGN FOR SYPHON TRANSITIONS:

The structural design of any particular syphon will, of course, vary with the shape and material. In general, recognized textbook design discussions of dead and live loading, lateral loading, foundation assumptions, frame, beam, and slab design, etc. are adopted. Some structural details and dimensions are not specifically govern by pure design, & must necessarily be determined upon the basis of experience and good practice. The structural design of open transitions are discussed above. The discussion is necessarily of a general nature applicable to the usual conditions met in practice. No rigid rules of structural design can be prescribed, and conditions of a special nature must be specifically met at the time of design.

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(ii) Freeboards provided at the cut-offs of transitions to or from tunnels should be the same as specified for syphons. The top of the transition should be level, except that if required it should be given a uniform slope so that at the portal it will not be lower than the intrados of the portal arch.

(iii) The freeboard at the junction of syphon or tunnel transitions and lined canal sections or bench flumes should be the same as the free board of the lining or bench flume. At the headwalls of syphons whose transitions join concrete lining or bench flume, the freeboard should be twice the height at the cut-off for an unlined canal, of the same depth, computed as in Subparagraph (i) above. The top of transitions between lined canals or bench flumes and tunnel portals should be level except as provided in subparagraph (ii).

(iv) Freeboards at the cut-offs of transitions between earth canals and lined canals or bench flumes should be computed as for syphon or tunnels, and the top of the concrete should have a uniform slope to the top of the lining or flume, respectively, as provided in the design. The top of the concrete of transitions between lined canals and bench flames should slope uniformly between the tops of the lining and flume as provided in the respective designs.

(v) Deviations from the above rules will be permitted in special cases if warranted. Such cases shall be referred to the engineer in responsible charge of design.

2.14. UPLIFT PRESSURE :

Design of hydraulic structures founded on permeable soils presents problems of complex nature in respect of determination of uplift pressures and exit gradients. The floor of syphon is subjected to uplift pressures due to seepage from the drainage bed to the bed of the canal. The worst condition in this case occurs when the river is in high flood and the canal is drained at its bed. In addition to the uplift pressures below these structures, the value of the exit gradient at the end of the structure has to be taken into account to ensure stability against failure due to piping.

In case of uplift pressure on the roof of the work, when the tail water level below a syphon is higher than the under surface of the culvert covering, uplift pressure is exerted on the covering or roof. The uplift would be maximum when the highest flood is running in the drainage. The uplift pressure at the downstream end of the barrel roof is equal to the difference between the downstream water level and the level of the under side of the roof. At any other point along the barrel, the uplift pressure is given by the ordinate between the hydraulic gradient line and the underside of the roof covering. The maximum uplift pressure occurs at the upstream end of the roof.

A syphon floor is subjected to uplift pressure due to two cases :-

(i) The static uplift pressure due to subsoil water, the water table in the river bed rises upto bed level at least for some part of the year. There will then be a static uplift pressure on the floor equal to the vertical distance between the drainage bed and the bottom of the floor.

(2) Due to seepage of water from the canal to the drainage. On account of difference of head between the canal and the drainage, seepage

flow will take place from the canal to the drainage. The seepage head will attain a maximum value when the canal is running full and the drainage is dry.

The flow through the pervious medium under the impervious foundations of a syphon structure is governed by three dimensional Laplace's equation.

$$\frac{d^2 \phi}{dx^2} + \frac{d^2 \phi}{dy^2} + \frac{d^2 \phi}{dz^2} = 0$$

where ϕ is the velocity function equivalent to $-$ kh; h the head, k the coefficient of permeability, x, y, and z are the coordinates. However, with the complex boundary conditions of a syphon structure, closed form solution of Laplace's equation is not possible. At present, no method is available to determine the value of exit gradient and uplift pressure below the floor of such a structure. The stability of the structures subjected to forces due to three dimensional seepage has, therefore, to be determined with the help of model studies.

The uplift pressures below the floor and exit gradient at the end of cutoff depends upon the differential head causing seepage, length and width of the structure, location of the point under consideration in relation to the upstream and downstream ends of seepage path, depth of upstream and downstream cutoff, spring level in relation to upstream and downstream water level, length of canl and river wings, presence of any other source of seepage, shape of the structure, homogeneity or otherwise of the subsoil, whether subsoil is anisotropic or isotropic and depth of impervious boundary.

In order to test the stability of a syphon, the uplift pressures below the foundation of barrels, exit gradient at the end of barrel cutoff in the drain bed and exit gradient at the end of cutoff in the canal bed have to be evaluated. Maximum uplift pressure occurs along the end barrels and the values decrease towards the central barrel, the variation depending on the width of the structure. For design of barrel floor, the uplift pressur are, therefore, required to be determined below the outer or end barrel at the mid-point and at the ends inside the barrel cutoffs.

The value of exit gradient at the end of barrel cutoff in the canal bed would be maximum when the drain is at high flood level and the canal is drained at the bed level.

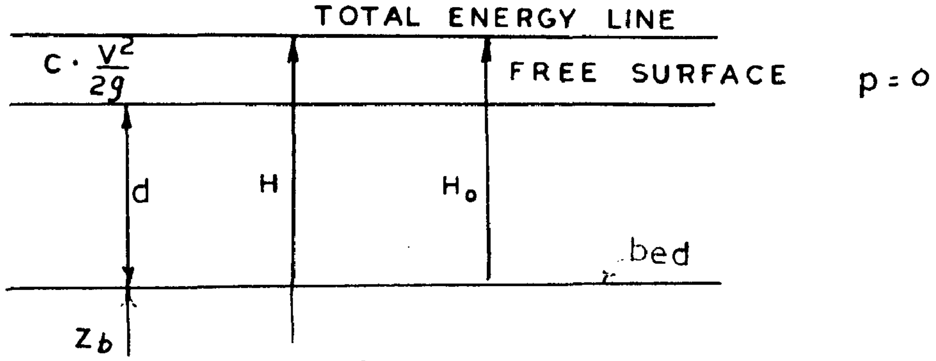


FIGURE 2·1

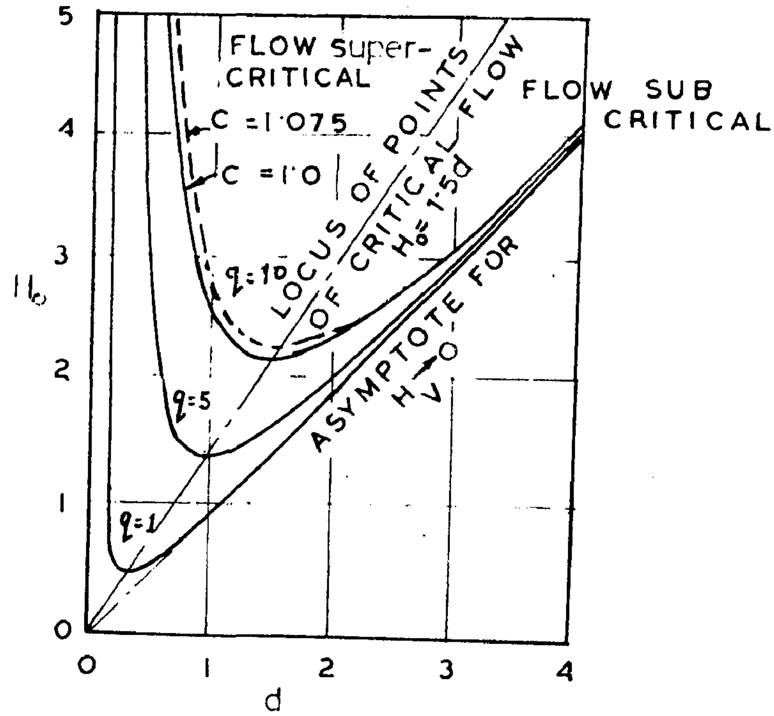


FIGURE 2·2

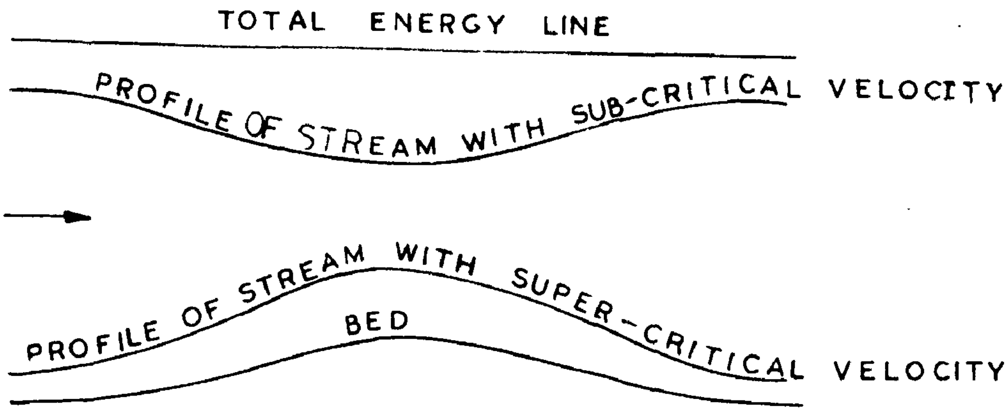


FIGURE 2·3

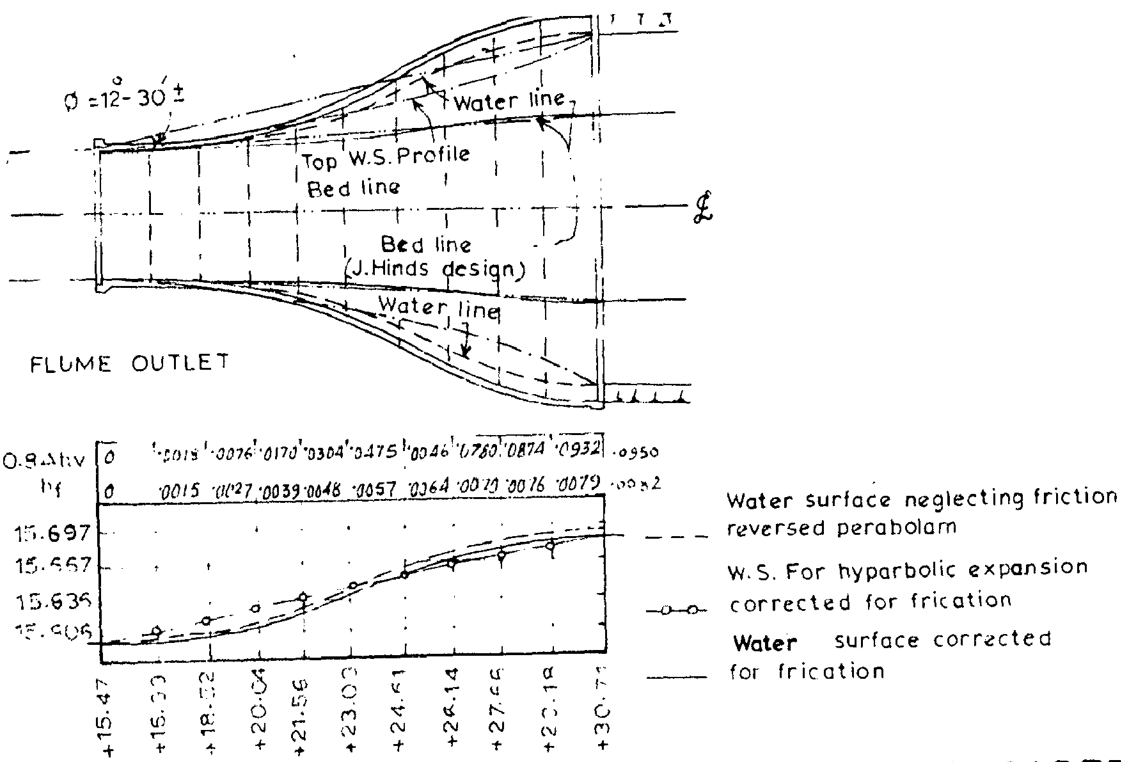


FIG.2.4_HINDS DESIGN AND HYPERBOLIC EXPANSION COMPARED

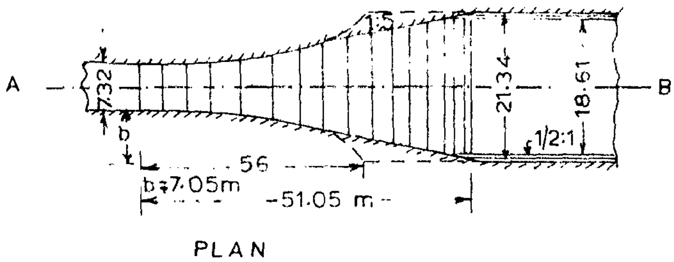


FIG.2.5_EXIT TRANSITION BUILT UP OF ELLIPTICAL ARC AND TANGENT

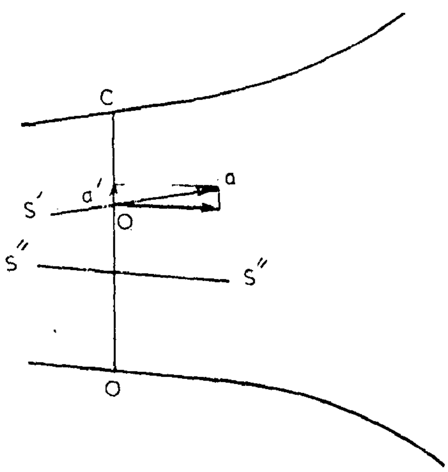


FIG. 2.6_LATERAL ACCELERATION IN DIVERGING FLOW

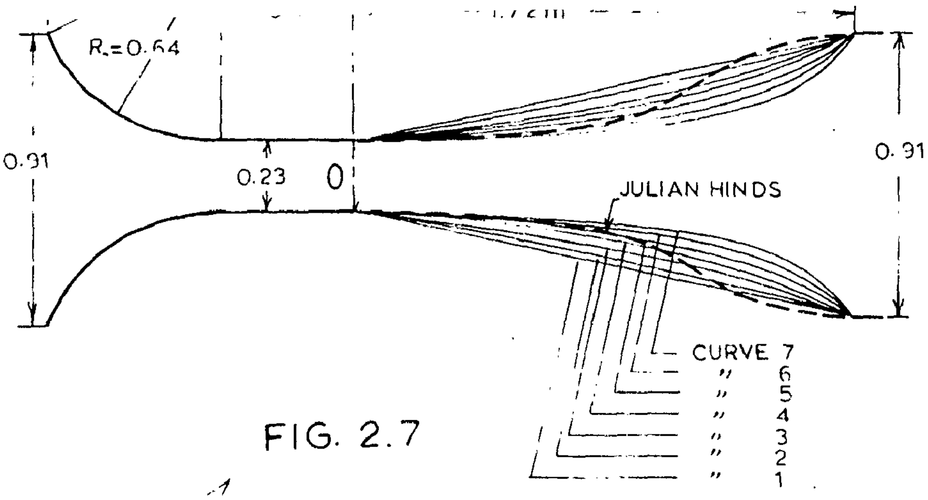


FIG. 2.7

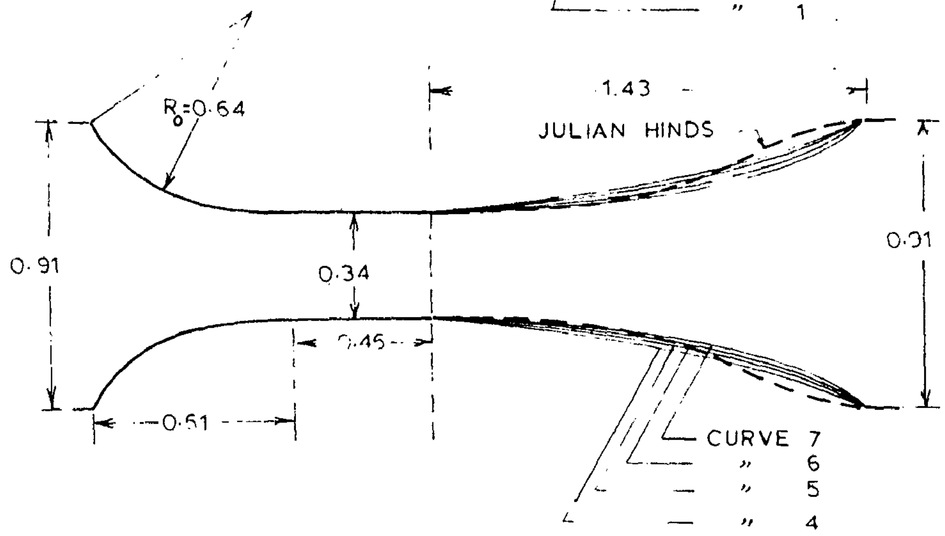
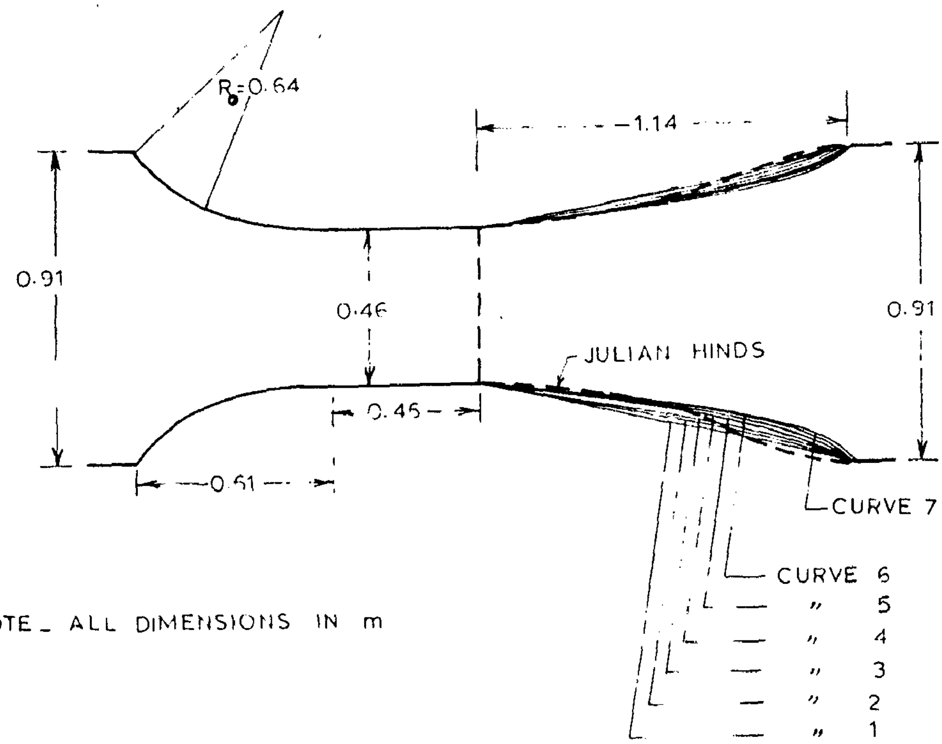


FIG. 2.7a



NOTE_ ALL DIMENSIONS IN m

FIG. 2.7 b

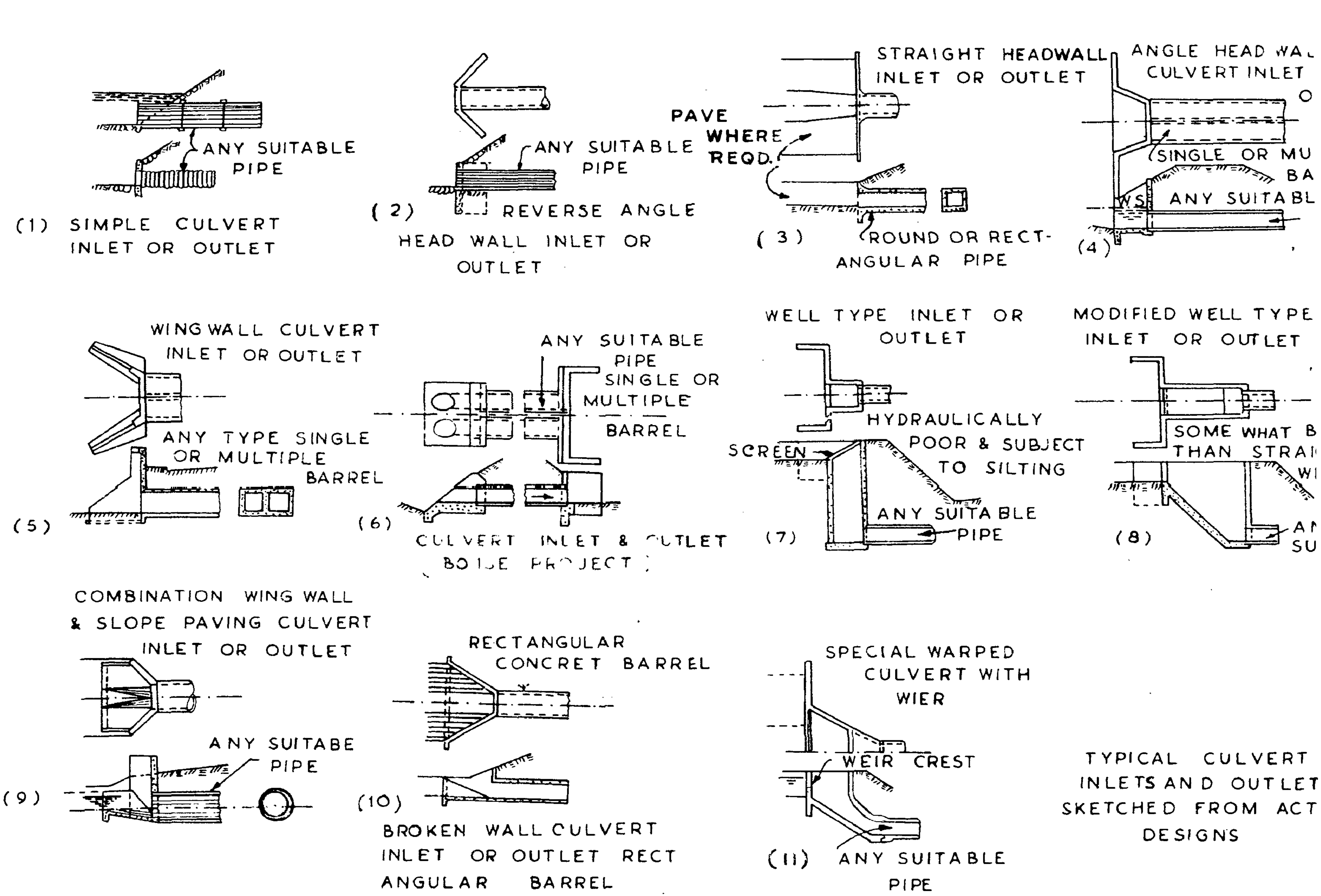

ANY SUITABLE PIPE
(1) SIMPLE CULVERT INLET OR OUTLET
ANY SUITABLE PIPE
(2) REVERSE ANGLE HEAD WALL INLET OR OUTLET
PAVE WHERE REQD.
STRAIGHT HEADWALL INLET OR OUTLET
ROUND OR RECT-ANGULAR PIPE
(3)
ANGLE HEAD WAL CULVERT INLET
SINGLE OR MU
BA
ANY SUITABL
WS
(4)
WINGWALL CULVERT INLET OR OUTLET
ANY TYPE SINGLE OR MULTIPLE BARREL
(5)
ANY SUITABLE PIPE SINGLE OR MULTIPLE BARREL
(6)
WELL TYPE INLET OR OUTLET
HYDRAULICALLY POOR & SUBJECT TO SILTING
SCREEN
ANY SUITABLE PIPE
(7)
MODIFIED WELL TYPE INLET OR OUTLET
SOME WHAT B THAN STRAI
(8)
COMBINATION WING WALL & SLOPE PAVING CULVERT INLET OR OUTLET
ANY SUITABE PIPE
(9)
RECTANGULAR CONCRET BARREL
(10)
BROKEN WALL CULVERT INLET OR OUTLET RECT ANGULAR BARREL
SPECIAL WARPED CULVERT WITH WIER
WEIR CREST
(11) ANY SUITABLE PIPE
TYPICAL CULVERT INLETS AND OUTLET SKETCHED FROM ACT DESIGNS

FIG. 2·8

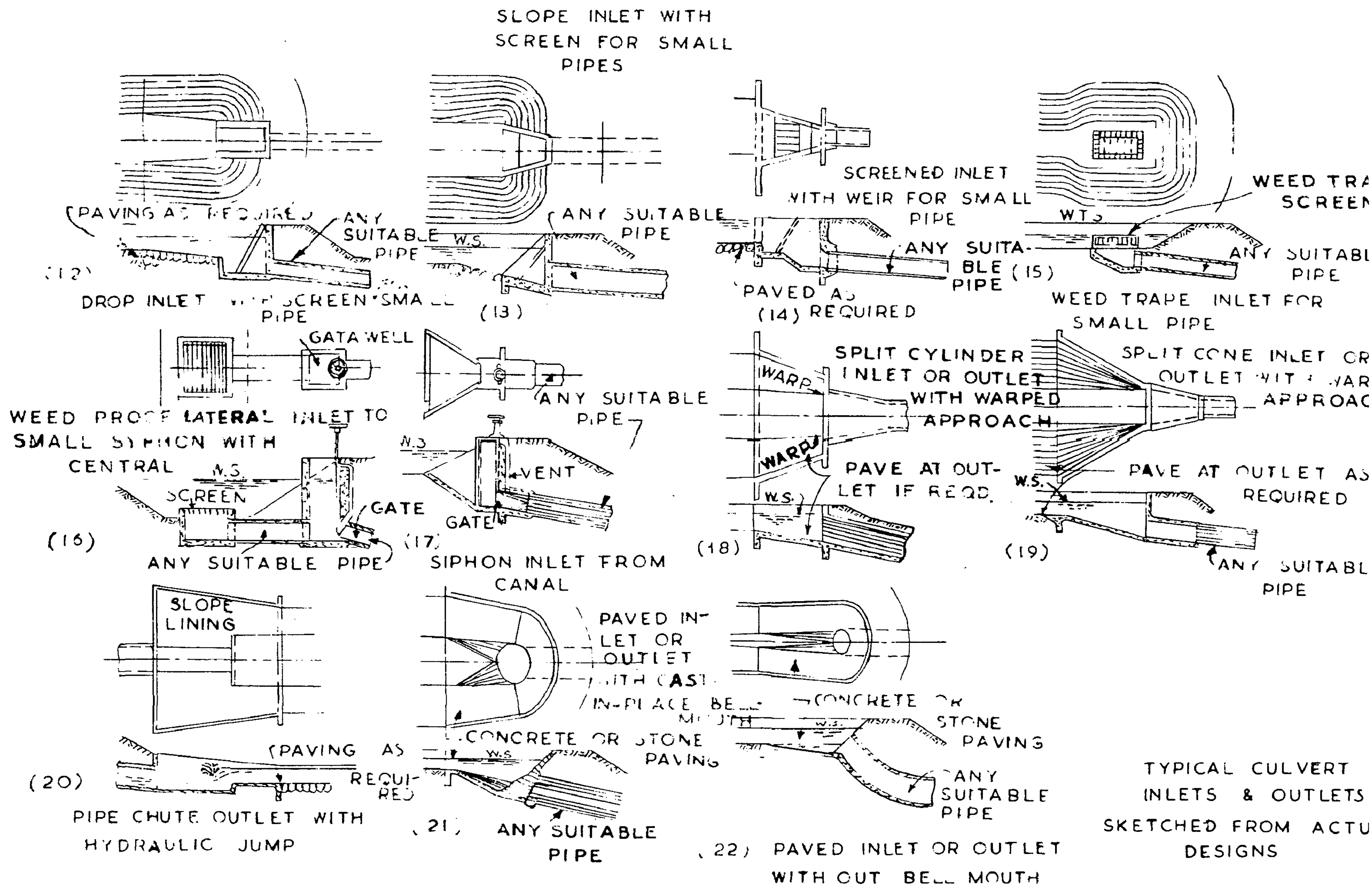

SLOPE INLET WITH SCREEN FOR SMALL PIPES
(12)
DROP INLET WITH SCREEN SMALL PIPE
PAVING AS REQUIRED
ANY SUITABLE PIPE
W.S.
ANY SUITABLE PIPE
(13)
SCREENED INLET WITH WEIR FOR SMALL PIPE
ANY SUITABLE PIPE
PAVED AS REQUIRED
(14)
WT.S
WEED TRA SCREEN
ANY SUITABL PIPE
(15)
WEED TRAPE INLET FOR SMALL PIPE
GATA WELL
WEED PROOF LATERAL INLET TO SMALL SYPHON WITH CENTRAL
W.S.
SCREEN
GATE
ANY SUITABLE PIPE
(16)
ANY SUITABLE PIPE
W.S
VENT
GATE
(17)
SIPHON INLET FROM CANAL
WARP
WARP
SPLIT CYLINDER INLET OR OUTLET WITH WARPED APPROACH
PAVE AT OUT-LET IF REQD.
W.S.
(18)
SPLIT CONE INLET OR OUTLET WITH WARP APPROAC
PAVE AT OUTLET AS REQUIRED
W.S.
ANY SUITABL PIPE
(19)
SLOPE LINING
PAVING AS REQUIRED
(20)
PIPE CHUTE OUTLET WITH HYDRAULIC JUMP
PAVED INLET OR OUTLET WITH CAST IN-PLACE BELL MOUTH
CONCRETE OR STONE PAVING
W.S.
ANY SUITABLE PIPE
(21)
CONCRETE OR STONE PAVING
W.S.
ANY SUITABLE PIPE
(22) PAVED INLET OR OUTLET WITH OUT BELL MOUTH
TYPICAL CULVERT INLETS & OUTLETS SKETCHED FROM ACTU DESIGNS

FIG. 2·9

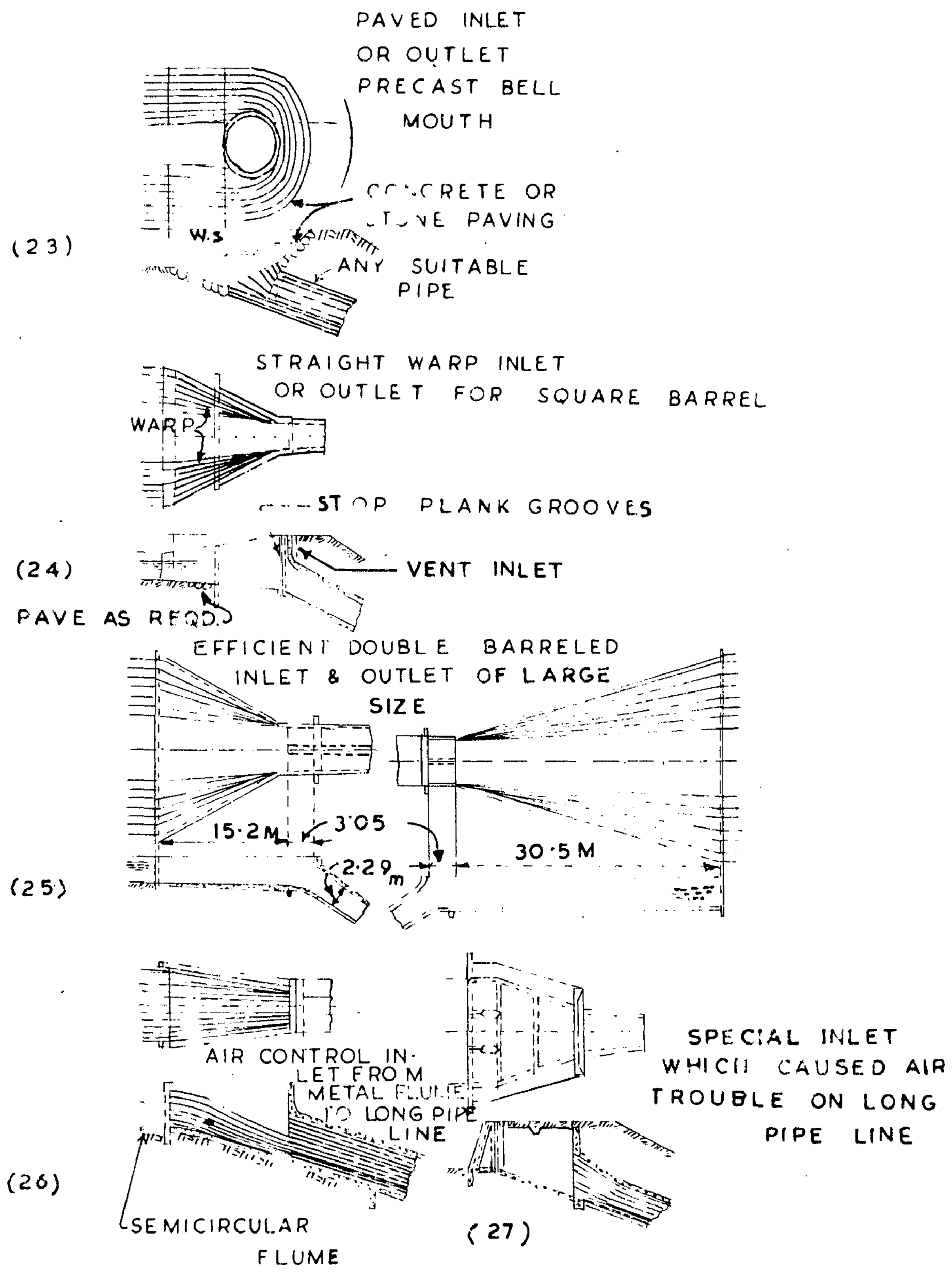


FIG. 2·10 TYPICAL SIPHON INLETS AND OUTLETS
SKETCHES FROM ACTUAL DESIGNS

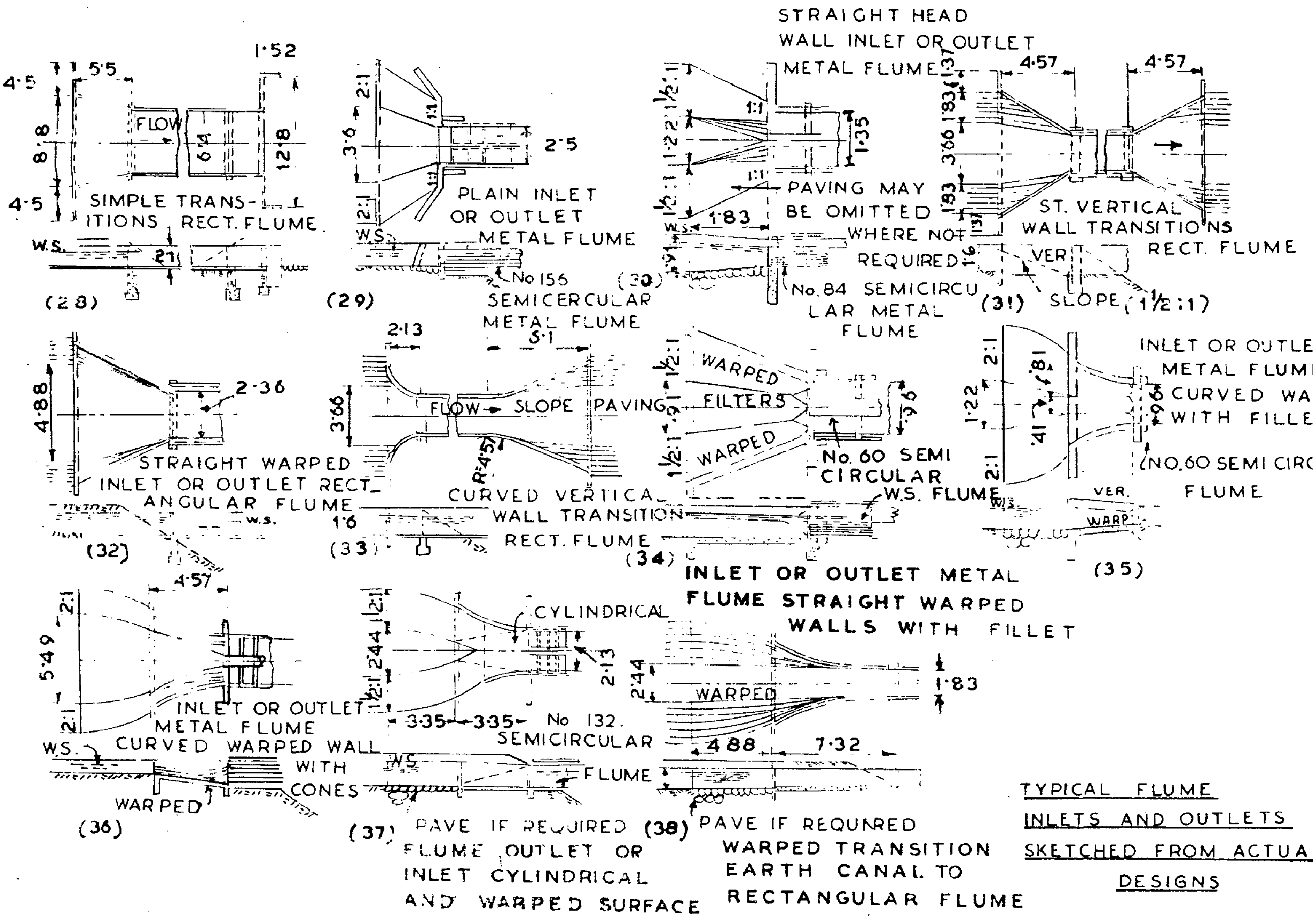
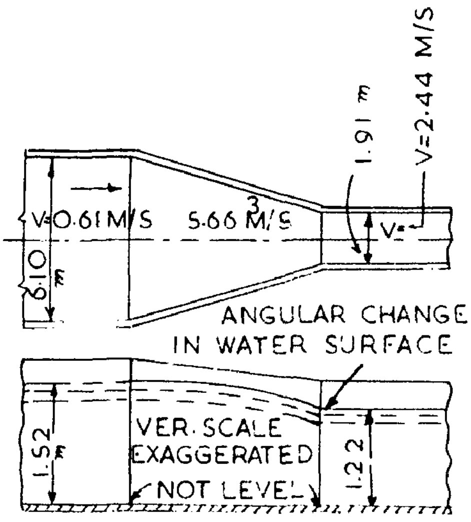


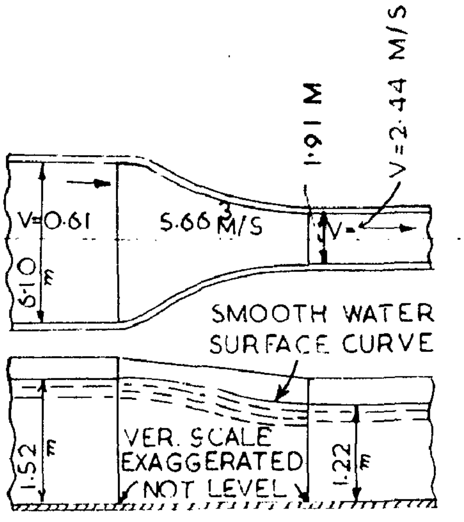
FIG. 2·11

NOTE – ALL DIMENSIONS ARE IN METR[ES]



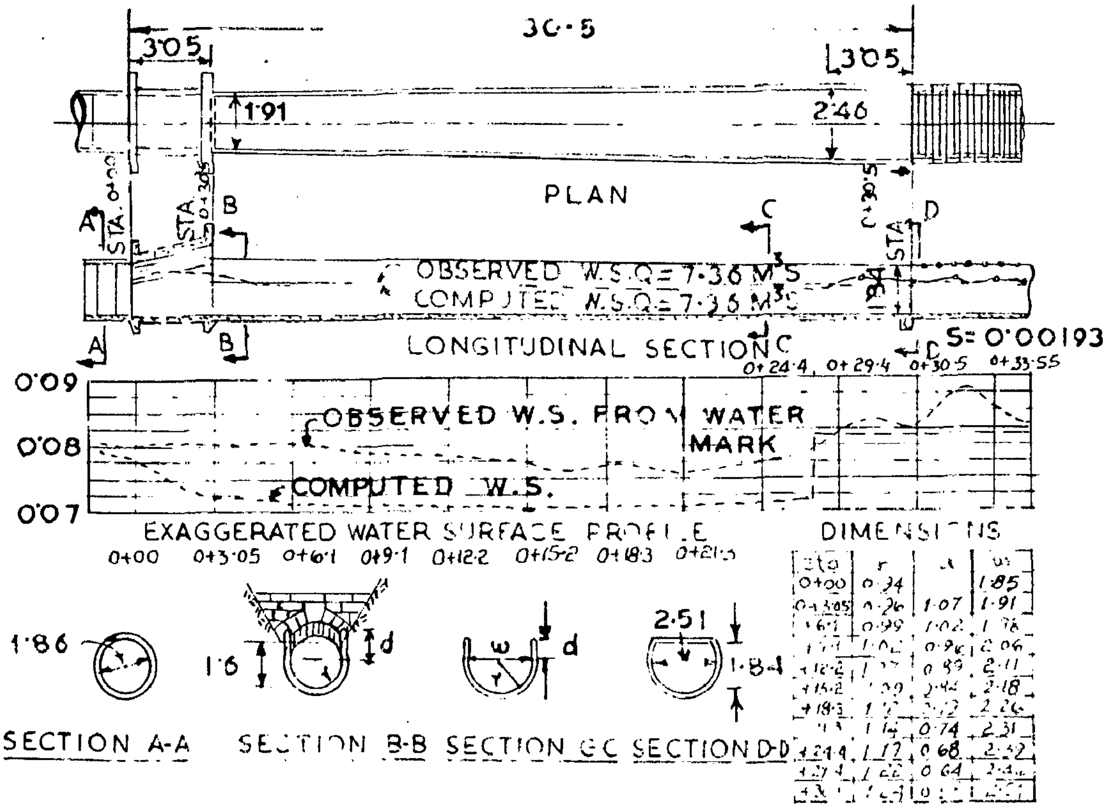
SIMPLE INLET STRAIGHT TAPER

FIG 2·12



SIMPLE CURVED INLET

FIG. 2·13



PLAN AND CROSS SECTIONS OF OUTLET COLUMNAR TUNNEL TIETON CANAL

FIG. 2·14

NOTE ALL DIMENSIONS CONVERTED TO METRE

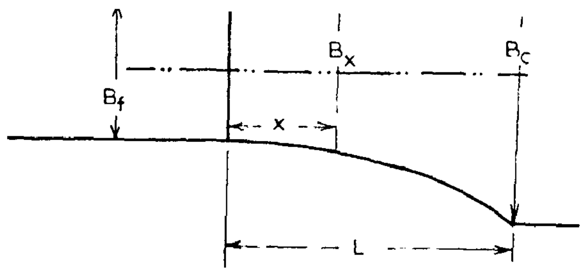


FIG. 2.15 – MITRA'S HYPERBOLIC EXPANSION

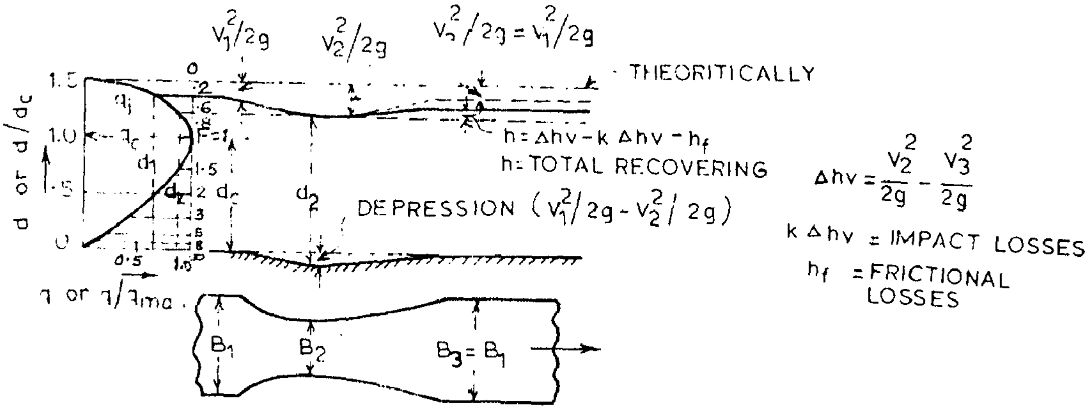


FIG. 2.16 – RECOVERY IN EXPANSIVE FLOW

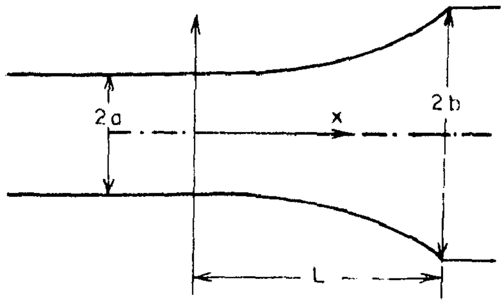


FIG. 2.17

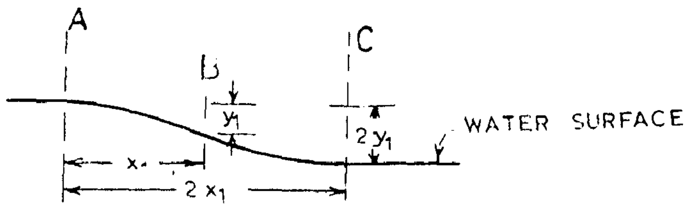


FIG. 2.18 – CONTRACTION TRANSITION WATER SURFACE CURVE

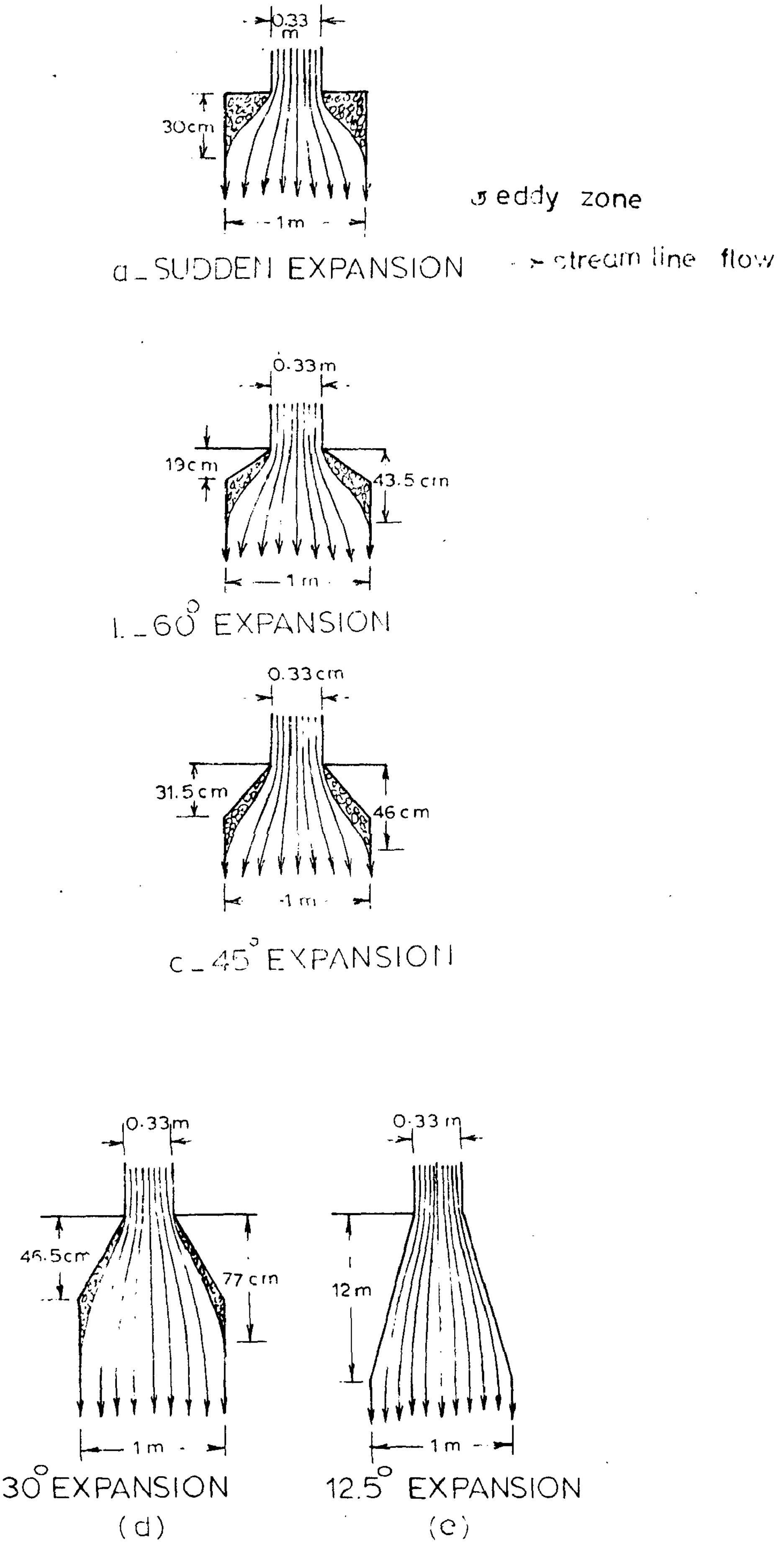


FIG. 2.19 _ FLOW CONDITIONS AT VARIOUS EXPANSIONS

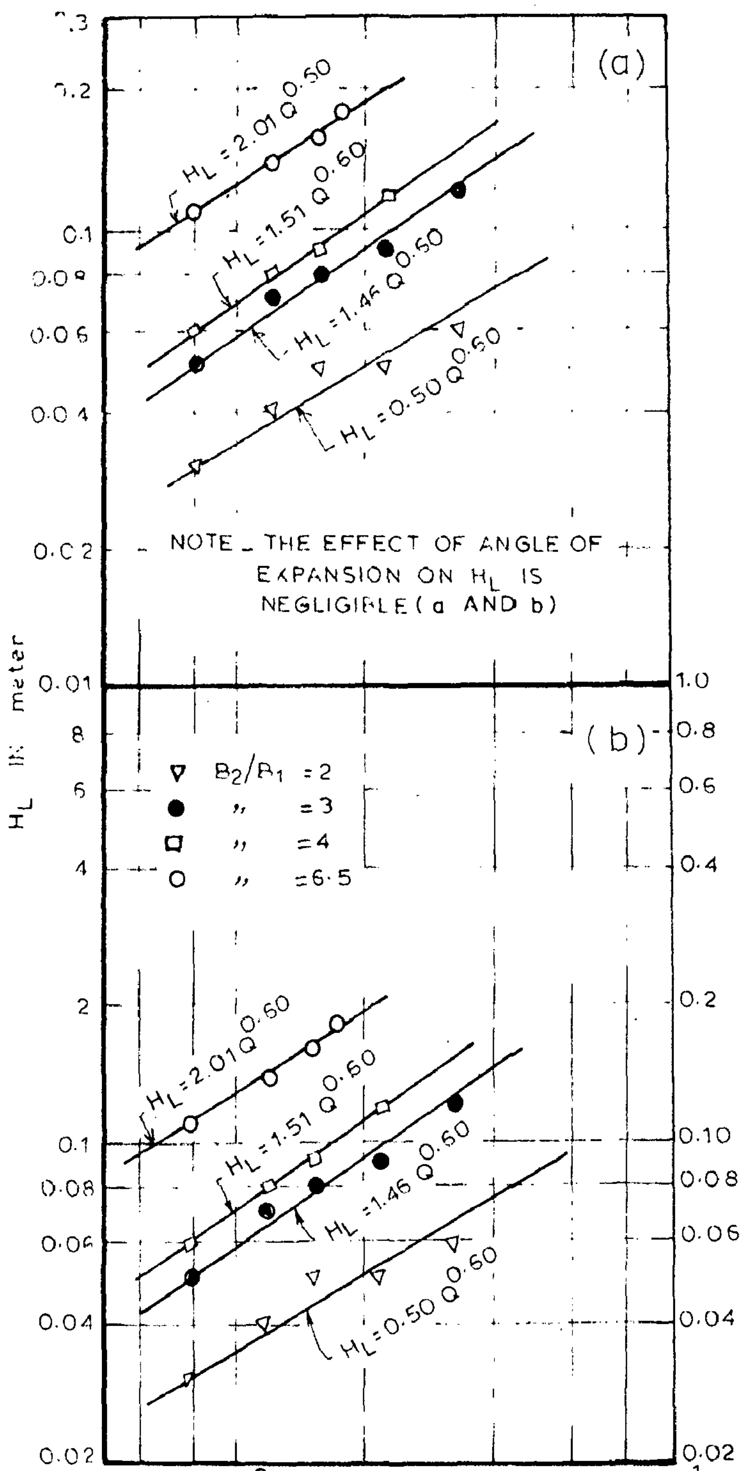


FIG. 2.20 _ VARIATION IN ENERGY LOSS WITH DISCHARGE FOR DIFFERENT EXPANSION RATIOS (METRIC SYSTEM)

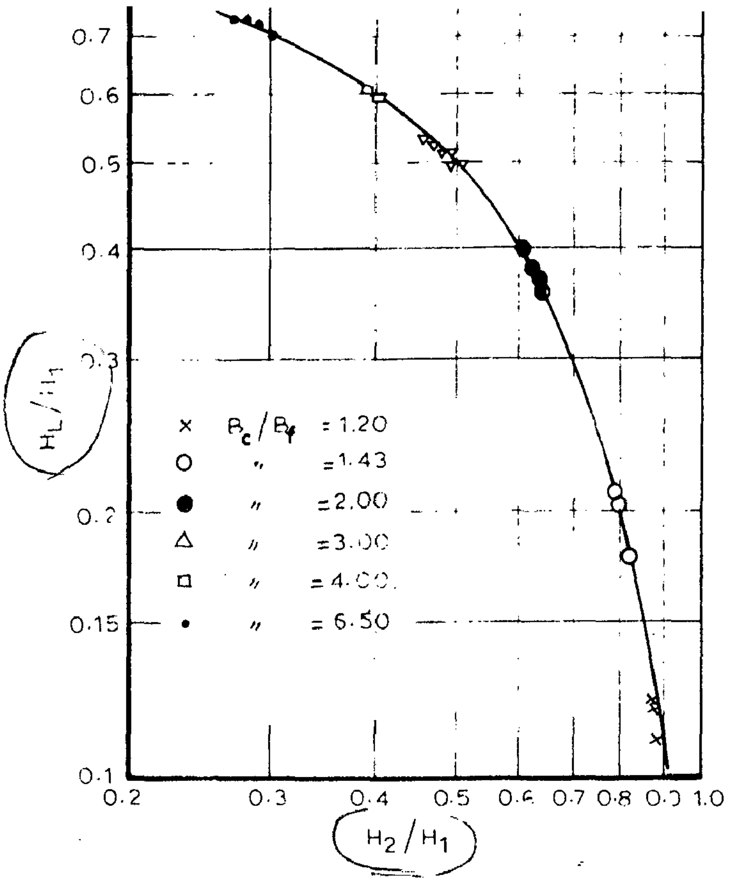


FIG. 2.21 _ VARIATION IN ENERGY LOSS WITH VARIATION IN TOTAL ENERGY DOWNSTREAM OF TRANSITION

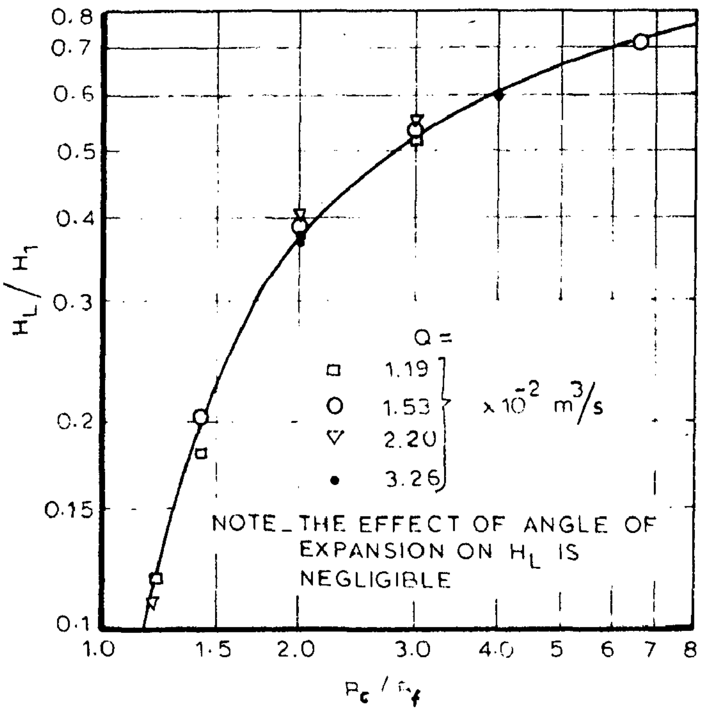
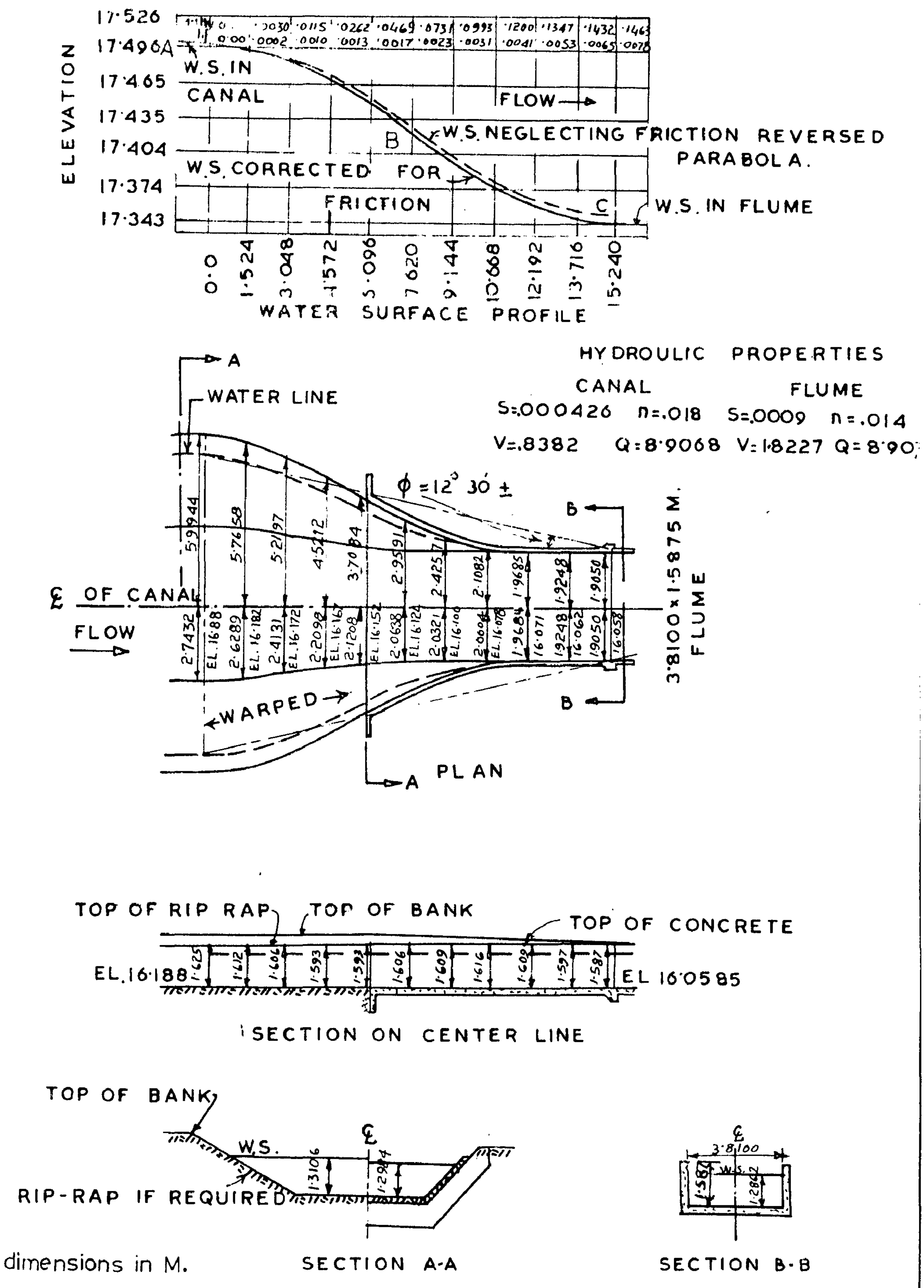


FIG. 2.22 _ VARIATION IN ENERGY LOSS WITH EXPANSION RATIO



dimensions in M.

FIG. 2·23 TYPICAL DESIGN OF AN INLET TRANSITION

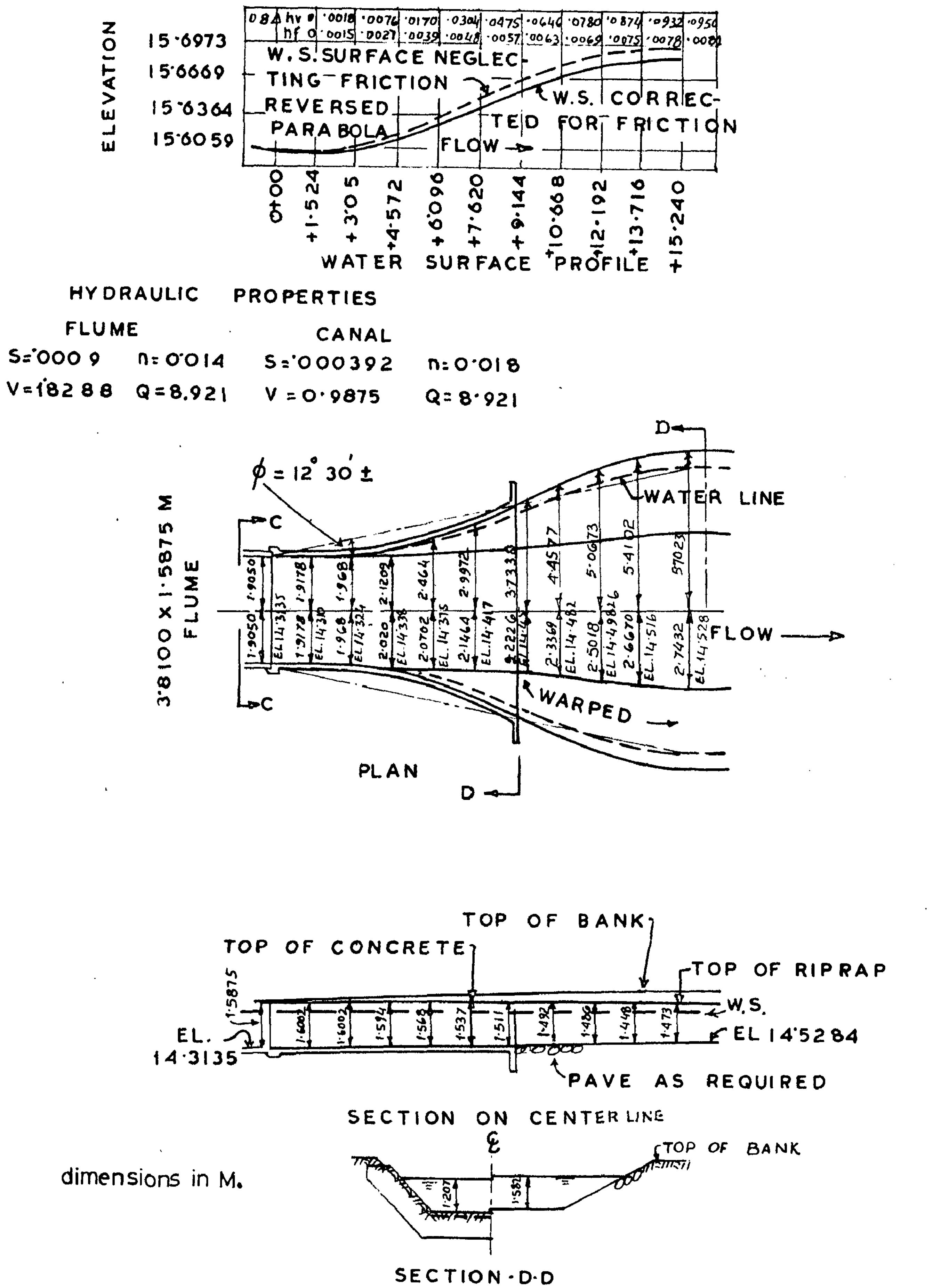
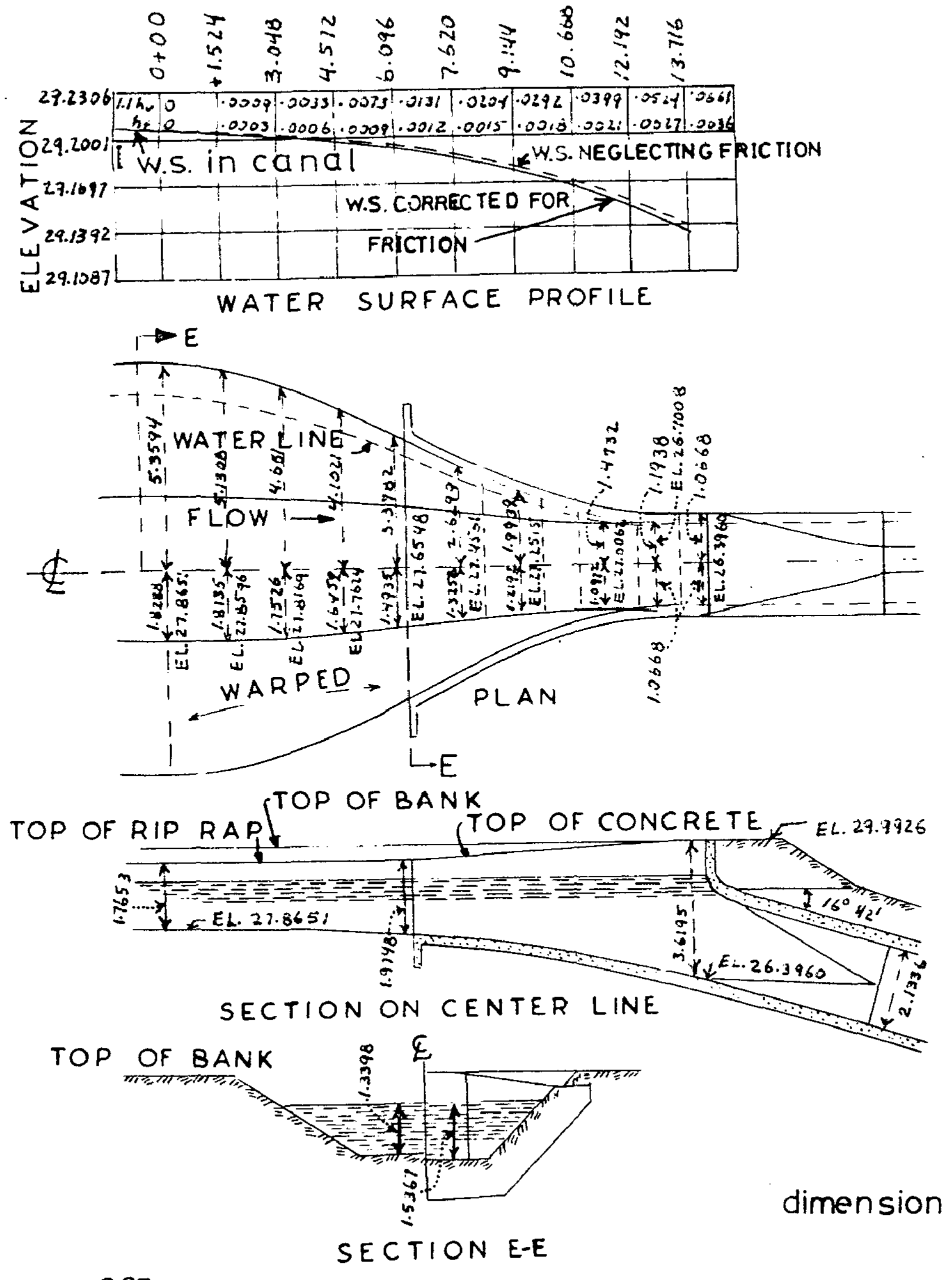


FIG. 2·24 TYPICAL DESIGN OF AN OUTLET TRANSITION



dimensions in M.

FIG.2.25 TYPICAL CONCRETE SYPHON INLET

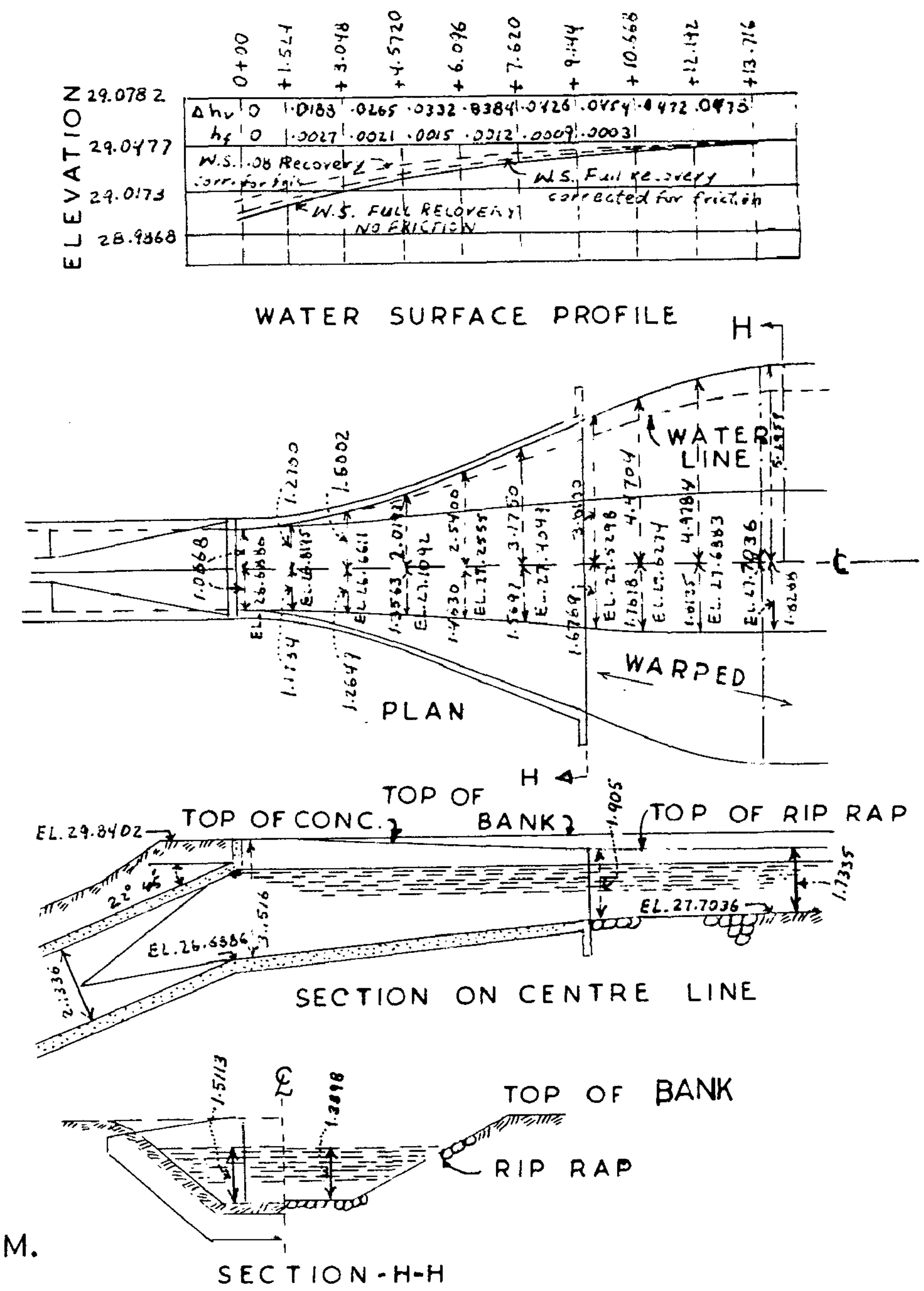


FIG.2.26 TYPICAL CONCRETE SIPHON OUTLET

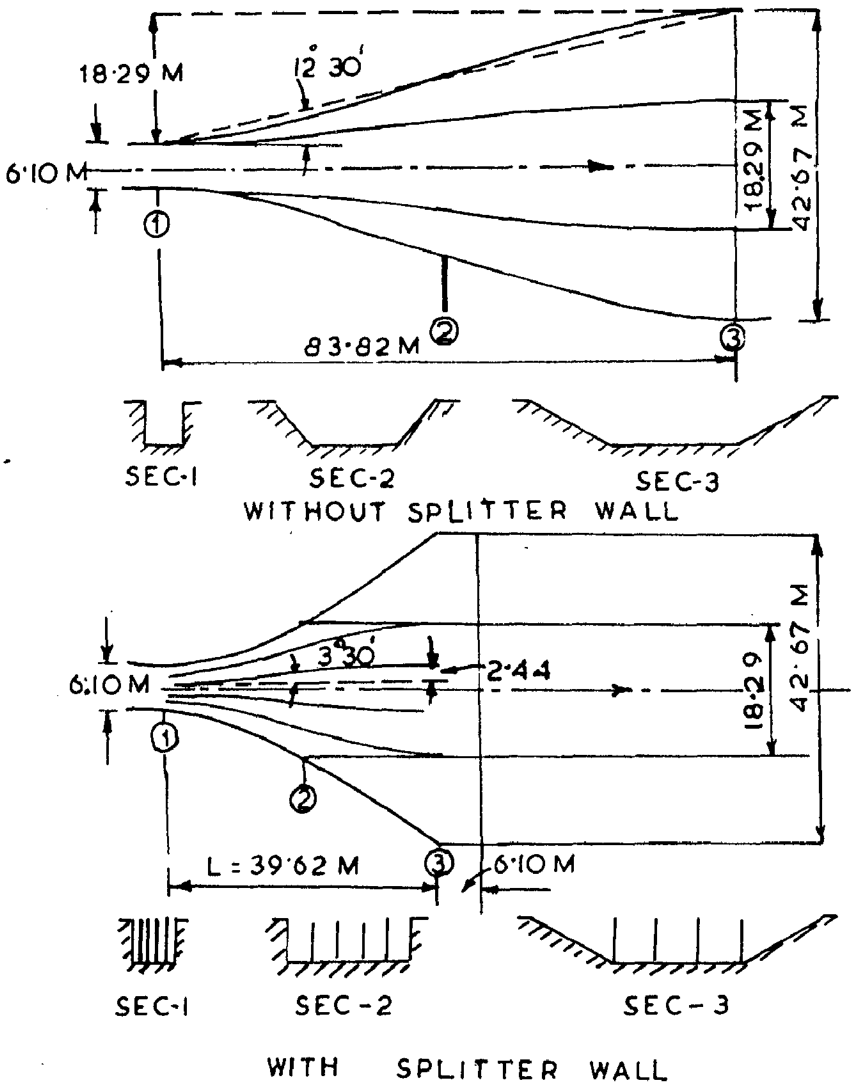
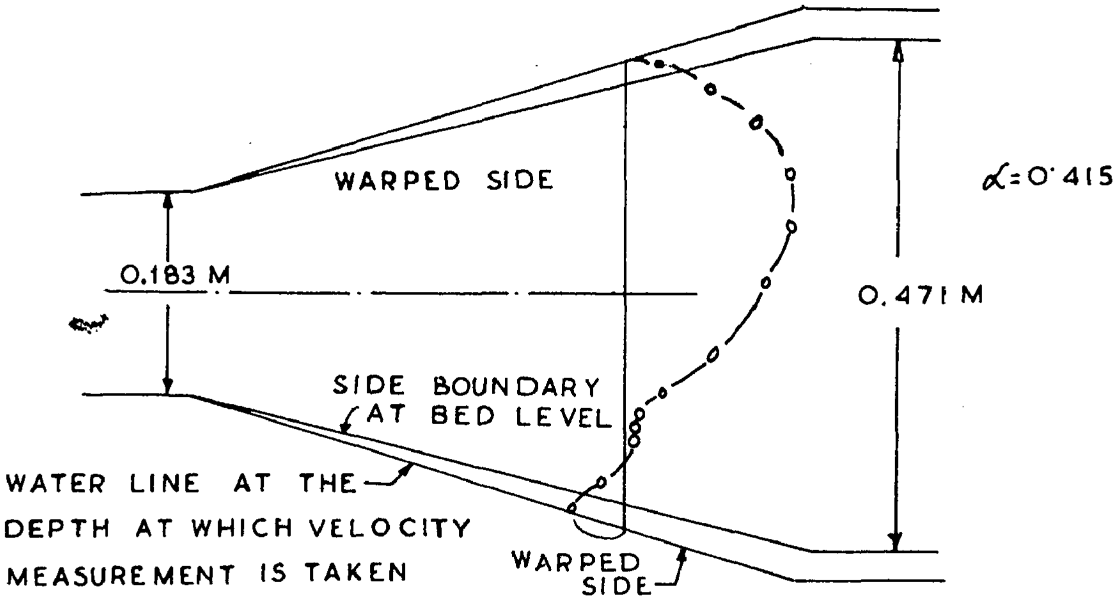
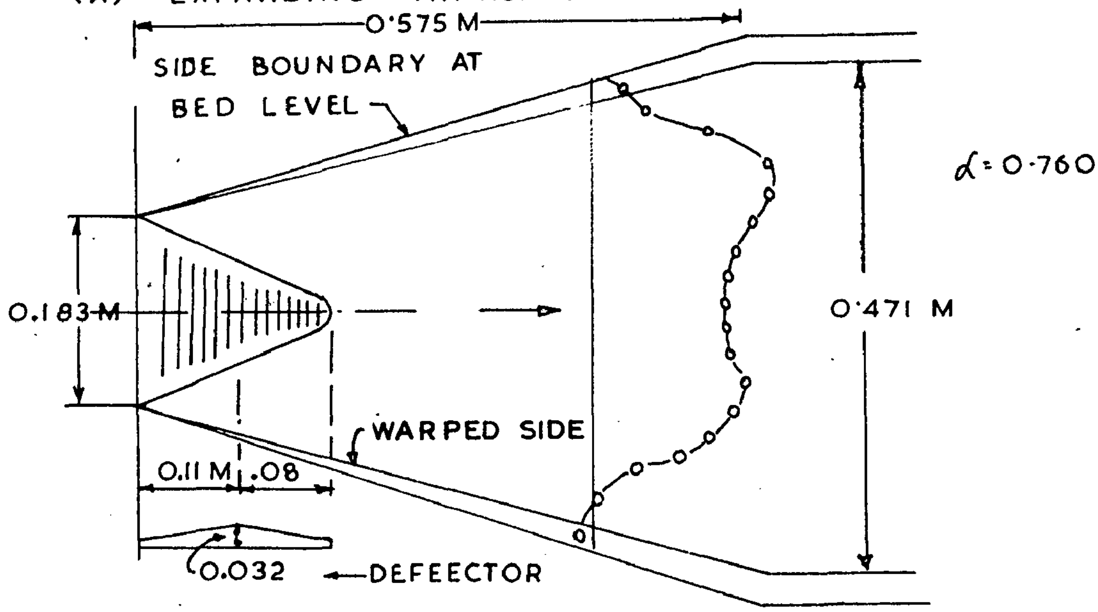


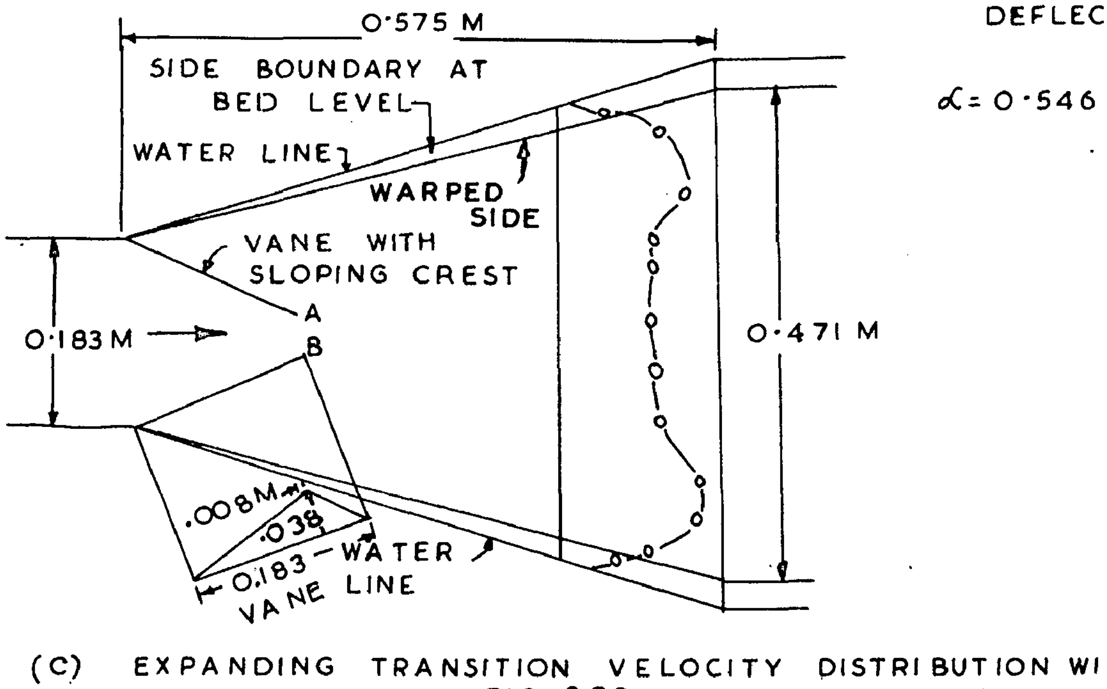
FIG. 2·27–USE OF SPLITTER WALLS



(A) EXPANDING TRANSITION VELOCITY DISTRIBUTION

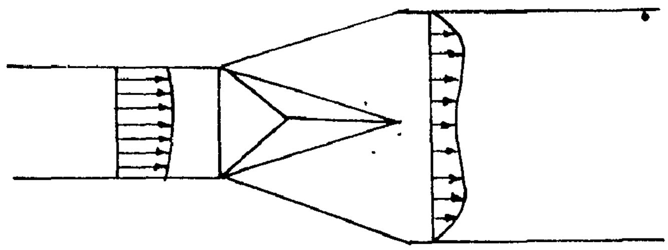


(B) EXPANDING TRANSITION VELOCITY DISTRIBUTION WITH BED DEFLECTOR.

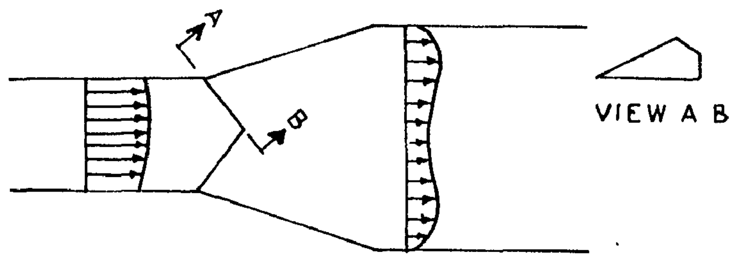


(C) EXPANDING TRANSITION VELOCITY DISTRIBUTION WITH VANE

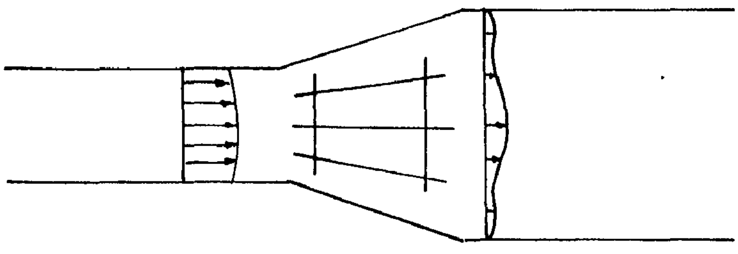
FIG. 2·28



(A) FLOW PATTERN WITH TRIHEDRAL SILL IN CHANNEL EXPANSION



(B) FLOW PATTERN WITH DEFLECTING PLATES IN EXPANSION



(C) FLOW PATTERN WITH SPLITTER PLATES IN CHANNEL EXPANSION

SEPARATION CONTROL DEVICES

FIG. 2·29

NOTE
WHEN L<b, USE BAFFLE
ARRANGEMENT SHOWN
FOR ABRUPT EXPANSION

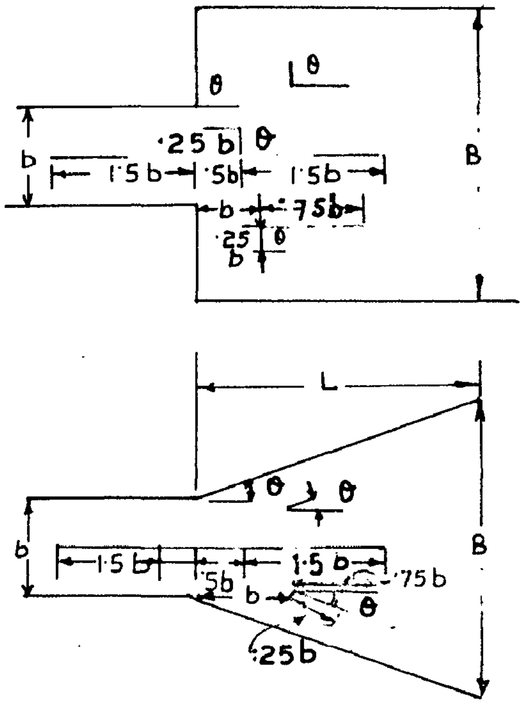
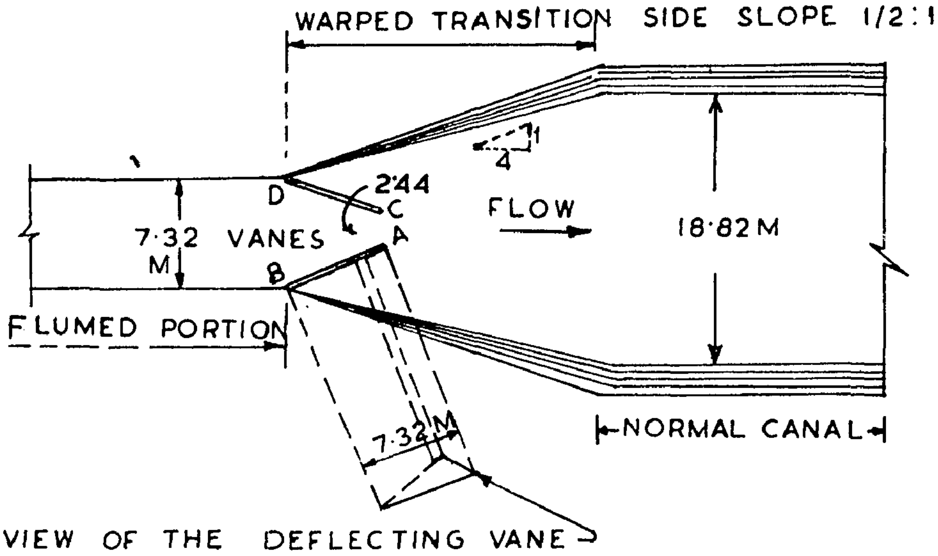
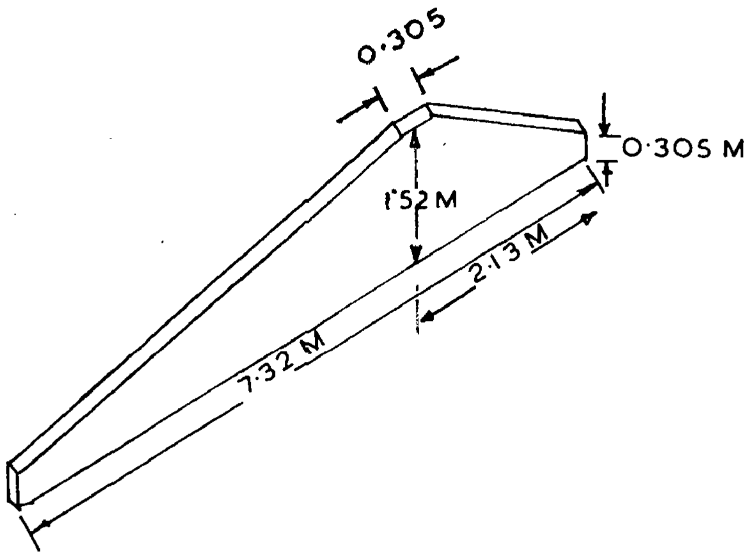


FIG. 2·30 DEFINITION SKETCH FOR OPEN CHANNEL EXPANSIONS WITH TRI-ANGULAR SHAPED BAFFLES.

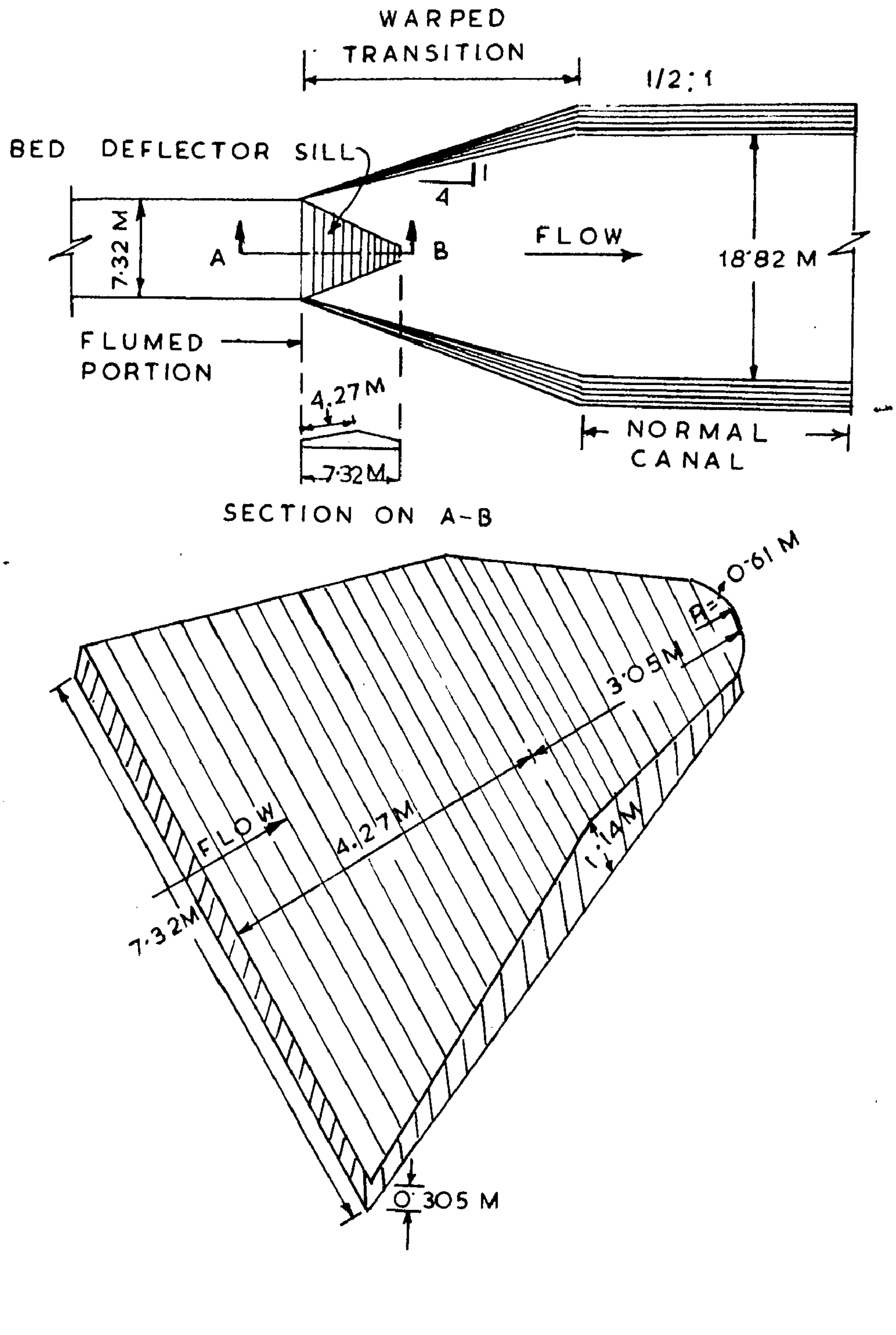


PLAN



SECTION B-A

FIG.2·31 DETAILS OF THE DEFLECTING VANE



DETAILS OF BED DEFLECTOR

FIG. 2·32

MINIMUM CLEARANCES OF REINFORCEMENT FOR WALLS & FLOORS

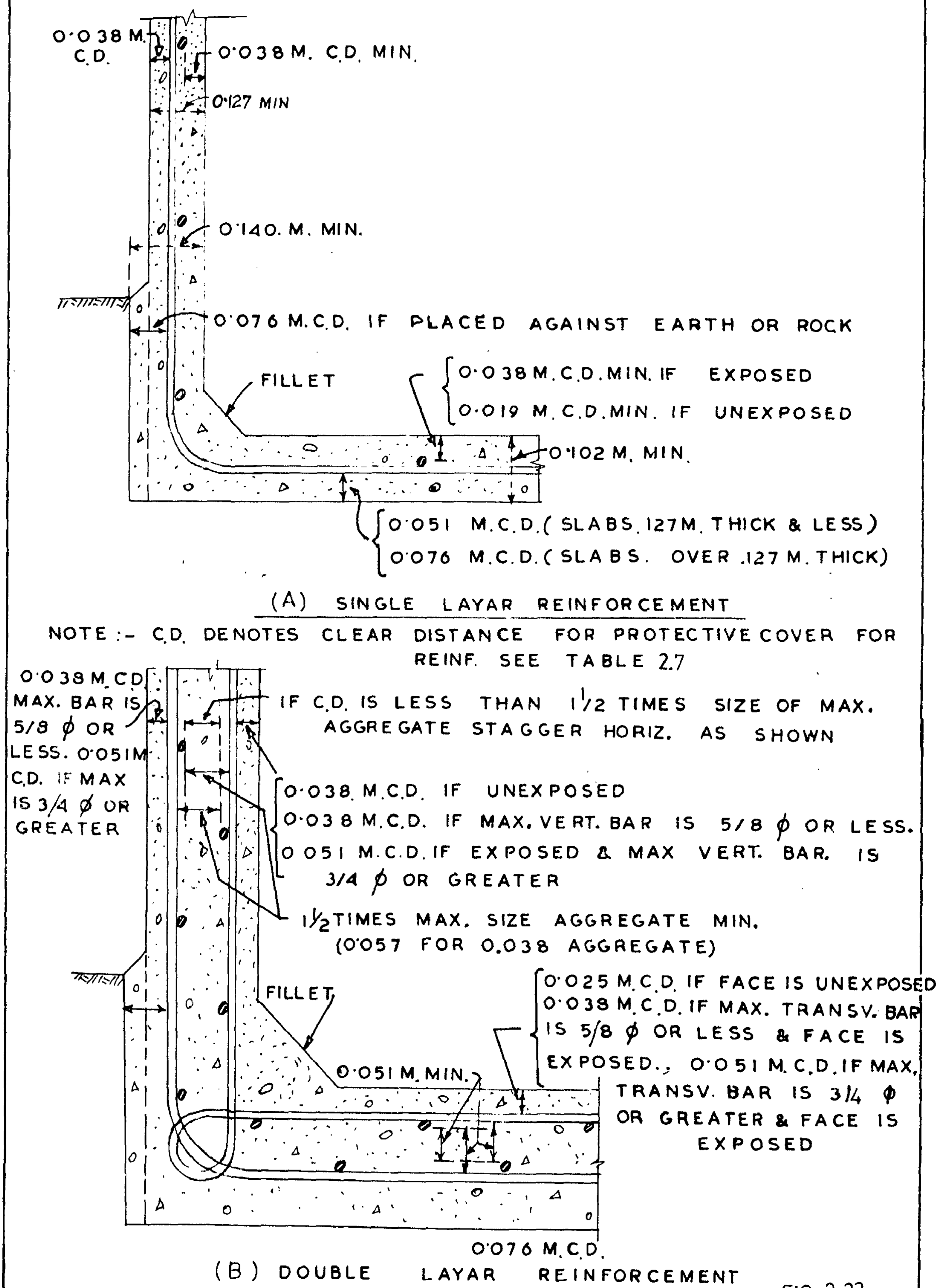


FIG. 2.33

CHAPTER-III

DESIGN OF CHANNEL TRANSITIONS IN SUPERCRITICAL FLOWS

In recent years the hydraulic designer has been increasingly confronted with problems of high-velocity flow in steep flood channels and spillway chutes. The specific character of such flow results from the fact that the velocities exceed considerably the critical velocity and therefore the velocity at which surface disturbances and waves are transmitted in free surface flow. It was found that similar problems of design are encountered in the field of high velocity gas dynamics and that, by analogy, a hydraulic theory could be deduced from the concepts and analytic developments already available in that science.

The design of channel transitions for supercritical velocities on the other hand, must be attacked quite differently because of the occurrence of standing waves. In economically feasible structures, standing waves can not be avoided, and that their characteristics must therefore be explored carefully to insure successful design. Velocities in supercritical flow will vary in magnitude and direction in a systematic fashion in transverse sections and surface elevations will not 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. Design for a minimum of standing waves, therefore, is the particular goal for flow at supercritical velocities so that economical structures may result.

The historical development of research in this field may be summarized briefly. In the United States the first impulse towards work in this field came in the early nineteen thirties when the engineers of the Los Angeles County Flood Control District found the conventional methods of designing flood channels not applicable to the steep gradients employed in their area. Extensive tests were performed by R.T. Knapp, M. ASCE, from 1935 to 1938, to study the flow at supercritical velocities through curved sections of rectangular channel in laboratory. The tests were performed under a variety of conditions as far as radii, slope, and channel forms were concerned. The test results(23,24,25,26) pointed toward characteristics of such flow, which, according to Theoder Von Ká'rman (27), M.ASCE, had their counterpart in the supersonic flow of gases and to which the previous finding of such flow could be adopted.

The analogy of supercritical flow of water to supersonic gas flow had been pointed outalso by D. Riabouchinsky(28) and L.Prandtl (29) without experimental evidence. Ernst Preiswerk (30) worked on the extension of the theory and conducted a number of systematic experiments on a so-called Laval nozzle at Zurich, Switzerland. Although these writers were primarily interested in applications to supersonic flow of gases, work was continued more along hydraulic lines at the California Institute of Technology under Mr. Knapp's direction, at Lehigh University, at M.I.T. under the direction of A.T. Ippen, and at the Iowa Institute of Hydraulic Research in Iowa City under the direction of Hunter Rouse.

Transitions for subcritical flow are designed for minimum energy loss and a smooth water-surface profile by proper streamlining of the sides and bottom; transitions for supercritical flow for minimum wave heights and least disturbances in the flow downstream of the transition. An improperly designed high-velocity transition will exhibit standing waves of excessive height and severe disturbances which not only pose the threat of overtopping of the channel but also lead to appreciable energy loss. The disturbances will persist for a long distance downstream by reflection from the side walls and increase the risk of erosion of the lining of the channel. Proper shaping of the transition is thus a matter of prime importance in the operation of steep channels.

3.1. MECHANICS OF WAVE FORMATION :

According to the gas dynamics analogy, a supercritical flow experiences the effect of a wall deflection only by means of "disturbance lines" originating from the point or zone of wall deflection or curvature. Consider, for example, an abrupt outward deflection of the wall (Fig.3.1(a)). A series of negative disturbance lines, or the Mach lines in gas dynamics, originate at the point of the wall deflection. As the streamlines cross the group of disturbance lines (sometimes referred to as the ' expansion fan'), the water level falls gradually while the Froude number increases correspondingly. The first disturbance line is inclined at the Mach angle $\beta_1 = \sin^{-1}\left(\frac{1}{F_1}\right)$ to the streamline, while the last disturbance line is inclined at the Mach angle $\beta_2 = \sin^{-1}\left(\frac{1}{F_2}\right)$ to the deflected flow. The disturbance lines may indicate positive or negative disturbances. Henceforth, positive disturbance or positive wave or surge fronts will be defined as those which deflect the flow toward the line of disturbance and cause a rise in the water surface. A wall curved into the flow and displacing fluid would be the source of such positive wave fronts. Negative disturbances or depression fronts are cased by boundaries curving away from the flow, providing larger cross-sections of flow and therefore producing a lowering of the surface and a deflection of the flow away from the wave front. It may be noted here that the intermediate disturbance lines are imaginary lines and may be thought of as discretized water surface contours. In an actual problem, the total flow deflection may be divided into steps of, say, 2^o and the corresponding disturbance lines located. In any zone bounded by discretized disturbance lines, the flow properties such as depth of flow and the Froude number are assumed to be constant. Except for a localized region near point O, the water surface slopes are not large. Hence, there is not wave formation as such & energy loss due to the flow expansion is neglgible.

Consider the inward deflection of the wall by an angle θ (Fig.3.1(b)). As in supersonic gas flow, this gives rise to the shock wave, which manifests itself on the water surface as an oblique standing wave of appreciable height. The shock wave is obviously accompanied by some energy loss. The depth of flow is increased below the shock wave while the Froude number is reduced. These can be evaluated by applying the momentum principle to the oblique standing wave. However, an inward wall deflection of very small value (say, a few degrees), producing a 'weak shock', may be thought of as giving rise to a solitary positive disturbance line that may be analyzed by the same methods as for outward wall deflection.

A simple analysis of the " weak wave " (both negative and positive) may be made on the following assumptions :

1. The disturbance are small;
2. Vertical components of velocity are negligible;
3. Hydrostatic pressure distribution prevails over the depth of flow at every point;
4. Energy dissipation due to friction and wave formation is negligible; and
5. Bottom shear is assumed to be balanced by component of the gravity force.

Consider a small inward deflection $d\theta$ of a supercritical flow by means of a weak wave having an obliquity β_1 with respect to the initial flow direction (Fig.3.2(a)). Let the velocities of flow before and after crossing the wave be V_1 and V_2. Resolving the velocities into components normal and tangential to the wave, ($V_{t1} = V_{t2}$) since the wave can affect only the momentum normal to it. The continuity equation is

$$d_1 V_{n1} = d_2 V_{n2} \tag{3.1}$$

where d_1 and d_2 are the depths before and after crossing of the wave. The momentum equation is

$$\frac{\gamma (d_1)^2}{2} + \frac{\gamma}{g} d_1 (V_{n1})^2 = \frac{\gamma (d_2)^2}{2} + \frac{\gamma}{g} d_2 (V_{n2})^2 \tag{3.2}$$

It is clear that the net pressure force acting to decelerate the flow can affect only the momentum of the stream normal to the wave front; and since no force component exists parallel to the front, the tangential components V_{t1} and V_{t2} of the velocity must remain unaltered as the flow passes under the wave front, or $V_{t1} = V_{t2}$.

From eqs. 3.1 and 3.2, the expression for normal component V_{n1} is obtained in terms of the depths d_1 and d_2.

$$V_{n1} = \sqrt{g\,d_1}\sqrt{\frac{d_2}{d_1}\ \frac{1}{2}\ \left(1 + \frac{d_2}{d_1}\right)} \tag{3.3}$$

In applying the momentum equation, V_{n1} has been automatically defined as the wave velocity, since the wave front was assumed stationary. In turn any wave of a certain height, $d_2 - d_1$, will assume in supercritical flow such a position that the normal component of V_1 with respect to the front will equal V_{n1} as defined by Eq.3.3. The wave front will not be stationary otherwise. It may also be stated that in agreement with the equation there must always be a stationary position for any wave in supercritical flow, until V_{n1} as defined by Eq.3.3. exceeds the value of V_1. If $V_{n1} = V_1$, the wave front assumes a position at a right angle to the flow, and becomes the familiar hydraulic jump. For $V_{n1} > V_1$ the jump will start moving upstream, detaching itself from the source of the disturbance, which then remains surrounded by subcritical flow.

In the limiting case of a weak wave, $d_2 = d_1$ and V_{n1} from Eq.3.3 becomes $V_{n1} = \sqrt{2gd_1}$. Hence,

$$\beta_1 = \mathrm{Sin}^{-1}\ \frac{\sqrt{gd_1}}{V_1} = \mathrm{Sin}^{-1}\left(\frac{1}{F_1}\right) \tag{3.4}$$

It follows that any continuous disturbance or weak wave in a supercritical flow orients itself to the flow such that the component of the flow velocity normal to the wave equals the critical velocity $\sqrt{gd_1}$.
Here there is a striking analogy with supersonic gas flow in that any Mach line in a supersonic gas flow also orients itself to the flow such that the component of the flow velocity normal to the waves equals the speed of sound.

The change in depth of flow and Froude number caused by the deflection of the flow can now be computed. For a weak expansion wave (Fig.3.2(b)), the law of sines applied to triangle ABC gives :

$$\frac{dV_n}{V} = \frac{\text{Sin } d\theta}{\text{Sin}(90^0-\beta_1+d\theta)}$$ and, for infinitesimal

changes of θ

$$d\,V_n = \frac{Vd\theta}{\text{Cos }\beta} \quad (3.5)$$

in which the subscripts may now be omitted. Rewriting the momentum equation for infinitesimal changes in depth and velocity, a second deferential expression for d V_n is obtained: $\gamma dd = \frac{\gamma}{g}\ d\,V_n\ d\,V_n$; or

$$d\,V_n = \frac{dd}{V_n}\ g \quad (3.6)$$

Since V_n may be replaced by V sin β , Eqs.3.5 and 3.6 may be combined into $\frac{Vd\theta}{\text{Cos}\,\beta} = \frac{gdd}{V\text{ Sin }\beta}$; or

$$dd = V^2/g\ \tan\beta\ d\theta \quad (3.7)$$

Equation 3.7 may be integrated to obtain the gradual change of depth with gradual angular deflections of the stream if the basic assumption made is next introduced to relate β and V to d namely, that energy dissipation may be disregarded for such flow in accordance with the Bernoulli theorem. Therefore: $H = d + \frac{V^2}{2g}$ = constants, and thus, $V = \sqrt{2g\,(H-d)}$. Since $\tan\beta = V_n/V_t = \text{Sin }\beta \Big/ \sqrt{1-\text{Sin}^2\beta} = \sqrt{gd}\Big/\sqrt{V^2-gd}$

$$= \sqrt{d}\Big/\sqrt{2H-3d}$$

∴ Eq. 3.7 can be transferred finally into

$$dd/d\theta = \frac{2g\ (H-d)\ d^{1/2}}{g\ (2H - 3d)^{1/2}} = - \frac{(2d/H)^{1/2}\ (1-(d/H))\ H}{(1 - 3/2\ d/H)^{1/2}} \qquad (3.8)$$

Integrating eq. 3.8

$$\theta = \sqrt{3}\ \tan^{-1} \left[\frac{1-3/2\ d/H}{3/2\ d/H} \right]^{1/2} - \tan^{-1} \left[\frac{3(1-3/2\ d/H)}{3/2\ d/H} \right]^{1/2} - \theta_1 \qquad (3.9a)$$

in which θ_1 constitutes the constant of integration defined by the condition that for $\theta = 0$ the depth d is the initial depth d_1 or $(d_1 = d_2)$.

Eq. 3.9a may also be written in an alternate form employing the Froude number to express $d/2\ H/3 = 3/(2+F^2)$ or $d/H = 2/(2+F^2)$. Substitution of this equivalent results in

$$\theta = \sqrt{3}\ \tan^{-1} (\ (F^2-1)/3\ )^{1/2} - \tan^{-1} (F^2 - 1\ )^{1/2} - \theta_1 \qquad (3.9b)$$

3.2 METHOD OF CHARACTERISTICS

To facilitate the analysis of wave systems due to continuous disturbances, such as those in curved channels or in channel contractions or expansions, a graphical method has been devised on the basis of the preceding theory with certain ingenious changes. This development was entirely the outcome of an early analysis of supersonic flow of gases by A. Busemann[31] and is known as the "Method of Characteristics". At the suggestion of Mr. Von Kármán this method was adopted in 1935 for use in the supercritical flow of water and was applied to the problem of flow in the curved sections of open channels during the analysis of the test data. Independently, Mr. Preiswerk[30] studied water flow through channel expansions and other problems related by analogy to the high - velocity flow of gases.

The basic equation for the graphical method of solving problems of supercritical flow of water is given by Eq. 3.8. For the purpose of obtaining the deflections of the streamlines as a function of the characteristics of the wave fronts, the ratio (d/H) appearing in that equation will be replaced by its equivalent from the Bernoulli theorem : $1 = d/H + V^2/2gH$; or with
$\bar{V} = V / \sqrt{2gH}$

Where $\bar{V}$ is the dimensionless velocity of flow

The above equation may be written as

$$\bar{V}^2 = V/2gH$$

By putting the value of $H = (2d + dF^2)/2$ it will give

$$\bar{V}^2 = F^2 / (2 + F^2) \qquad (3.10)$$

Simplifying eq. 3.10 we will get

$$F^2 = 2\,\bar{V}^2 / (1 - \bar{V}^2) \qquad (3.11)$$

Putting the above value in eq. 3.9b

$$\theta = \sqrt{3}\,\tan^{-1}\left((3\,\bar{V}^2 - 1)/3\,(1-\bar{V}^2)\right)^{1/2} - \tan^{-1}\left((3\,\bar{V}^2 - 1)/(1-\bar{V}^2)\right)^{1/2} - \theta_1$$

$$(3.12)$$

If θ_1 is ignored and $\bar{V}$ is plotted against θ, using polar coordinates, the resulting curve, is an epicycloid between the circles of radii $1/\sqrt{3} = 0.577$ and 1, and occupying the vectorial angle $\theta = 65^\circ\ 53'$.

When $F = 1$, from eq.3.10 $\bar{V} = 1/\sqrt{3} = 0.577$, from equation 3.12 $\theta = 0$, When $F =$, from 3.11 $\bar{V} = 1$, and $\theta = 65^\circ\ 53'$.

If the complete family of the left-running and right running epicycloids are drawn for θ varying by a small increment of about 2°, we get the "characteristics diagram" Fig.3.3. At every point in the flow, the characteristics of the left-running and right running families of curves are inclined at an angle of $\sin^{-1} 1/F$ to the flow direction. In physical terms, the characteristics define the directions of propagation of an elementary disturbance on the surface at the point under consideration.

In solving a problem, the initial point in the diagram is chosen on the $\bar{V}x$ axis ($\theta = 0^o$) with the appropriate value of $\bar{V}$. If the boundary deflects through an angle of θ, then starting from the initial point, one of the epicycloidal branches leading inward or outward is followed, according as whether the disturbance is positive or negative, till it intersects the radial line corresponding to θ. The radius vector to this point gives the new value of $\bar{V}$.

The direction of the corresponding disturbance line can be shown to be that of the normal at themid point of the epicycloidal are just traversed. This can be easily located by using an elliptical template, known as the Mach ellipse, having its semi-axis equal to $\bar{V} = 1$ and $\bar{V} = 0.577$, respectively. If the Mach ellipse is centred on the characteristics diagram in such a way that the point on the characteristics diagram falls on it, the major axis of the ellipse will represent the direction of the disturbance lines.

3.3. <u>APPLICATION OF THE METHOD OF WEAK WAVES TO THE DESIGN OF EXPANSIONS:</u>

The application of the ' Method of Weak waves' for the construction of the flow in a transition requires the establishment of rules governing wave interaction, like those in gas dynamics. These rules are :-

(1) A positive or negative wave gets reflected from a straight wall as a positive or negative wave of the same strength (Fig.3.4(a)). Here

the strength of a wave is reckoned by the flow deflection which it is capable of producing. The direction of the reflected wave can be obtained from the characteristics diagram. It may be noted that the incident and reflected waves are not equally inclined to the straight wall.

(2) If the waves intersect, they cross over and proceed with undiminished strength. But the wave front suffer refraction as shown in Fig.3.4(b). The orientations of the waves can be worked out from the characteristics diagram.

While the gas dynamics analogy is no doubt interesting, its limitations should not be lost sight of. In a supersonic flow, every particle of the flow is moving at a velocity exceeding that of a pressure wave and hence may be thought of as a potential source for a Mach wave or a shock wave. On the other hand, in supercritical flow of water, the velocity of flow exceeds only the celerity of a solitary gravity wave of flow height on the surface of the flow. The velocity of flow is evidently nowhere near the velocity of sound in water; unlike the case of supersonic flow of a gas, the special effects in a supercritical flow of water are only surface effects. For example, a pitot tube immersed in a water flow will have almost the same coefficient in subcritical and supercritical flows, whereas a pitot tube inserted in a gas flow will have quite different coefficients for subsonic and supersonic flow regimes. Other factors which limit the usefulness of the gas dynamics analogy are : fluid viscosity and the associated boundary layer effects on the gravity waves, and surface tension, giving rise to capillary waves. The finite height of the waves and and the small depth of flow further complicate the phenomenon. Nevertheless, the gas dynamics analogy gives quite useful results when applied to the design of supercritical transitions.

It is possible to design a supercritical expansion in such a way that: (1) there is no separation of the high velocity sheet in the transition; (2) shock waves are absent, and (3) the negative disturbances created by the initial outward wall deflection are cancelled as they reach the opposite wall, so that the flow beyond the transition will be free from disturbances. The ideal expansion thus consists of, (1) on initial expansion AB (Fig.3.5) of a reasonably good shape that lies within the boundary profile of a freely expanding rectangular jet at the same Froude number; (2) a straight tangent at the peak wall inclination θ_{max}. Upto C where the first wave from the opposite wall meets the tangent BC; and (3) a " region of wave cancellation" CD in which the negative waves from the opposite wall are just cancelled by an appropriate inward wall deflection. Because of the symmetry, the centre line of the channel can be thought of as an imaginary wall giving rise to wave reflection, as shown in Fig.3.5.

Based on his experiments on the free expansion of rectangular supercritical jets, Rouse[32] suggested that the initial expansion profile may be taken as

$$y/B_f = K \left(X/B_f \, F_1 \right)^{3/2} + 1/2 \qquad (3.13)$$

The constant K can be varied from 1/2 to 1/8 according as whether a short or a long (and better) transition is desired.

Although the method of weak waves is based on the questionable assumptions of zero friction and zero bed slope, these two effects are to some extent mutually cancelling, with the result that the results given by this method are sufficiently accurate for most practical purposes. Given the initial flow conditions and the final width at the end of the expansion, the conventional method of design involves the choice of a trial value of the peak wall deflection θ_{max}, the determination of the corresponding final width on the basis of wave cancellation, and trial and error adjustment of θ_{max}. Such that the computed final width equals the given final width. A more straightforward design procedure, not involving any trial and error, has been given. A new function G has been introduced so that F_2 can be computed directly.

Applying the continuity equation to the flow upstream and downstream of the transition,

$$B_f\, d_1^{3/2}\, F_1 = B_c\, d_2^{3/2}\, F_2$$

or

$$d_2 / d_1 = (B_f\, F_1 / B_c\, F_2)^{2/3} \qquad (3.14)$$

Since energy losses are neglected,

$$d_1 (1 + 1/2\, F_1^2) = d_2 (1 + 1/2\, F_2^2)$$

$$d_2/d_1 = (1 + 1/2\, F_1^2)/(1 + 1/2\, F_2^2) \qquad (3.15)$$

Equating the two expressions (3.14 and 3.15) for d_2/d_1 and rearranging the terms.

$$F_2^{2/3} /(1 + 1/2\, F_2^2) = (F_1^{2/3}/ 1 + 1/2\, F_1^2) \times \left[1/ (B_c/B_f)^{2/3} \right].$$

Denoting the function $F^{2/3} /(1 + 1/2\, F^2)$ as G,

$$\text{Therefore, } G_2 = G_1 / (B_c / B_f)^{2/3} \qquad (3.16)$$

Making use of a graph of G Vs. F (Fig.3.6), G_2 can be computed, and hence the downstream Froude number F_2 is determined.

Equation (3.9b) shows that any value of the Froude number is associated with a corresponding value of the function θ (F). Hence, the Froude number increases from F_1 to F_2 calls for a total flow direction of ($\theta_2 - \theta_1$) , where $\theta_2 = \theta(F_2)$ and $\theta_1 = \theta(F_1)$. But the final flow is obviously in the same direction as the initial flow. It follows that half of $(\theta_2 - \theta_1)$ is brought about by the negative waves originating from the initial expansion, while the remaining half is accounted for as the flow crosses the reflected negative waves and is brought back to the axial direction. Hence, the peak flow deflection, given by

$$\theta_{max} = \frac{1}{2} (\theta_2 - \theta_1),$$ is also determined.

The graph of θ Vs. F (Fig.3.6) helps the evaluation of θ_{max}.

The remainder of the design consist in fixing the complete wall profile ABCD and constructing the wave pattern of the flow in the expansion.

3.4. BLAISDELL'S GRAPHICAL METHOD FOR THE CONSTRUCTION OF THE FLOW IN EXPANSIONS :

In 1944, Blaisdell[34] presented a graphical procedure for the construction of supercritical flow in transition by the method of characteristics. Two templates are needed (Fig.3.7) for applying this method, and may be cutout of 3 mm. perspex sheet. The figure also gives the polar coordinates of the epicycloidal curve for a maximum radius vector of 30 cm.

It is best to describe the graphical procedure with respect to the specific example of a curved supercritical expansion.

The zone numbering system described here is somewhat more meaningful than that originally proposed by Blaisdell. All the weak waves considered in a problem are of equal strength, capable of a flow deflection of approximately 2^{o}. The exact value will depend on the value of θ_{max}. In Fig.3.5, AO represents the initial flow. As the initial expansion of the boundary turns the flow, the flow passes through zones, A_2, A_4, etc. the number always representing the approximate inclination of the flow to the axial (or initial) direction. When the flow crosses the reflected waves, the prefix A changes to B, C, etc. For example, in zone D_2, the flow has crossed three reflected waves and is inclined at nearly 2^{o} to the axis.

3.5. RIGOROUS SOLUTION FOR THE SUPERCRITICAL FLOW IN EXPANSION:

Given the boundary geometry of an expansion, supercritical flow in the expansion can be analyzed rigorously taking into account the bed slope of the transition and friction loss. However, the analysis can not normally be applied to constructions because of the formation of shock in constructions.

Assuming two-dimensional planar flow with negligible vertical acceleration and hydrostatic distribution of pressure, the differential equation becomes[35].

$$u\frac{\partial d}{\partial x} + d\frac{\partial u}{\partial x} + v\frac{\partial d}{\partial y} + d\frac{\partial v}{\partial y} = 0 \qquad (3.17)$$

$$u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial d}{\partial x} = g\ (S_o - S_{fx}) \qquad (3.18)$$

$$u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial d}{\partial y} = -g\ S_{fy} \qquad (3.19)$$

where x and u are the coordinate of the velocity in the downstream directions, y and v the coordinate and the velocity in the lateral direction, d is the depth of flow, g the acceleration due to gravity, So the channel slope, and S_{fx} and S_{fy} are the component friction slopes in x and y directions. Adopting Chezy's formula for the friction slope,

$$S_{fx} = \frac{u\sqrt{u^2 + v^2}}{C^2 d} \tag{3.20}$$

$$S_{fy} = \frac{v\sqrt{u^2 + v^2}}{C^2 d} \tag{3.21}$$

The dependent variable are u, v, d, which are unique functions of x and y, defined by the boundary geometry and the initial conditions. Equations (3.17) to (3.19) form a set of quasi-linear hyperbolic partial differential equations, and reduce to a three-characteristic problem when solved by the method of characteristics. Of the three characteristics, one is the streamline itself while the other two are inclined to the velocity vector at the Mach angle $\sin^{-1}(\frac{1}{F})$. The equation can be solved in a computer by transforming them to the corresponding finite difference equations. The solution gives u, v, and d at every point of the flow.

Liggett and Vasudev[36] have reported that since the effects of bed slope and channel friction are to some extent mutually cancelling, the "frictionless, zero-slope design" is entirely adequate for practical problems on gradual channel expansions. The velocities and depths calculated by the simple method of characteristics may be relied upon to an accuracy of 10% or so.

3.6 EXAMPLE :

Design a supercritical expansion, given $B_f = 2$ m, $d_1 = 0.5$ m, $B_c = 4$ m, and $F_1 = 2$.

Referring to Fig. 3.6 $G_1 = 0.528$ and $G_2 = 0.528 \times .5^{.67} = 0.333$

Referring to the graph, $F_2 = 3.40$

Again, from Fig. 3.6, $\theta_1 = 18.0^o$ and $\theta_2 = 34.6^o$

Therefore, $\theta_{max} = \dfrac{34.6-18.0}{2} = 8.3^o$

This is rounded of to 8.4^o so that the total wall deflection may be considered in 4 steps of 2.1^o each. This exact value of $\theta = 8.4^o$ will be used in the graphical construction, but it will be taken as 8^o for purposes of zone numbering.

The initial expansion follows the expression in eq. (3.13). Assuming $k = 1/4$,

$$y/B_f = \frac{1}{4} \left(\frac{x}{B_f\ F_1}\right)^{3/2} + 1/2$$

Differentiating,

$$\frac{dy}{dx} = \tan \theta = \frac{3}{8F_1} \left(\frac{x}{B_f\ F_1} \right)^{1/2}$$

The initial expansion divided into 4 arcs of 2.1^o each. By putting $\theta = 2.1^o$, 4.2^o, etc. in the above expression, x and y can be solved.

In Fig. 3.5, AO represents the initial flow. As the flow expands, we move to AB on the diagram. Thereafter as the flow crosses the reflected waves, we move back to EO of interest in the characteristics diagram and is filled up with the left and right-running characteristics at intervals of 2.1^o.

As the boundary deflects through an angle of 2.1^o, the flow properties correspond to A_2 on the characteristics diagram. The corresponding disturbance line is drawn on the flow diagram, starting from the point of mid-slope of the walls and parallel to a normal to the arc $A0 - A_2$ of the characteristics diagram at the mid point of the arc. When this disturbance line reaches the imaginary wall at the axis of the channel, it gets reflected as a new negative wave. The graphical construction progress step by step, as the primary and reflected wave interact in the flow diagram producing wave refraction. When the first reflected wave reaches the wall at point C on the tangent BC inclined at θ_{max}, the region of wave cancellation begins. At C, the boundary is turned inward by 2.1° so that the negative wave which would otherwise have been produced by reflection of the incident wave is just cancelled out by the positive wave which would normally result from an inward wall deflection. Finally, the wall is brought back to the axial direction at D.

Check whether the final width at the end of the expansion is equal to B_C. If there is any slight discrepancy, the necessary correction to the expansion profile can be easily made graphically. Starting from the end of the expansions, D is shifted to D' (Fig.3.5) and the corrected segments of the wall profile are drawn backwards, such that they are parallel to the corresponding original segments. When the profile reaches the initial expansion zone, the readjustment necessary will be barely discernible.

For each zone of the flow diagram, $\overline{V}$, and hence F can be obtained from the characteristics diagram. d/d_1 can also be worked out so as to give the water surface profile. The final depth at the end of the transition is only 0.44 times the initial depth. Unless the bed slope is increased beyond the transition, a S_3 profile forms after the transition, attaining normal depth asymptotically.

Although the above example has been worked out with 2.1^{o} wall elements for the sake of clarity, a practical design should employ much smaller elements of 0.5^{o} or so. It may be seen from the graphical construction that the smaller the wall elements, the greater will be the length of transition indicated by the graphical construction. In order to correct the length of transition for the element size, the limiting or last disturbance line originating from point B in (Fig.3.5) should be traced. The point where this line meets the downstream wall beyond D yields the corrected length of expansion. In Fig.3.5, such a construction would yield a length of 13.2 m. A computer study gives θ_{max} as 8.23^{o} and the corrected length of expansion as 13.1. m.

Recent experiment indicate that the low-curvature region near the end of the expansion can be shortened by as much as 10% of the corrected length of expansion without significantly affecting the performance of the expansion. The factors that critically affect the performance of the expansion are θ_{max} , the correct location of point C and the correct shaping of the wave-concellation region immediately following, and in the vicinity of C.

3.7. OBLIQUE SHOCK WAVE AND DESIGN OF CONTRACTION:

Just as in gas dynamics, an inward wall deflection of a high velocity flow usually gives rise to a shock wave, whether the flow is turned suddenly or somewhat gradually. In the case of gradual inward wall deflection, the positive disturbance lines converge downstream and eventually merge into a shock-wave. The shock-wave manifests itself as an oblique hydraulic jump accompanied by energy loss.

We have seen that the obliquity of a standing wave can be written as

$$\sin \beta_1 = \frac{1}{F_1}\sqrt{\frac{1}{2}\, d_2/d_1\,(1 + d_2/d_1)} \qquad (3.22)$$

Solving for d_2/d_1, we get

$$d_2/d_1 = \frac{1}{2}\left[\sqrt{1+8F_1^2 \sin^2 \beta_1} - 1\right] \qquad (3.23)$$

Also, $V_{n1} = V_{t1} \tan \beta_1$; $V_{n2} = V_{t2} \tan(\beta_1 - \theta)$, $V_{t1} = V_{t2}$; and $d_1 V_{n1} = d_2 V_{n2}$. So,

$$d_2/d_1 = \tan \beta_1 / \tan(\beta_1 - \theta) \qquad (3.24)$$

Combining eqs.(3.23) and (3.24)

$$d_2/d_1 = \frac{1}{2}\left[\sqrt{1 + 8F_1^2 \sin^2 \beta_1} - 1\right] = \frac{\tan \beta_1 + \tan \theta \tan^2 \beta_1}{\tan \beta_1 - \tan \theta} \qquad (3.25)$$

$$\tan \theta = \frac{\tan \beta_1 \sqrt{1+8F_1^2 \sin^2 \beta_1} - 3}{2\tan^2 \beta_1 - 1 + \sqrt{1+8F_1^2 \sin^2 \beta_1}} \qquad (3.26)$$

$$F_2 = \sqrt{d_1/d_2\left[F_1^2 - \frac{1}{2}\, d_1/d_2\,(d_2/d_1 - 1).(d_2/d_1 + 1)^2\right]} \qquad (3.27)$$

It may be noted that when $\beta_1 = 90^\circ$, the equations for the normal hydraulic jump are obtained.

$$\frac{d_2}{d_1} = \frac{1}{2}\left[\sqrt{1 + 8F_1^2} - 1\right] \qquad (3.28)$$

$$F_2 = \frac{F_1}{\left[1/2\left(\sqrt{1 + 8F_1^2} - 1\right)\right]^{3/2}} \qquad (3.29)$$

It is interesting to plot the variation in the value of the dimensionless velocity $\overline{V}$ through an oblique jump (Fig.3.8). Let A represent the initial value of the dimensionless velocity $\overline{V}_1$. If a wall deflection θ produces on oblique jump, point B will represent $\overline{V}_2$ after the jump. The locus of point B for various values of θ gives rise to the curve shown in Fig.3.8, well-known as the " shock polar " in gas dynamics.

A circle with radius $\overline{V} = 0.577$ separates supercritical and subcritical flow regimes. Point C represents the flow condition below a normal hydraulic jump.

The energy loss in the jump is given by $\Delta H = (d_2 - d_1)^3 / 4\, d_1 d_2$. For $d_2/d_1 = 2$, ΔH is approximately $1/8\ d_1$. The result of energy loss is that, if two successive flow deflections occur, the flow characteristics at the second deflection can not be related to the initial shock polar.

Although the oblique shock wave is an interesting hydraulic phenomenon, it has not been studied in as much detail as its counterpart in gas dynamics. The actual obliquity of the shock wave in water has been found to be slightly different from the theoretical value, indicating the presence of secondary effects that have not been investigated in detail.

2.8. SHOCK WAVE CHART:

It is possible to prepare a set of charts giving the values of β_1, F_2 and d_2/d_1 for any given set of values of F_1 and θ such a chart (Fig.3.9) is quite helpful in the quick design of supercritical contraction.

2.9. DESIGN PROCEDURE FOR CONTRACTIONS:

The aim of rational design for supercritical flow must be oriented, first, towards lower standing waves and, second, toward reduction or possible removal of standing wave pattern in the channel section downstream from the contraction.

A practical supercritical contraction is invariably accompanied by a shock-wave, as it is not possible to eliminate the shock unless the contraction is made excessively long. Since the height of the shock wave is a function of the deflection, the prism design criterion is low peak deflection of the flow.

Typical side wall profiles for contractions are as shown in Fig.3.10. Since the straight contraction produces the least flow deflection in a transition of given length, straight contractions with slight rounding of the corners are preferable to curved contractions.

The ideal supercritical contraction should be designed such that the positive shock wave created by the initial inward turn of the wall is just cancelled by the negative waves produced by the subsequent outward turn into the downstream channel. Theoretically the downstream channel should then be free from waves.

Referring to Fig.3.11, shock-wave cancellation will be achieved if the following geometrical condition is satisfied.

$$\frac{B_c}{2} \cot \beta_1 + \frac{B_f}{2} \cot \ (\beta_2 - \theta) = \frac{B_c - B_f}{2} \cot \theta$$

$$\frac{B_f}{B_c} = \frac{\cot \theta - \cot \beta_1}{\cot \theta + \cot (\beta_2 - \theta)} \qquad (3.30)$$

For a given initial Froude number and depth ratio, the optimum value of θ for shock-wave cancellation can be obtained by trial and error.

As in subcritical flow, excessive contraction of the supercritical flow can be lead to choking of the flow. However, two distinct choking mechanisms are involved here. A part from the more common mode of choking leading to critical conditions below the contraction, there occurs a second mode of choking by the formation of a jump upstream of the contraction, the contraction then functioning as a subcritical contraction producing critical conditions downstream. It can easily be shown that all initial Froude numbers, the second mode of choking would set in first, as the contraction is increased. Since choking of the supercritical flow in a contraction may produce violent disturbances in the flow with the consequent risk of overtopping of the channel, it is desirable to make sure in the design that choking of the flow would not occur.

3.10.1. EXAMPLE :

Design a straight supercritical contraction for $B_c = 4$ m, $d_1=0.5$m, $F_1 = 3$, and $B_f = 3.0$ m.

The contraction ratio $B_f / B_c = 0.75$. The wall inclination should be evaluated by trial and error such that the computed value of B_f/B_c also equals 0.75.

First Trial :- Let $\theta = 3^o$

Referring to the graphs in Fig.3.9 for $F_1 = 3$ and $\theta = 3^o$, $\beta_1 = 22.2^o$ and $F_2 = 2.72$ (by estimation)

Again, for $F_2 = 2.72$ and $\theta = 3^o$, $\beta_2 = 24.4$ (by estimation).

Computed $\frac{B_f}{B_c} = \frac{\text{Cot } 3^\circ - \text{Cot } 22.2^\circ}{\cot 3^\circ + \cot(24.4-3)^\circ} = 0.768$

Second Trial:

Let $\theta = 4^\circ$

proceeding as before, $\beta_1 = 23.2$, $F_2 = 2.64$ and $\beta_2 = 25.8^\circ$

Computed $\frac{B_f}{B_c} = 0.712$

Now, interpolating for the actual value of $B_f/B_c = 0.75$, we get $\theta = 3.32^\circ = 3.3^\circ$ (say), $F_2 = 2.7$ and $F_3 = 2.42$, $d_2/d_1 = 1.19$, $d_3/d_2 = 1.18$, and $d_3/d_1 = 1.40$

Check continuity :

$$B_c \; d_1^{1.5} F_1 = 4 \times 0.5^{1.5} \times 3 = 4.24$$

$$B_f \; d_3^{1.5} F_3 = 3 \; (0.5 \times 1.4)^{1.5} \times 2.42 = 4.25$$

The expressions check well.

Length of Transition :-

The total length of the contraction is given by

$$L = \frac{1}{2} (B_c - B_f) \cot \theta = 8.67 \text{ m.}$$

3.10.2. CHECK FOR THE POSSIBILITY OF CHOKING:

When the second mode of choking set in, the Froude number F_3 below the contraction drops to 1, and the contraction behaves subcritically. Neglecting frictional loss in the contraction, for $F_3=1$ and $B_f/B_c = 0.75$, the Froude number F_2 just upstream of the contraction can be determined as follows :

For $F_3 = 1$, the function $G = 0.667$. For $B_f / B_c = 0.75$ the value of the function just upstream of the contraction is given by :

$$G_2 = G_3 \left(B_f / B_c \right)^{2/3} = 0.667 \times 0.826 = 0.551$$

By trial and error, the corresponding $F_2 = 0.48$. If this Froude number should prevail below a normal hydraulic jump, the upstream Froude number F_1 should be equal to 2.4. Since the actual value of F_1 is greater than this, choking of the flow can not occur.

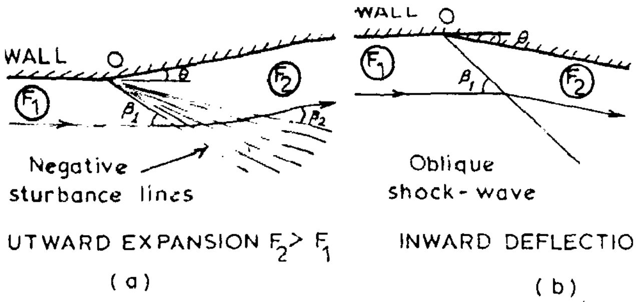


FIG. 3.1

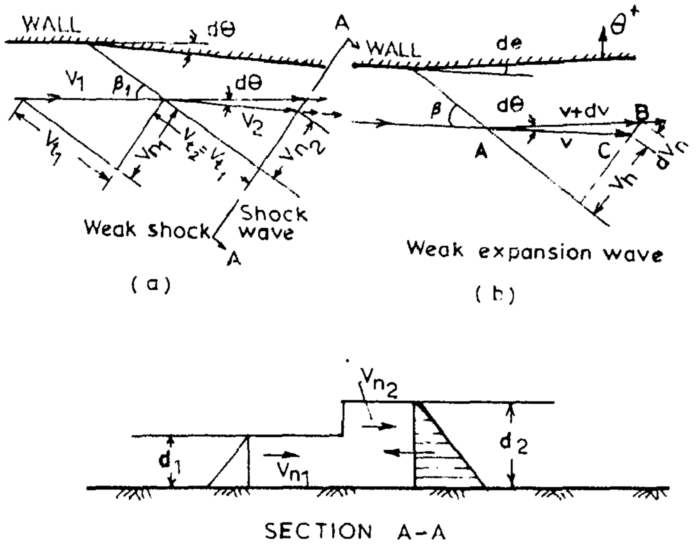


FIG. 3.2

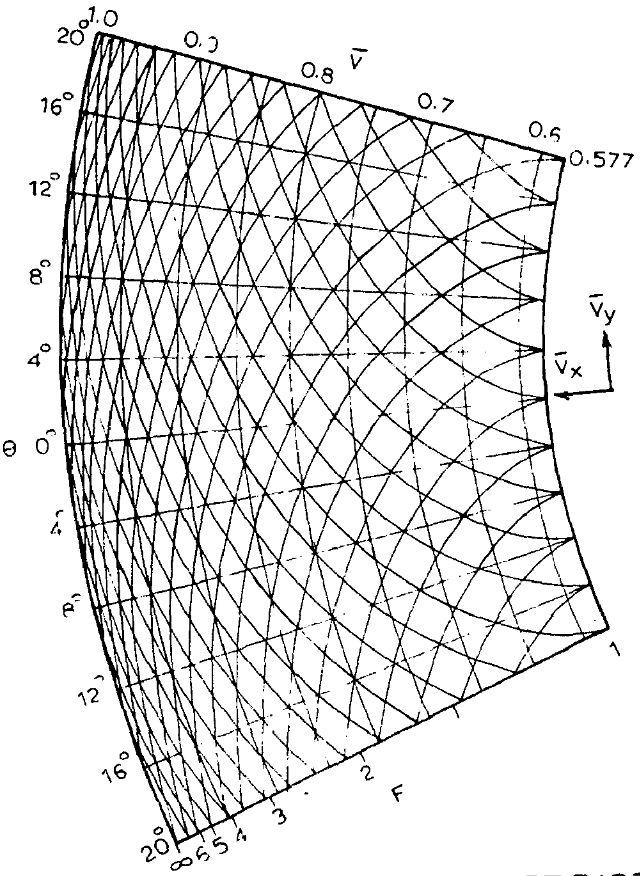


FIG. 3.3 _ CHARACTERISTICS DIAGRAM

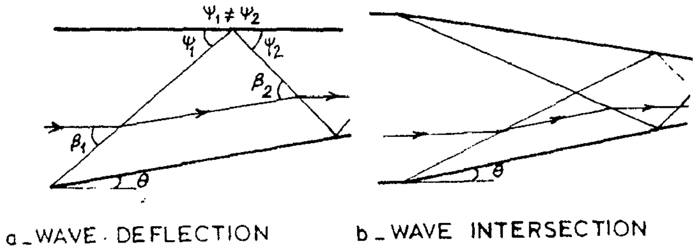
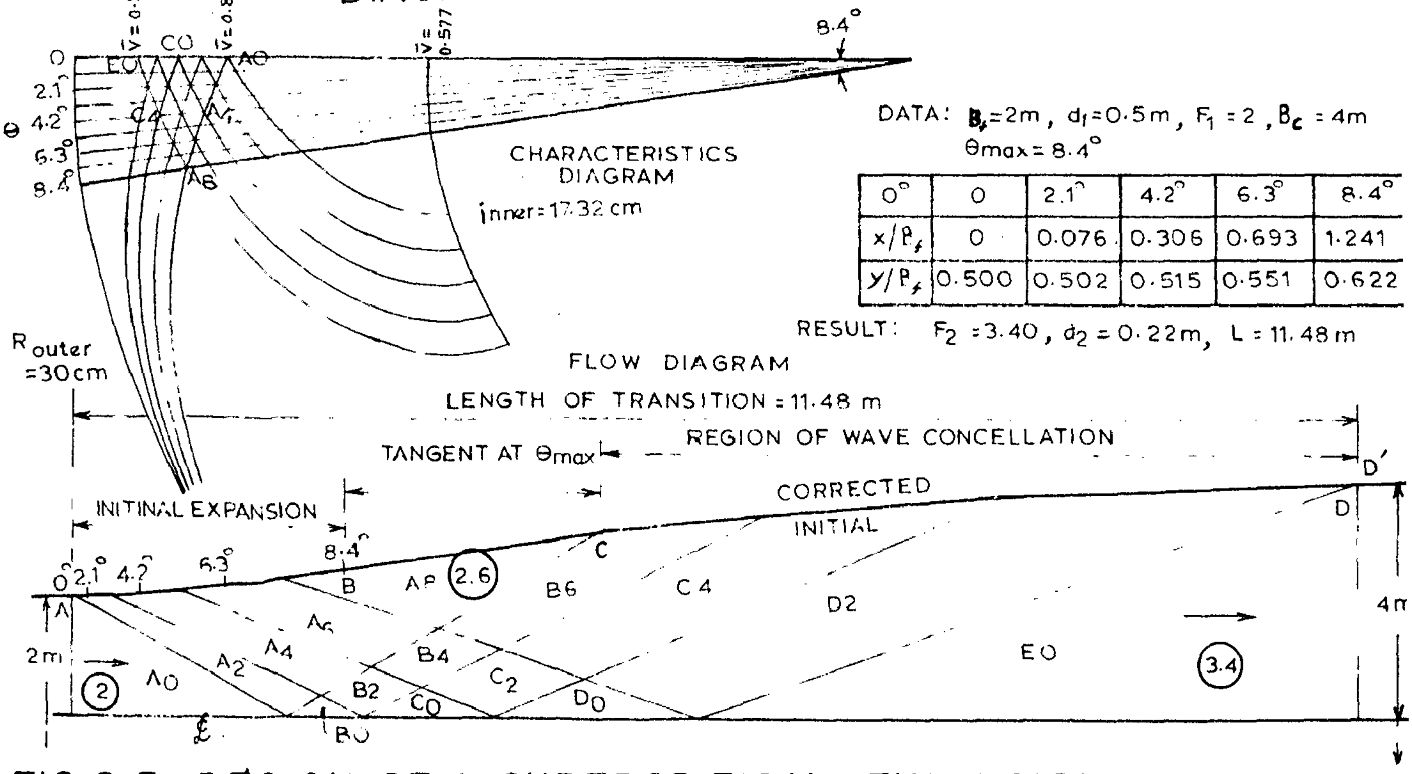


FIG. 3.4



DATA: $B_f = 2$m, $d_1 = 0.5$m, $F_1 = 2$, $B_c = 4$m
$\theta_{max} = 8.4°$

θ°	0	2.1°	4.2°	6.3°	8.4°
x/B_f	0	0.076	0.306	0.693	1.241
y/B_f	0.500	0.502	0.515	0.551	0.622

RESULT: $F_2 = 3.40$, $d_2 = 0.22$m, $L = 11.48$m

FIG. 3.5 _ DESIGN OF A SUPERCRITICAL EXPANSION

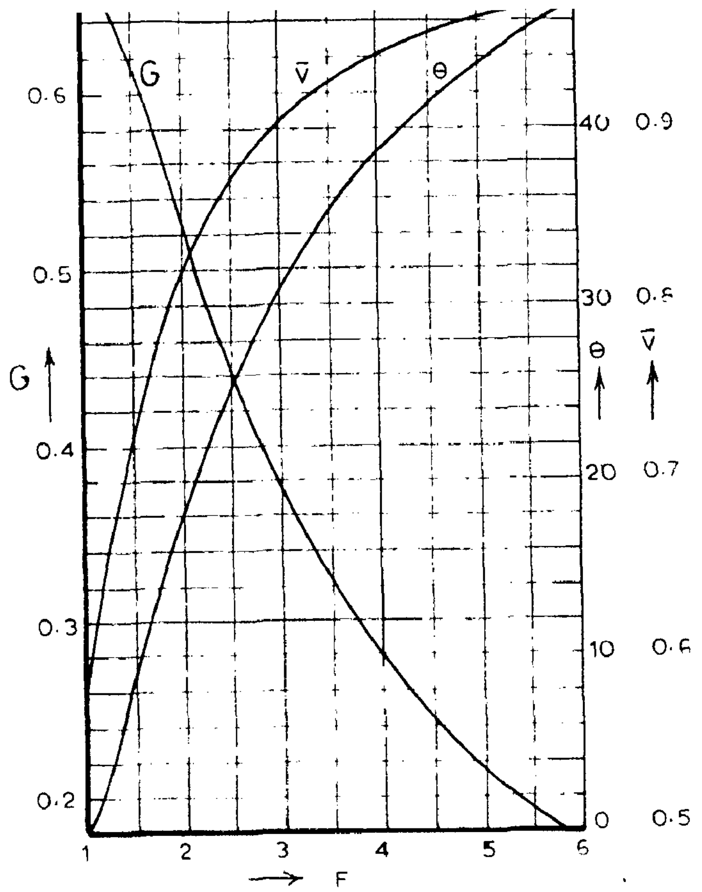


FIG. 3.6_DESIGN CHART FOR SUPERCRITICAL EXPANSIONS

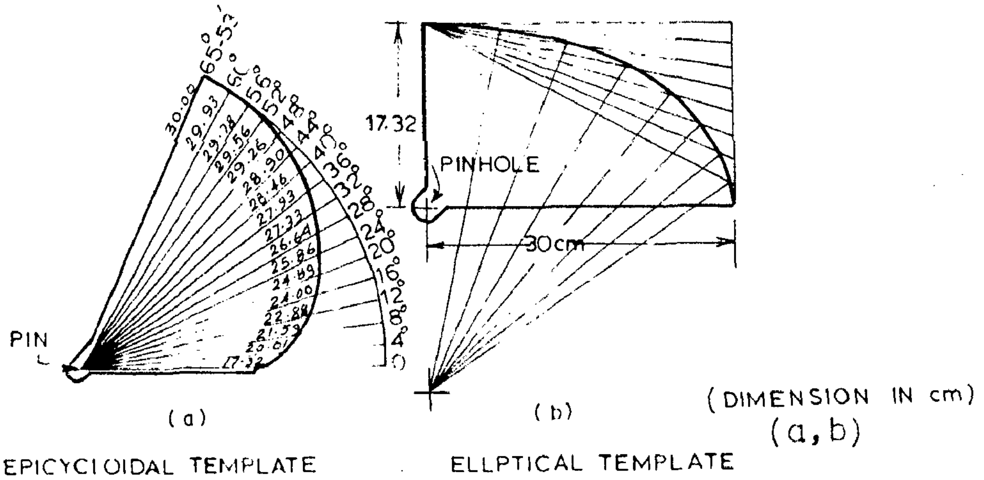


FIG. 3.7

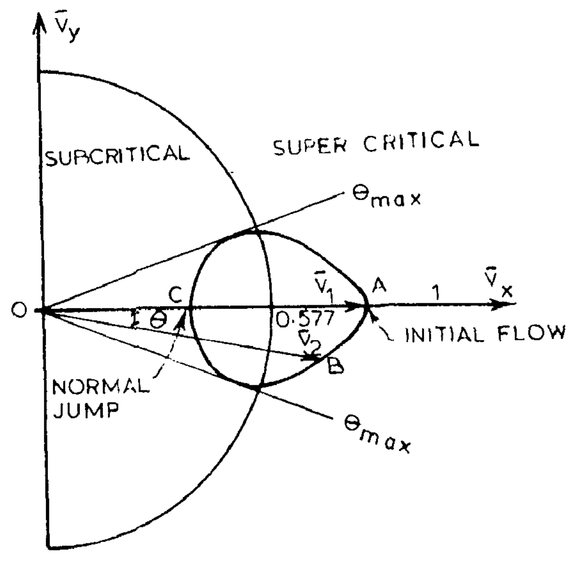


FIG. 3.8_ SHOCK POLAR

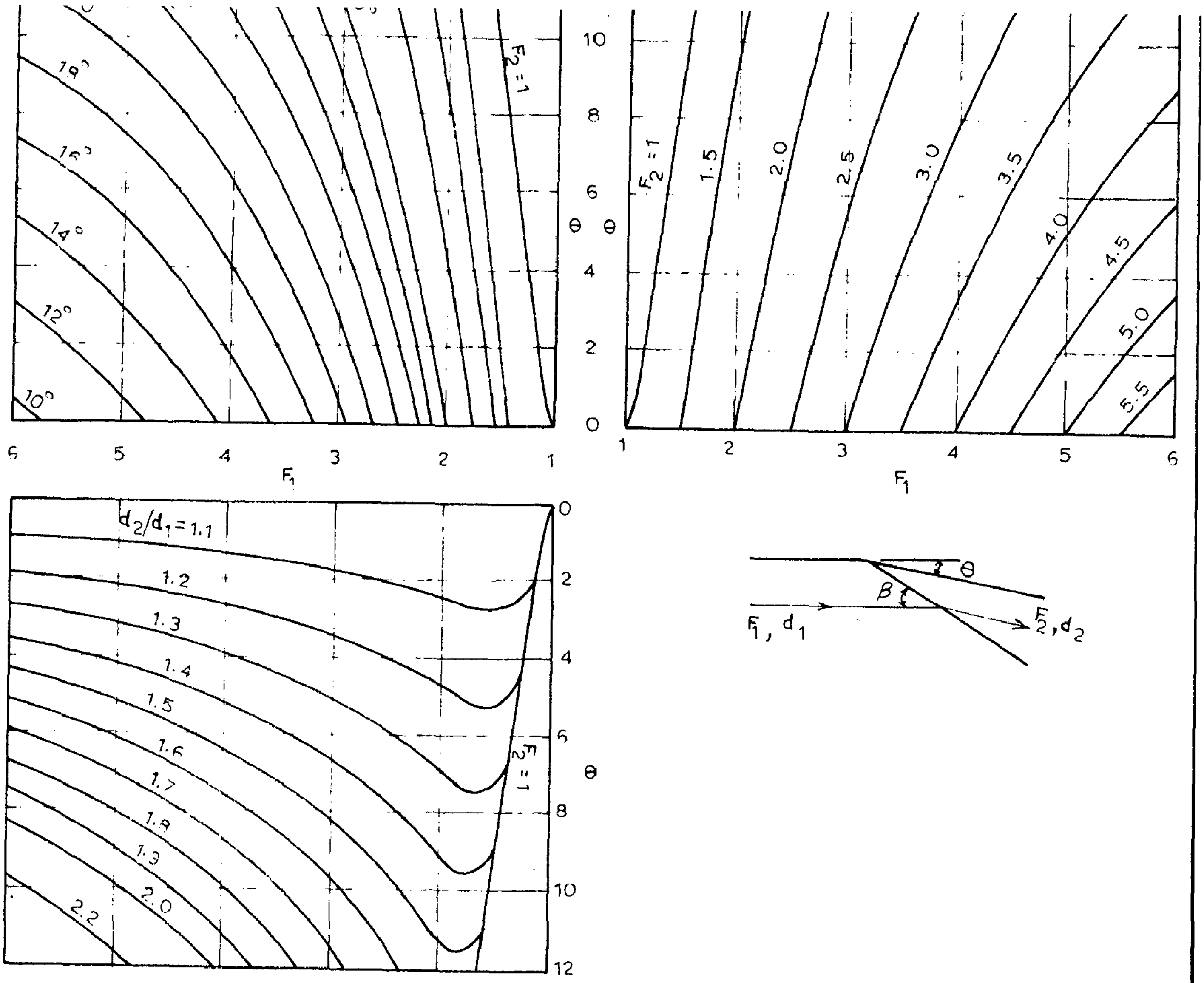


FIG. 3.9 _ DESIGN CHART FOR SUPERCRITICAL CONTRACTIONS

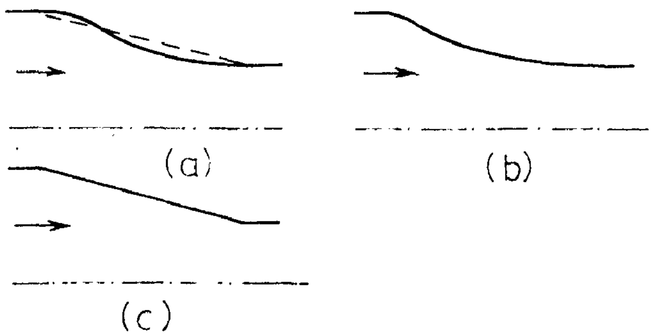


FIG. 3.10 _ CONTRACTION PROFILES

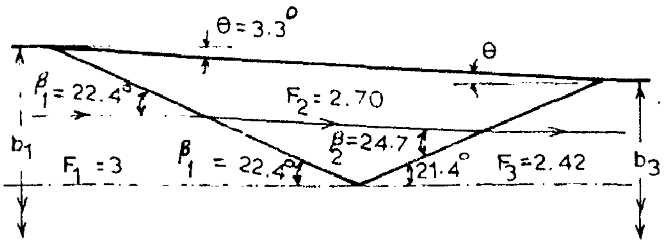


FIG. 3.11 _ CONTRACTION DESIGN

CHAPTER-IV

DISCUSSION AND CONCLUSIONS

4.1. INLET TRANSITION:

The flow of water is 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. By making various assumptions of more than doubtful validity, it is quite possible to arrive at a result, seemingly correct, to any number of decimal places, but which is not in fact any more accurate than the assumptions on which it is based.

In the past, a common design for low velocity contractions has been the warped wall reversed parabola transition, first proposed by Hinds[2]. The criteria for design recommended in Hinds' paper are, " .... a straight line joining the flow line at the two ends of the transition ...... will make an angle of about $12\frac{1}{2}^{\circ}$ (degrees) with the axis of the structure," and " .... the computed water surface profile through the transition shall be a smooth, continuous curve, approximately tangent to the water surface curves in the channels above and below " [2]. The first recommendation determines the length for the transition, and the second the geometric form. The selection of $12\frac{1}{2}^{\circ}$ was made arbitrarily, although it was undoubtedly known that this angle would give a gradual contraction. The average rate of side wall convergence is approximately 1 in $4\frac{1}{2}$.

The connecting surface curve between the upstream and downstream water surface profiles is an S-curve shape. This may be drawn as a reversed parabola consisting of two equal parabolas; however, it is permissible to draw in any smooth surface using a French curve. The drop in water surface elevation is taken as $1.1 \Delta h_v$, which allows $0.1 \Delta h_v$ for enterance head loss. The total energy line may be drawn with a drop in elevation of $0.1 \Delta h_v$ in the length of the transition.

The ends of the transition are shaped to fit the respective channel shapes. Normally this is trapezoidal at the start of the transition and rectangular at the end, and therefore a " warped wall " or three-dimensional curved surface is required. The shape of the transition at intermediate section is determined by trial and error. The floor profile and side walls must be smooth curves throughout their length, and the cross-sectional area at each section must satisfy the equation

$$A = Q / \sqrt{2g\ (TEL - WS)} \qquad (4.1)$$

in which TEL-WS is the vertical distance between the total energy line and the water surface as previously drawn. The procedure is based on the assumption that there is no separation and that the water surface is level transversely. A typical design for the warped wall inlet transition is shown in Fig.2.23.

The warped wall transition design has been accepted by a number of agencies. It meets all of the hydraulic requirements previously listed, and in fact, the hydraulic performance can not be improved upon. However, the structure is simple to design, either hydraulically or structurally and it is relatively expensive. The hydraulic design requires a trial and error procedure, the structural design involves varying steel and a counterfort wall, and the complicated forming and great length of the structure add greatly to the cost.

Hinds transition, used as inlet, is unnecessarily elaborate and costly. It is believed that a completely satisfactory transition can be designed without using warped walls, and with a much shorter length than is used in the Hinds transition.

Flow at subcritical velocities in converging transitions is inherently stable and self-streamlining. It should not be necessary to use a gradual warp, or indeed any kind of a curve, in regions where the velocity is naturally low, such as at the start of the converging transition. Of course, in regions of high velocity in and near the end of the transition, there must be a streamlined convergence. However, the streamlining may be constructed with vertical walls throughout simply by extending the vertical walls upstream into the low velocity region at the start of the transition.

Based on the theory presented above, several inlets were designed and model tested by Clifford D. Smith[39]. The hydraulic dimensions of the inlet finally recommended are shown on the definition sketch in Fig. 4.1 The length of this inlet is from one-half to one-third of the length of the Hinds S-curve inlet for comparable conditions. The width at the inlet, B, is the same as the average water width in the approach canal, that is, half the sum of the bed width and the surface width. The side walls of the inlet converge rapidly at 1 lateral to 1 $\frac{1}{2}$ longitudinal, using a straight vertical wall. The straight wall is tangent to a simple curve with a tangent distance $T = 0.5\ (B_a - B_f)$ and a radius $R_o = 3.302\ T$. The end of the curve is tangent to the side wall of the contracted section, giving a transition length of $L = 1.25\ (B_a - B_f)$. It should be noted that since the bed width is less than B_a, the toe of the side slopes actually extends into the inlet a short distance.

The experimental programme carried out to verify the adequancy of the proposed inlet included 3 areas of investigation :
(1) Flow conditions in and downstream from the transition; (2) head losses caused by the transition; and (3) water surface profiles in the transition.

(1) Flow conditions:- The model in Fig.4.2 is shown in operation at design discharge in Fig.4.3 The Froude number downstream from the transition is 0.6 for the test shown. This Froude number is designated as $F_2 = V_2 / \sqrt{gd_2}$, in which V_2 and d_2 are the downstream velocity and depth respectively.

As indicated in Fig.4.3, the flow is smooth and tranquil throughout. There is no flow separation in the transition, even at the relatively abrupt inlet. This somewhat surprising result may be attributed to the fact that in the approach channel the shallow water lying on the side slopes, outside the width B_a, has a very low natural velocity and contributes little to the discharge.

The simple curve, although giving a more rapid convergence than the S-curve warped wall inlet, produced smooth downstream flow conditions. This hydraulic performance was found to be valid upto a value of $F_2 = 0.67$. Beyond this value, the flow in the contracted section began to show surface undularity, and the undularity increased greatly as the Froude number approached unity. However, this flow condition was found to occur with the S-curve inlet as well. It may be differred that if a smooth downstream water surface is desired, the maximum Froude number should be limited to about 0.67. This is not a particularly serious limitation, since it would allow, for example, a velocity of 2.44 m/sec with a depth of 1.37 m.

Despite the use of fine sand in the trapezoidal approach section, there was no evidence of any scour near the entrance. Test runs of 1 hr. or more did not produce any measurable change in the channel, including the toe slopes at the entrance.

(2) Head Losses : To measure head losses over a range of Froude numbers, head loss tests were carried out using a fixed boundary tilting glass sided flume, 0.305 m. wide and 11.28 m. long.

The method used to determine the head loss is illustrated on the definition sketch in Fig.4.4 Depths were calculated from readings taken with a vernier point gauge mounted on a travelling carriage. The gauge read directly to the nearest 0.0003 m., but readings could be estimated to the nearest half division, or 0.00015 m.

Four measurements were required for each head loss determination: (1) The floor slope S_o; (2) the discharge Q; (3) the upstream depth d_o ; and (4) the downstream depth d_2. The upstream and downstream velocity heads were computed from

$$h_{vo} = \frac{(Q / B_f d_o)^2}{2g} \qquad (4.2)$$

and

$$h_{v2} = \frac{(Q / B_f d_2)^2}{2g} \qquad (4.3)$$

The corresponding specific energy was given by

$$E_o = d_o + h_{vo} \qquad (4.4)$$

and

$$E_2 = d_2 + h_{v2} \qquad (4.5)$$

The depth d_o was measured at a distance La upstream from the contracted section, in a region beyond the influence of local drawdown due to the inlet transition. Since the friction loss in the approach section was negligible in comparison to the floor slope, the equivalent position of the total energy level at the start of the contracted section was found from

$$E_1 = E_o + S_o La \tag{4.6}$$

The head loss to be charged to the inlet was calculated from

$$H_L = E_1 - E_2 \tag{4.7}$$

and the coefficient of loss from

$$C_L = H_L / h_{v2} \tag{4.8}$$

The depth d_2 in the contracted section was controlled by the tailgate at the end of the flume. If the tailgate was too low, the water surface in the contracted section was concave down. If the tailgate was too high, the water surface was concave up. It was necessary to set the tail gate repeatedly and read d_2' until a setting was found for which water surface was parallel to the floor.

Data and calculations for the inlet loss for the two inlets are collected [39]. All observations were taken in the Froude number range 0.28 to 0.67. It was impossible to make accurate observations at a lower Froude number because the velocity head and head loss became to small for accurate measurement. The specific energy, E_1 and E_2 was calculated to four decimal places despite the fact that d_o and d_2 were taken only to the nearest 0.00015 m. This was done because the slope S_o could be easily read to four decimals (only two significant figures), and the velocity heads could be calculated to four decimals (three significant figures).

If these values were " rounded off " prematurely, the scatter of the final result was considerably increased.

Despite the care taken in the tests, the scatter in the data for the coefficient of loss is ± 100%. However, this result is not as bad as it seems. The head loss, which is a very small value, is calculated as being the difference between two very large values, E_1, and E_2. A slight percentage error in either E_1 or E_2 or both, will give a very large percentage error in their difference. The significant point about the data is collected for head loss is that the head loss is almost negligible for both the S-curve inlet and the recommended inlet. An allowance of $0.1 \Delta h_v$, as recommended for the S-curve inlet, is undoubtedly conservative. In fact, the head loss could be nglected without serious error for either inlet. An adequate allowance for head loss is given by

$$H_L = 0.06 \ (1 - B_f / B_a) \ V_2^2/2g \qquad (4.9)$$

(2) Water Surface Profiles :

Test showed that, were the head loss the only factor to be considered, a satisfactory inlet could be designed even shorter than the inlet shown in Fig.4.1. However, in a very rapid convergence, the velocity distribution across the transition will vary widely, and this will be accompanied by the production of a water surface that is not level transversely. A non level transverse water surface at the end of the transition will result in the development of cross waves, which will persist for a great length in the contracted section downstream. Generally this should be avoided.

Water surface profiles were measured along the wall and down the centre line for the flow condition shown in Fig.4.3.

The observed water surface profiles are plotted in Fig.4.5 Although the water surface in the transition does not remain precisely level transversely, the maximum difference in level does not exceed 1.5% of the depth. This is well within acceptable design limits. In comparable tests on the longer. S-curve inlet, the maximum difference between wall and centre line profiles was only 0.5%, but this small difference is performance is not significant, and certainly can not be used to justify more than doubling the length of the transition.

A final point with regard to the profiles concerns the accuracy with which the flow depth may be calculated from theory at any section. This may be required in the design for transition wall height. The depth d may be calculated from $q = Vd$ and $E = V^2/2g + d$, provided the unit discharge q and specific energy E are known. The calculations may be based on the assumption that $q = Q/B_x$, in which Q is the total discharge and B_x is the width of the transition at the section in question, and $E = E_o \pm \Delta Z - h_{LS}$, in which E_o is the specific energy of the flow in the approach section, and ΔZ and h_{LS} are the change in floor elevation and head loss upto the section in question. The term Z is needed because the floor of the transition may not be level; in fact, in the usual case for a flume inlet there is a drop in elevation between the bed of the approach channel and the floor of the flume. Since the head loss in the transition is small, the effect of the term h_{LS} may be negligible, but it can be included by arbitrarily distributing the total loss over the length L.

Calculations for the profile, corresponding to the test and data for the observed profile are given for comparison. Generally the observed centre line depth agrees very well with the calculated depth always within 1%. The wall depth is both higher and lower than the calculated depth, but always with in 1.5%. It is interesting to note that in the high velocity region between Station 0.0 and -0.6, the calculated depth is between the two observed depths. The calculated depth undoubtedly can be used in design to locate the water surface.

4.2.1. OUTLET TRANSITION:

The proper criterion for determining the best shape should be the optimum flow conditions, as such a curve should also yield the near-optimum head recovery. Considering both the geometry and the positive acceleration, only curves 4,5 and 6 (Fig.2.7) seem to be promising while the divergence of curve 7 is very severe at the exit. It may, however, be noted that for maximum recovery, the curve at the exit may theoretically become asymptotic in the y direction. But the head recovery alone can not be the right criterion. It was thus decided not to use bigger values on n than 2 in the general equation to start with, and to restrict the comparative model examination to these seven transitions amongst them Mitra's hyperbolic expansion (curve 5) is one of the family and also to examine, for the sake of comparison, a Hinds transition for the very same data.

Three series of experiments for these eight transitions were planned in a 0.91 m. wide, 2.44 m. long flume in the University of Roorkee laboratory (India)[12]. In the first series, the constriction was kept 75%, i.e. the throat width was kept 0.23 m. only. A standard flumed approach as shown in (Fig.2.7) was provided upstream of the 0.46 m. long throat. Run with discharges of 0.0283 cumecs, 0.0198 cumecs and 0.0099 cumecs the downstream channel was in each case allowed to maintain normal depth. The splay was made 1 in 5, equal to an angle of diversion of 13°, against a $12\frac{1}{2}^{\circ}$ limit recommended by Hinds. This gave a transition length of 1.72 m. For this series and others, the floor was kept throughout horizontal. It may be mentioned that the requirement of a horizontal floor is quite common on the existing aqueducts, bridges and regulators, etc. Also, if a depressed bed in the throat is desired, the absence thereof would equally influence the flow condition in each transition, and in any case the effects of a slight lowering of the bed has an influence which is secondary to the wall expansion.

In the second series, the constriction permitted, was $62\frac{1}{2}$ %, i.e. the throat width was kept only 0.34 m. as against a channel width of 0.91 m. This is generally considered to be the limit of practical construction rarely permitted for the canal structures, so that the previous series was planned more for theoretical considerations, under the most severe conditions. The splay in the second series was also 1 in 5 or 13°, giving a transition length of 1.43 m. The discharges run were again 0.0283 cumecs, 0.0194 cumecs and 0.0099 cumecs for the second series of experiments.

In the third series, the constriction imposed was 50%, i.e., the throat width was 0.46 m. as against the canal width of 0.91 m. The transition length was 1.14 m. for 1 in 5 expansion as before, and in all other aspects the approach was exactly the same as in the first and second series of experiments. This construction represents normal practice for canal structures.

For simplifying the flow conditions, the comparative models were run with the channel of vertical sides. This makes the investigation a bit unrealistic as the earthen canals are invariably trapezoidal. However, the essential features of the flow would remain the same. If the transition is designed for the bottom and the water spread, the warping smoothly converts the vertical sides into channel side slopes as shown earlier (Fig.2.4) in the example while comparing the hyperbolic expansions with Hinds design. In all the three series, the transitions were made of well seasoned wood which kept friction small and uniform.

4.2.2. DISCUSSION OF RESULTS:

1. First series of Experiments (Constriction 0.23 m. & 0.91 m)

(i) Conditions of flow : The observed water surface and the theoretical water water profile shape, as calculated from one-dimensional considerations referred to earlier, are also plotted to compare the actual values with the theoretical. The latter can only be as accurate as the assumptions.

With this constriction (0.23 m. to 0.91 m), the surface configuration in all the transitions exhibited local disturbance leading to diagonal waves, starting well within the throat and persisting towards the entire length of the transition. The Froude number in the throat was 0.5 and at the exit 0.12 on an average.

As expected, the flow conditions showed improvement to start with precisely according to the anticipated transition characteristics. The gradually increasing lateral acceleration was noticeably reflected in the progressively diminishing size of the eddy, created by separation, in the one-sided flow for curves 1 to 6. The straight line transition 1 and 2, with negative acceleration, were evidently the worst giving the largest eddy with reverse flow.

The conditions, however, again deteriorated with curve 7 when the eddy revived and looked as strong as ever before. It was thus apparent that the optimum conditions were obtained with transition 6 ($n = 3/2$) and not with curve 5 which is Mitra's hyperbolic expansion ($n=1$)

The flow condition of transition 8 (J. Hinds) based on the U.S.B.R. design criterion. The flow conditions, in spite of the ideal water surface profile and the geometry of the structure, are far from satisfactory, the reverse eddy being as bad as in the worst cases of the flow strongly one sided.

It can be seen from the derivation of the curves, based on the lateral velocity, $dy/dt = ky^n$ of the general formula, that the derived formulae are independent of the discharge. In the presence of increased frictional forces and turbulence, with a much bigger discharge in the prototype, it is reasonable to expect that a basic design functioning satisfactorily in the smooth model can prove at least equally efficient on the bigger scale. This also stands verified from a number of field observations in the case of hyperbolic expansions as referred to earlier. Thus, the flow conditions obtained for curve 6, though not very ideal in the model for such severe constrictions, may be taken as tolerable under prototype conditions.

(ii) Effect of B/d ratio :

The expansive flow in each case was examined with 0.0283 cumecs, 0.0198 cumecs and 0.0099 cumecs. It was observed that except for one or two cases, difficult to explain at the moment, the flow conditions generally remained unaffected with the reduced depth of flow upto 0.0170 cumecs. As may be seen, the velocities were not proportionately affected with this discharge, because the flow in each case was maintained at the normal depth. However, with a discharge of 0.0099 cumecs, there was general deterioration and much increased separation at the sides. The experiments, therefore, clearly indicate that the sub-critical flow conditions can deteriorate significantly with the increasing B/d ratio in expansion transitions, only when the discharge is reduced to about 50% for normal depths of flow.

(iii) Efficiency and downstream scour :

Examining the recovery of head, there is a noticeable improvement from transition 1 to 6. The percentage efficiency, based on the available total kinetic energy and head recovery has been examined. The superiority of transition 6 is apparent.

It is not possible to include all the scour charts for comparing such large number of runs (about 50). That the minimum scour was obtained for transition 6 may be mentioned. This is in conformity with the flow conditions.

(iv) Effects of the entry conditions :

The results with longer throat length (5.49 m.) (simulating a long aqueduct upstream) indicated a significant improvement in the flow conditions for all the three transition 5, 6 and 7. However, in transition 6 only the separation effects were almost completely eliminated and the improvement was thus comparatively remarkable.

This brings out very clearly the importance of entrance conditions for the expansion transitions. For short throated structures like bridges and culverts, if the constriction is severe, more elaborate upstream transitions are required than considered necessary at the moment. Only then it is possible to provide satisfactory entrance conditions for the downstream expansive transition.

The percentage recovery of head, however, remained virtually in the same proportion for all the transitions but maximum efficiency of transition 6 was easily maintained.

(v) Theoretical water surface profile :

To calculate the water profile, the frictional loss was taken as negligibly small, and the loss of energy at each section of the transition was assumed only as the expansion or transition loss (approximately 60% of Δh_v) with a uniform variation from the throat of the exit. Thus, knowing the energy of flow and width of transition at each section the stage of the flow is computed by solving the cubical equation given below, from section to section :-

$$\frac{V^2}{2g} + d = E$$

$$\frac{1}{(B_f\ d)^2}\,\frac{1}{2g} + d = E$$

or

$$\frac{1}{B_c^2\ 2g} + d^3 = Ed^2 \qquad (4.10)$$

The theoretical water profiles very clearly indicate the general improvement in shape from transition 1 to 6 Fig.4.6 in conformity with the flow conditions obtained in the models. They are, however, far different from the actually observed water profiles. The deformation of water profile for transition 1 is an index if its complete unsuitability, while the progressive improvement upto transition 6 upholds the anticipated characteristics. It will be seen that transition 5 and 6 automatically give highly satisfactory water profiles, of the two that of transition 6 is more balanced almost an exact reverse parabola. As according to Hinds criterion, which has considerable statistical backing from field results, a smooth water surface profile like that of transition 6, is expected to be in variably successful, this gives an indirect support to the satisfactory geometrical characteristics for transition 6 and also more promising field performance.

(vi) Practical limits of constriction :

It might be recalled that 75% constriction experiments were under taken to start with, more for academic interest than for their practical utility under field conditions. However, as lengths of transitions with divergence 1 to 5 are usually economically justifiable, construction ratio upto 75% with transition 6 can be considered a practical possibility for flumed structures in open channel flow, more so far long throated structures like aqueducts and syphons. It is the accidental satisfactory performance of transition 6 which has brought out this limiting condition.

(2) Second series of Experiments (Constriction ratio 0.34 /0.91)

Performed for obtaining confirmation of the trends obtained in the first series of experiments, as also for examining the effects of variation in the fluming ratio, this constriction is generally attempted for structural design and has wide applications in the field. Only curves 4, 5, 6 and 7 as also the U.S.B.R. design were examined since the others were inferior to be persued any further (Fig.2.7a).

(i) Flow conditions :-

All the five transitions indicated improvement but in number 4 and 5, it was not so significant. Transition 7 was comparatively much better this time, but the high velocity stream remained one-sided although the separation was not so marked. The surface disturbances were generally much subdued in all the transitions. The diagonal streaks starting within the throat disappeared midway within the transition length but persisted longer in transition 7. The Froude numbers for this series was of the order of 0.31 (average) for the throat and 0.11 for the exit for 0.0283, 0.0198 and 0.0099 cumecs discharges.

Transition 6 was again the very best with virtually little surface disturbances, and in this case, even the high velocity thread was absent yielding a broad belt of central stream flow of uniform velocity as evidenced by the flat curves for 0.0283 cumecs and 0.0198 cumecs velocities (Fig.4.7). The only thing which could be objected to was the velocity defect at one end near the exit. The extent of this stagnating pocket was so small that it could at best be considered more a local feature than reflection on the general flow conditions. The downstream scour was also the minimum for transition 6 again.

Except for curve 6 of the second series, the flow for all the transitions, both in the first and second series, in the downstream portion of the channel below the transitions, was invariably getting one sided and persisted as such for a considerable distance downstream.

The flow condition in the U.S.B.R. design for 0.34 m/0.91 m constriction was again far from satisfactory.

(ii) Effect of B/d ratio :

Except for the slight improvement in the flow conditions with 0.0198 cumecs discharge of curves 5 and 6, difficult to explain, there was no significant change in 0.0283 cumecs and 0.0198 cumecs flow as before for all the transitions.

(iii) Head recovery and water surface profile :

In respect of head recovery, the efficiency of transition 6 curve was again maximum. Similarly, the theoretical water surface profile, based on one dimensional flow criterion, was again the most satisfactory for curve 6.

(3) Third Series of Experiments, (Constriction ratio 0.46/0.91)

(i) Flow conditions:-

With 50% constriction all the five curves 4,5,6,7 and the U.S.B.R. design indicated considerable improvement, but the best and most satisfactory performance was obtained with transition 6 yielding a broad belt of exit flow. The hyperbolic expansion, transition 5 curve, was only as good as transition 7, while transition 4 indicated slight reverse flow at one side near exit, and otherwise quite satisfactory.

The velocity profiles of curve 6 are completely flat and almost ideal, those of curves 5 and 7 indicate central deformations indicating one sidedness of maximum velocity thread, also reflected in the scour pattern. Curve was also outstanding in so far that it gave equally good velocity distribution for the 0.0283, 0.0198 and 0.0099 cumecs flow.

(ii) Head recovery :

The maximum percentage head recovery was obtained with curve 6, although comparatively there was not much difference in head recovery amongst the five different transitions.

Theoretical water surface profile for curve 6 was again balanced better, (Fig.4.8).

The average Froude number in the third series had already become about 0.21 at the throat and 0.105 at the exit for the all three discharges mentioned above. It was thus not much of importance to continue the investigations for easier constrictions than 50%. It is apparent that for less severe fluming ratios, the side effects should necessarily become less pronounced. It seem probable that in the region of 25% constriction (0.69/0.91) perhaps all the five transitions, investigated in the second and third series, may give satisfactory results. Thereafter, perhaps even the straight line transition may be nearly as good as the rest.

Thus, the success of the hyperbolic expansion in the past can be explained as largely due to the fact that, besides being a fairly satisfactory type inherently, it had been mostly tried for the 50% and easier fluming ratios. In this region, it should evidently work well enough.

4.3. SUPERCRITICAL TRANSITION:

Several aspects of supercritical flow in transitions need further experimentation to yield more refined guidelinoss for design.

The change in the Froude number associated with a supercritical transition is unfortunately just the opposite of what would generally be required. For example, the Froude number increases after an expansion designed as in Chapter 3, whereas an expansion is generally provided where it is desirable to increase the cross-section and reduce the bed slope and Froude number. Inevitably, supercritical transitions designed solely on the basis of wave interaction will give rise to S_2 or S_3 surface profiles in the downstream channel. It remains to be checked whether a suitable bed transition can be incorporated with the wall transition (as normally done in subcritical transition) so as to eliminate the downstream surface profiles.

Another important aspect of the design of supercritical transitions is the ' Froude number sensitivity ' of the design. Although uniform flow in a steep channel shows only a limited variation of the Froude number for a reasonably wide range of depths, yet some variation in the Froude number may have to be taken care of in the design. The conventional methods of design have the drawback that they achieve wave cancellation and quiet flow only at the design Froude number. At other Froude numbers, the flow will exhibit standing waves and disturbances in the downstream channel.

Tursunov[40] has pointed out this drawback of supercritical transition involving only a wall transition. He suggests the use of a curved bed transition in conjunction with the wall transition so as to make the design less dependent on the Froude number of the flow. It is proposed to experiment on this type of transition with a view to arrive at the design showing theleast Froude number dependence.

The third aspect of the supercritical flow in a transition requiring advanced study is the effect of the channel roughness on the wave formation. It is well known that surface profile in a transition computed by the elementary method of characteristics do not agree well with the actual profiles(41). It is necessary therefore to make a critical comparison of the experimental surface profiles with the theoretical profiles obtained from a computer solution taking into account bed slope and channel friction.

Lastly, with regard to shock waves themselves, the actual inclinations for shock-waves in water are found to differ from the theoretical wave inclinations based on the shallow water gravity wave theory. It may be that the discrepancy is the result of the action of the boundary layer and fluid viscosity, the effect of vertical accelerations or air-entrainment of the flow in the wake of the shock wave.

4.4. CONCLUSIONS:

(1) The recommended inlet (Fig.4.1) is scour-free at the entrance, has a low head loss, and produces a smooth downstream water surface. In addition, it has the advantage of simpler hydraulic design, simpler structural design, simpler construction, and much shorter length than the Hinds' S-curve transition, Trial and error is not required in the hydraulic design, as identical design factors are used for all transitions.

(8) For minimum head loss and no separation of flow, the optimum length of expansion transition is governed by the side splay varying from 7 in 1 to 9 in 1. But from the practical considerations, e.g., cost of construction, splaying longer than 5 in 1 or at best 6 in 1 may not be profitable. A little sacrifice in efficiency may be helpful in overall economy of construction.

(9) Providing too long expansion transitions is quite costly. Transitions of shorter length are, therefore, preferred. Different separation control devices e.g. splitter vanes, bed deflector sill, baffles etc. can be used for reducing the separation of flow and minimising the head loss in expansion transitions. All these devices are associated with increased head loss and their use is predidcated on the permissible head loss.

(10) The best geometrical shape for subcritical expansion transitions is given by the formula[12].

$$X = \frac{M}{y^{3/2}} + N$$

or $$X = \frac{L\, b^{3/2}}{b^{3/2} - a^{3/2}} \left[1 - (a/y)^{3/2} \right]$$

gives the transition curve which obtains both the optimum flow conditions and the maximum percentage recovery of head, under all conditions of subcritical expansion flow.

(11) An indirect confirmation of the superiority of the above curve is obtained from the fact that the most balanced theoretical water surface profile a statistically justified Hinds criterion is automatically obtained for the different fluming ratios.

(12) The provision of short reverse curves on either side near the exit makes absolutely no impression on the general flow conditions. Due to the reduced velocities, their effect is extremely local and as such their necessity is not at all obvious.

(13) Outlet losses can be computed as mentioned in section 2.7.2. In case of syphon outlet it may be assumed as $0.2 \Delta h_v$.

(14) The design procedure for supercritical expansions and the graphical construction of the resulting wave pattern have been presented in an improved form which obviates the need for a trial and error solution. New design charts have been presented for the design of supercritical contractions. The limitations of the gas dynamics analogy in the design of supercritical transitions have been stressed, and the need for further experimentation pointed out.

(15) There is still scope for further study and research on this topic. As a matter of fact an experiment are now under way in the hydraulic engineering laboratory of the Indian Institute of Technology, Madras (India), on many aspects of the design of channel transition for supercritical flow, with a view to suggesting better guidelines for design[42].

The absence of a warped wall greatly simplifies structural design and construction.

(2) An adequate allowance for inlet head loss is given by

$$H_L = 0.06\ (1 - B_f / B_a)\ V_2^2/2g$$

(3) Performance of the theoretical designs for the subcritical expansion transitions, based on one-dimensional approach, can not be considered much dependable, more so when the constriction is 50% or greater.

(4) The hyperbolic expansions, now in common use on canal structures in different parts of India, can only be relied upon when the constriction is easier than 50% and not otherwise.

(5) There is little difference in the flow conditions due to change in B/d ratio with normal depths when the discharge falls to nearly two-third. In the region below 50% discharge, the flow conditions generally deteriorate for the fluming ratios more severe than 50%.

(6) Flow conditions at the entrance are very material. Any instability upstream gets still more aggravated downstream. Conversely, the flow downstream is much more stable if uniform hydrostatic flow is obtained by a long throat or a fairly long constricting upstream transition. Thus, greater constriction is possible for long aqueducts than ordinary short throated bridges, viz., the efficient expansion flow downstream.

(7) With the satisfactory performance of the transition 6 upto 75% constriction (0.23 /0.91), it can be taken as the limiting fluming ratio for canal structures, which can be also economically feasible.

(8) For minimum head loss and no separation of flow, the optimum length of expansion transition is governed by the side splay varying from 7 in 1 to 9 in 1. But from the practical considerations, e.g., cost of construction, splaying longer than 5 in 1 or at best 6 in 1 may not be profitable. A little sacrifice in efficiency may be helpful in overall economy of construction.

(9) Providing too long expansion transitions is quite costly. Transitions of shorter length are, therefore, preferred. Different separation control devices e.g. splitter vanes, bed deflector sill, baffles etc. can be used for reducing the separation of flow and minimising the head loss in expansion transitions. All these devices are associated with increased head loss and their use is predidcated on the permissible head loss.

(10) The best geometrical shape for subcritical expansion transitions is given by the formula(12).

$$X = \frac{M}{y^{3/2}} + N$$

or
$$X = \frac{L\, b^{3/2}}{b^{3/2} - a^{3/2}} \left[1 - (a/y)^{3/2} \right]$$

gives the transition curve which obtains both the optimum flow conditions and the maximum percentage recovery of head, under all conditions of subcritical expansion flow.

(11) An indirect confirmation of the superiority of the above curve is obtained from the fact that the most balanced theoretical water surface profile a statistically justified Hinds criterion is automatically obtained for the different fluming ratios.

(12) The provision of short reverse curves on either side near the exit makes absolutely no impression on the general flow conditions. Due to the reduced velocities, their effect is extremely local and as such their necessity is not at all obvious.

(13) Outlet losses can be computed as mentioned in section 2.7.2. In case of syphon outlet it may be assumed as $0.2 \Delta h_V$.

(14) The design procedure for supercritical expansions and the graphical construction of the resulting wave pattern have been presented in an improved form which obviates the need for a trial and error solution. New design charts have been presented for the design of supercritical contractions. The limitations of the gas dynamics analogy in the design of supercritical transitions have been stressed, and the need for further experimentation pointed out.

(15) There is still scope for further study and research on this topic. As a matter of fact an experiment are now under way in the hydraulic engineering laboratory of the Indian Institute of Technology, Madras (India), on many aspects of the design of channel transition for supercritical flow, with a view to suggesting better guidelines for design[42].

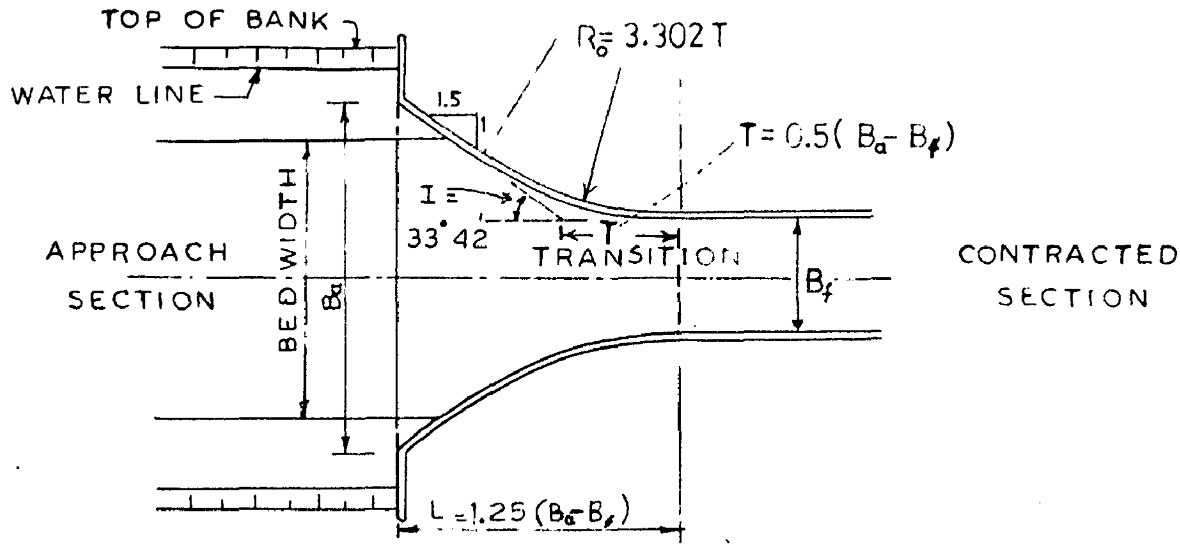


FIG. 4.1 DEFINITION SKETCH FOR PROPOSED INLET

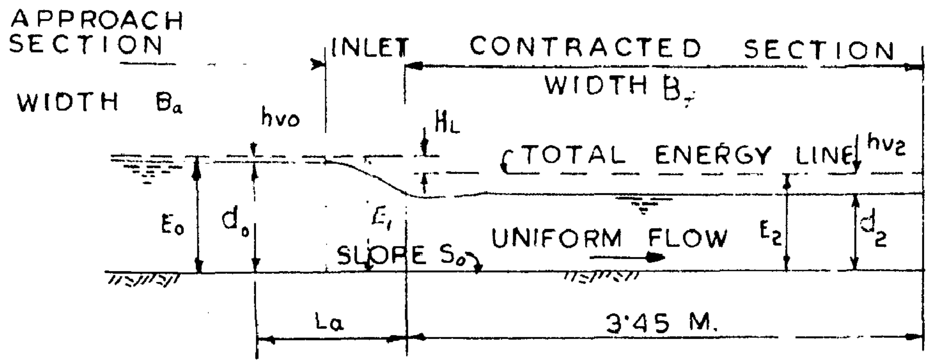


FIG. 4.4 DEFINITION SKETCH FOR HEAD LOSS AT AN OPEN CHANNEL INLET TRANSITION

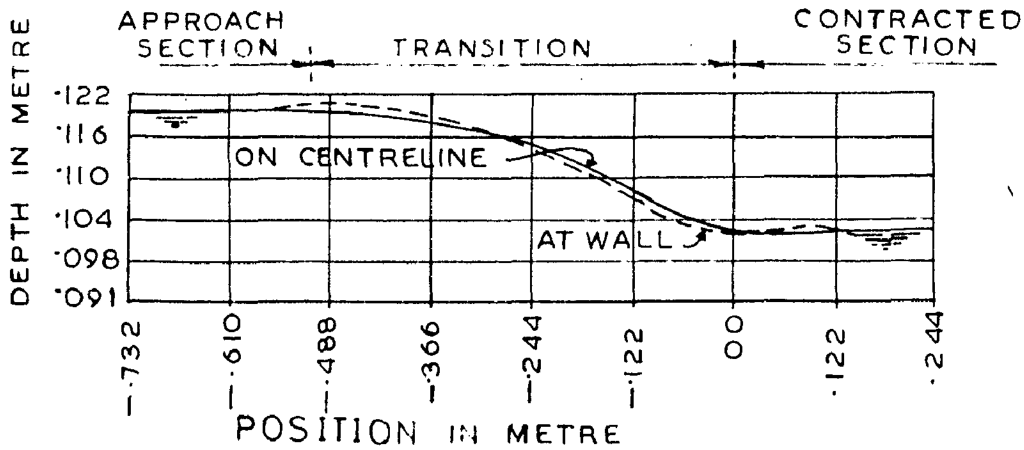


FIG. 4.5 TRANSITION WATER SURFACE PROFILE B_a/B_f=3·0 AND F_2= 0·6

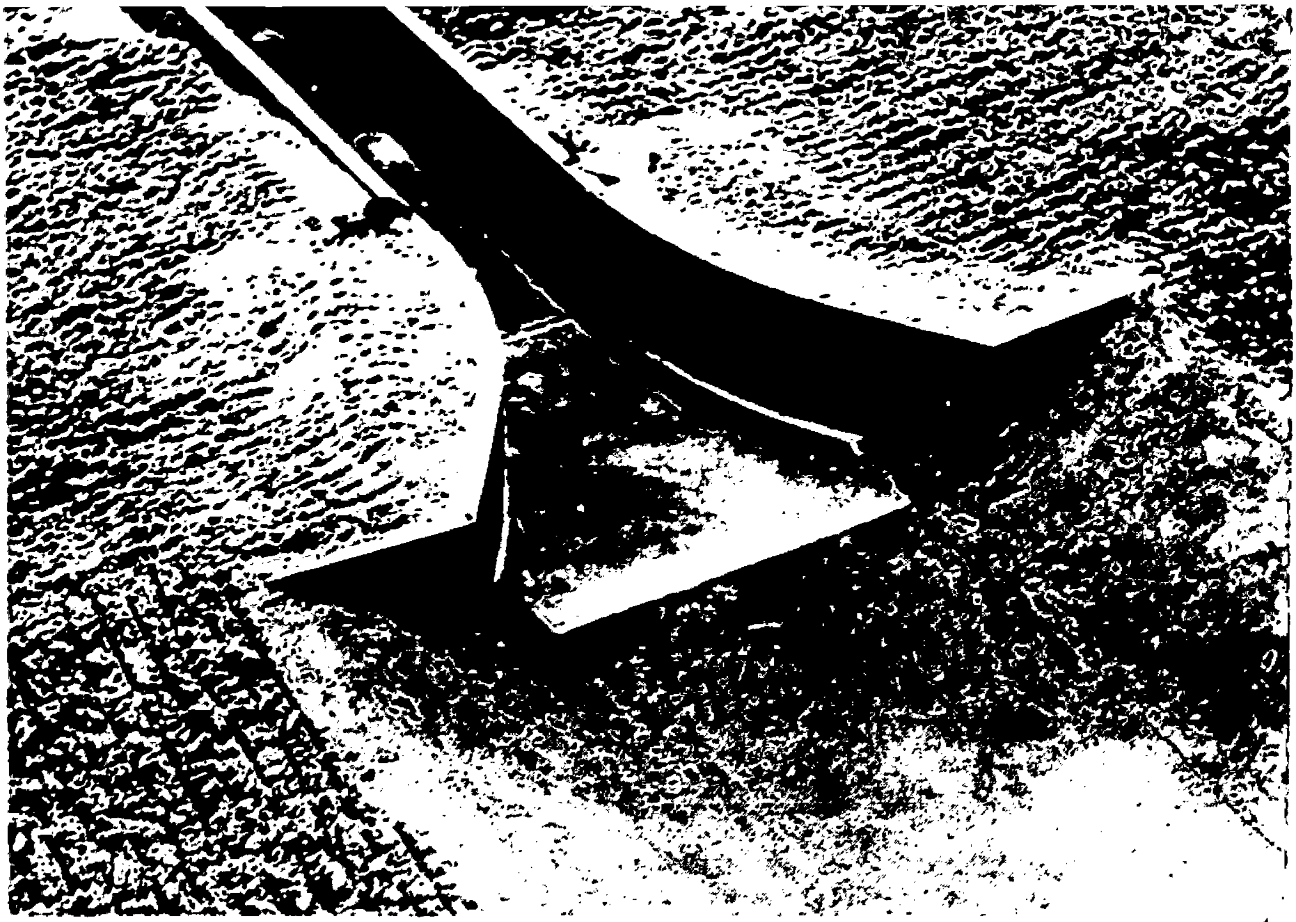


FIG. 4.2 –MODEL OF RECOMMENDED INLET, $B_a/B_f = 3.0$

FIG. 4.3 –FLOW CONDITIONS IN THE INLET, F_2 = 0.6 IN THE FLUME

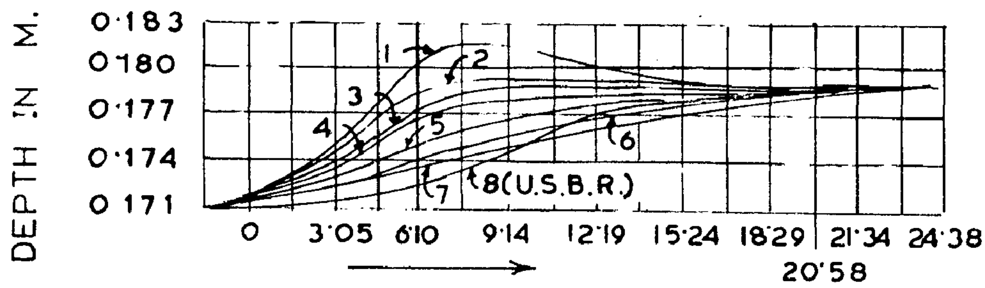


FIGURE 4.6

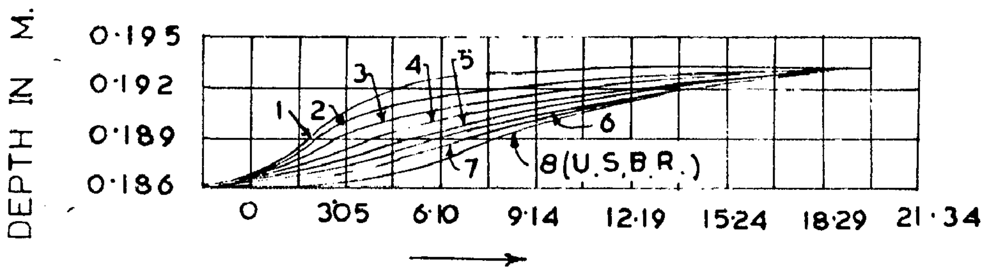


FIGURE 4.7

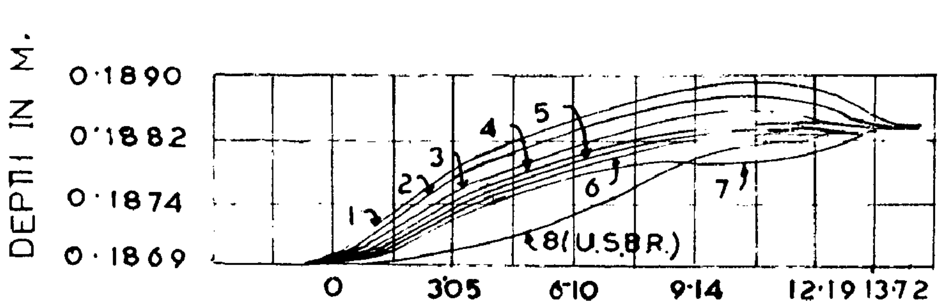


FIGURE 4.8

POSITION IN M. FROM THROAT OF FLUME (FIGS. 4.6-4.7-4.8)

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