HYDRAULIC DESIGN OF CROSS DRAINAGE WORKS –AQUEDUCT & SYPHON AQUEDUCT

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CERTIFICATE

This is to certify that the work entitled, **"HYDRAULIC DESIGN OF CROSS DRAINAGE WORKS-AQUEDUCT AND SYPHON AQUEDUCT"** submitted by **CHANDRESH KUMAR AND MOUSHUMI SAMAJDAR,** in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

23.5.08

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Certified the above mentioned project work has been carried out by the said group of students.

23.5.08

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ABSTRACT

In this project, the analysis and design of a Cross Drainage Work employed at the crossing of a canal and a natural drain, so as to dispose water without interrupting the continuous canal supplies. These hydraulic cross drainage works are structures which built at the crossing of a canal and a natural drain, so that drainage water may be disposed off without interrupting the non intermittent canal supplies. There are four types of cross drainage works namely, aqueduct, syphon aqueduct, super passage and the canal syphon. These structures are created to provide the best economic alternative for two crossing water masses. These structures are sometimes flumed in order to provide the best waterway path which is also economic. At various points in the flumed width, as water passes head loss occurs. This head loss has been calculated by Unwin's formula. The flumed section transition design has been designed by Mitra's method of transition design.

One of the main objectives of the present study to get the clear understanding of the various steps required in the hydraulic design of the aqueduct and siphon aqueduct was satisfactorily learnt in the process of designing these cross drainage works.

In this project a computer programme has been developed to find out the hydraulic design parameters of aqueduct and syphon aqueducts. The computer programme thus developed makes manual designing of these hydraulic structures easy enabling engineers to forego cumbersome and time consuming calculations. The programme satisfactorily designs the following with user defined inputs.

ACKNOWLEDGEMENTS

The success of any project depends largely on the encouragement and guidelines of many others. So we take this opportunity to express our sincere gratitude to the people who have been instrumental in the successful completion of the project.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

$P =$ Wetted Perimeter

$Q =$ Total Discharge of Canal

$Q_d =$ Discharge of Drain

$L =$ Length of Barrel

$R =$ Radius of Barrel

$V_a =$ Velocity of Approach

$V =$ Velocity of Flow

$S =$ Scour Depth Deasured Below the H.F.L

$B_n =$ Bed Width of Normal Channel Section

$B_f =$ Bed Width of Flumed Channel Section

$B_x =$ Bed Width at Any Distance x From the Flumed Section

$L_f =$ Length of Transition

$Hs =$ Total Seepage Head

$h_1 =$ Loss of Head at U/S Entry at the Contraction Transition

$$= K_1\left[\frac{\left(V_1^2 - V_2^2\right)}{2g}\right]$$

$h_2 =$ Loss of Head Due to Friction in Contraction Transition.

$h_3 =$ Loss of Head Due to Friction in Trough.

$h_4 =$ Loss of Head Due to Friction in Expansion Transition.

$h_5 =$ Loss of Head at Exit $= K_2\left[\frac{\left(V_3^2 - V_4^2\right)}{2g}\right]$

Abbreviations

(1) HFL – High Flood Level

(2) FSL – Full Supply Level

(3) IS- Indian Standard

(4) ASCE – American Society Of Civil Engineering

(5) U/S – Upstream Side

(6) D/S – Downstream Side

(7) RL – Reduced Level

(8) RCC – Reinforced Cement Concrete

(9) R/C – Reinforcement

(10) HGL – Hydraulic Gradient Line

(11) CBL – Canal Bed Level

(13) DBL – Drain Bed Level

(14) C/C – Center To Center

(15) GVF – Gradually Varied Flow

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Chapter 1
INTRODUCTION

1.1 GENERAL INTRODUCTION

Canals are artificial channels used for carrying water. There are two types of canals: irrigation canals; which are used for the delivery of water and waterway, which are navigable transportation canals used for passage of goods and people, often connected to existing lakes, rivers, or ocean.

A river is a natural drain of water, usually fresh water, flowing toward the ocean, a lake, or another stream. In some cases a river flows into the ground or dries up completely before reaching another body of water. Usually larger streams are called rivers. A river is a component of the water cycle. The water within a river is generally collected from precipitation through surface water, groundwater recharge (as seen at baseflow conditions / during periods of lack of precipitation) and release of stored water in natural reservoirs, such as a glacier.

Cross-drainage works are required when canal and river cross each other in order to make suitable and easy passage of flow of water of canal and river. A cross drainage work is a structure which is constructed at the crossing of a canal and a natural drain, so as to dispose off drainage water without interrupting the continuous canal supplies. Sometimes due to alignment of river and canal such conditions arises that it becomes unavoidable to cross these two. In these circumstances cross drainage works are provided. In order to reduce the cross drainage works, the artificial canals are generally aligned along the ridge line called the Water Shed. Once the canal reaches the Water Shed line, cross drainage works are generally not required, unless the canal alignment is deviated from the water shed line. However, before the water shed is reached, the canal which takes off from the river has to cross a number of drains. At all such crossings cross drainage works are required.

1.2 TYPES OF CROSS DRAINAGE WORKS

The drainage water intercepting the canal can be disposed off on in either of the following ways:

I. By passing the canal over the drainage. This may be accomplished either through (A) An Aqueduct, or through (B) A Syphon Aqueduct.

II. By passing the canal below the drainage. This may be accomplished either through (A) a Super- Passage, or through (B) Canal Syphon generally called a Syphon.

III. By passing the drain through the canal, so that the canal water and drainage water are allowed to intermingle with each other. This may be accomplished through (A) a Level- Crossing or through (B) Inlets and Outlets.

The figures of the above structures are depicted in Figs 1.1- 1.5.

Aqueduct is a bridge like structure wherein canal passes over the river or stream. Both the flows are at atmospheric pressure only.

In Syphon Aqueduct, the river or stream is siphoned below the canal flow and during this passage; its flow is a pressure flow.

Super passage is also similar to an aqueduct. However, in this case canal is below the drainage, there is sufficient free board between the FSL (full supply level) of canal and the underside of drainage trough. A super passage is provided when the drainage is small and is at high level.

A canal syphon (also called irrigation siphon) is a type of cross-drainage work having a closed conduit running full under pressure to carry canal water under the drainage channel (or the canal, or road or a railway line).

If the discharge in the canal is small, precast R.C.C. (reinforced concrete cement) pipes may be used.

1.3 SELECTION OF SUITABLE TYPE OF CROSS DRAINAGE WORK

Selection suitable type of cross-drainage works depend upon the following factors.

(1) Suitable canal alignment.

(2) Permissible head loss in canal.

(3) Nature of available foundation.

(4) Position of water table and availability of dewatering equipment.

(5) Suitability of soil for embankment.

(6) Availability of funds.

Many type of cross drainage works is used in a canal to drain the water. Like aqueduct, syphon aqueduct, super passage, canal siphon, level crossing and inlet outlets are used in cross drainage of canal.

Compared to an aqueduct, a super passage is inferior and should be avoided whenever possible. Similarly, a syphon aqueduct (unless large drop in drainage bed is required) is superior to siphon. A level crossing may inevitable in certain cases. For example, when a large canal crosses a large torrent at almost equal bed levels, a level crossing may remain to be only answer.

1.4 OBJECTIVES OF PRESENT STUDY

The present study is aimed to get clear understanding of hydraulic design of aqueduct and syphon aqueduct. Computer aided programme makes easy and accurate way of design of any structure and save the time incurred in the manual calculations. Thus it was aimed to develop a computer programme for hydraulic design of aqueduct and syphon aqueduct of the same using the computer language C or C++.

1.5 LIMITATION OF THE STUDY

In the present study the hydraulic design of aqueduct and siphon aqueduct is being studied and a computer programme is being developed for the same. The hydraulic design of the other types of cross drainage works (For example; canal siphon, super passage etc.) are beyond the scope of the present study.

The structural design of the aqueduct and syphon aqueduct are also beyond the scope of the present study and so here has not been referred.

Chapter 2

AQUEDUCT AND SYPHON AQUEDUCT

2.1 INTRODUCTION

The canal is taken over by the natural drain, such that the drainage water runs below the canal either freely or under siphoning pressure. When the H.F.L (high flood level) of the drain is sufficiently below the bottom of the canal, so that the drainage water flows freely under gravity, the structure is known as an Aqueduct. However if the H.F.L of the drain is higher than the canal bed and water passes through the aqueduct barrels under symphonic action, the structure is known as a Syphon Aqueduct.

The Aqueduct and Syphon Aqueduct drawings are provided at the end of the report.

In this type of works, the canal water is taken across the drainage in a trough supported on piers. An inspection road is generally provided along with trough, as shown in the drawings. An aqueduct is just like a bridge except that instead of carrying a road or a railway, it carries a canal on its top. An aqueduct is provided when sufficient level difference is between the canal and the natural drainage, and canal bed level is sufficiently higher than the torrent level.

They may be classified into three types depending upon the sides of the Aqueduct:

Type I

In this type, the sides of the aqueduct are earthen banks with complete earthen slopes. The length of the culvert through which the drainage water has to pass under the canal should not only be sufficient to accommodate the water section of the canal but also the earthen banks of the canal with adequate slopes.

Type II

In this type, the canal continues in its earthen section over the drainage, but the outer slopes of the canal banks are replaced by retaining walls, thereby, reducing the length of the drainage culvert by that much extent.

Type III

In this type, the earthen section of the canal is discontinued and the canal water is carried in masonry or a concrete trough. The canal is generally flumed in this case, so that it serves the best possible economy in construction.

The culvert length or width of the aqueduct is maximum in *Type I* and maximum in *Type III.* An intermediate value exists in *Type II.*

The selection of a particular type out f three types of aqueduct or syphon aqueducts lies on the consideration of economy. The cheapest of the three types at a particular place shall be the obvious choice.

2.2 SOME MAIN AQUEDUCTS AND SYPHON AQUEDUCTS IN INDIA

2.2.1 Bhima Aqueduct, Maharashtra

This aqueduct is an optimal solution for the aqueduct to carry canal water and a bridge deck for vehicular traffic over the river. Aqueducts suspended on hollow circular piers 40m high, constructed with climbing forms. The pictorial view is shown in Photo No. 2.1

Salient Features

Discharge = 42.5 cumecs

Length = 947 m

Width of Roadway = 3.5 m

Continuous Spans = 41.5

Circular Cross Section = 4.8 m Ø

Truncated circular cross section of 4.8m Ø is for high hydraulic and structural efficiency and transverse effects reduced by transverse pre-stressing. Rapid and economic construction achieved by the RMC(Ready mix concrte).

2.2.2 <u>Solani Aqueduct, Solani</u>

This aqueduct was completed in 1856 at Solani river, in Roorkee, India. It carries the Ganges canal. Its structural type is an arch bridge.The pictorial view is shown in Photo No. 2.2

Salient Features

Total Length = 338.55 m

Span Lengths = 15*15.25 m

Width of Canal = 52.46 m

The aqueduct is a structure of fifteen arches spanning a valley a thousand feet wide. The river below, the Solani, is an intermittent stream whose broad valley is usually dry as a bone. The valley is so shallow that the ground is never more than about 25 feet below the floor of the aqueduct.

2.3.3 <u>Narmada Main Canal</u>

Narmada Main Canal, which is a contour canal, is the biggest lined irrigation canal in the world. It is about 458 km long up to Gujarat -Rajasthan border having discharging capacity 1133 cumecs (40000 cumecs) at its head tapering to 71 cumecs (2500 cusecs) at the Gujarat -Rajasthan border. The canal extends further in Rajasthan to irrigate areas in Barmer and Jhalore districts of Rajasthan. The cross section of the canal at its head is 73.1m x 7.6m (Bed width x Full supply depth) with 2:1 inner side slope having canal velocity at head as 1.69 m/sec. The entire length of the Main Canal is proposed to be lined with in-situ plain cement concrete to minimize seepage losses, to allow higher velocities and control water logging problems in future. The lining work is under construction with mechanized pavers, which is being operated on such a large scale for the first time in India. In all, there are 593 Structures on the Narmada Main canal. Out of this 320 structures are cross drainage structures, comprising of 5 Aqueducts, 15 canal syphons, 177 drainage syphons, 26 canal crossing and one super passage. The pictorial view is shown in Photo No. 2.3

2.2.4 Mathur Aqueduct

Mathur Aqueduct or Mathur Hanging Trough is an Aqueduct in Southern India, in Kanyakumari District of Tamilnadu state. Built over the Pahralj River (also called *Parazhiyar*), it takes its name from Mathur, a hamlet near the Aqueduct, which is at a distance of about 3 kilometres from Thiruvattar town and about 60 km from Kanyakumari, the southernmost town of India. It is one of the longest and highest aqueducts in South Asia and is also a popular tourist spot in Kanyakumari District. The pictorial view is shown in Photo No. 2.4

Construction

Mathur Aqueduct is a concrete structure held up by 28 huge pillars, the maximum height of the pillars reaching 115 ft. The trough structure is 7 ft in height, with a width of 7.5 ft. The trough is partly covered on top with concrete slabs, allowing people to walk on the bridge and also see the water going through the trough. Some of the pillars are set in rocks of the Pahrali river, though some of the pillars are set in hills on either side.

Road access allows one to drive in to one side of the Aqueduct (up to one end), while it is also possible to drive into the foot of the Aqueduct (the level where the Pahrali flows) on the opposite side. There is also a huge flight of stairs (made in recent times) that allows one to climb from the level of the Pahrali River to the trough.

Salient Features

Length of flume	1240 ft (378 m)
Width	7.5 ft (2.3 m)
Height of trough	7 ft (2.1 m)
Velocity	5.1 ft/s (1.55 m/s)
Discharge	204 ft^3/s (5.8 m^3/s)

No. of span pillars	28
Length of span	40 ft (12.2 m)
Bed level, trough at start	91 in (2.31 m)
Bed level, trough at end	90 in (2.29 m)
Maximum height above ground level	115 ft (35 m)
Construction cost	Rs. 12.90 lakhs (Rs. 1,290,000, US$27,446.80) - in 1966

2.2.5 Gomti Aqueduct

A dream of the Engineers of Uttar Pradesh Irrigation Deptt reached the stage of realization with the commissioning of the voluminous and gigantic work of Gomti Aqueduct on republic Day, 26th January, 1978. The water of river Ghagra and Sarda after crossing Gomti, through this aqueduct, is now available for irrigation in the districts of Raebareli, Pratapgarh, Allahabad, Varanasi, Jaunpur etc. lying in Gomti –Sai and Sai-Ganga doabs. The completion of this phase of the Sarda Sahayak Feeder Channel, one of the world's biggest system, has created total irrigation potential of 5 lacs hectares in the region. The pictorial view is shown in Photo Nos. 2.5 (a,b).

Salient Features

Discharge during floods	= 4530 cumecs
High flood level	= R.L 109.5

For Main Aqueduct Portion:

Hydraulic Discharge through Aqueduct	= 357 cumecs
Velocity of Water through Aqueduct	= 4.16 m/s
Maximum Depth of water in R.C.C Trough	= 6.7 m

Free Board above Full Supply Level	=	0.75 m
Length of Aqueduct	=	382 m
Length of Span	=	31.8 m c/c
No. of Spans	=	12
Size of Foundation Well	=	12 m * 27 m (Double D type)
Design Load on Bearing	=	1300 T
Depth of Pre-stressed Concrete girder	=	9.9 m
Internal Dimension of R.C.C Trough	=	12.8 m* 67 m clear waterway
Clear Road Way (At Top of Girder)	=	4.25 m

Flumed Approaches:

Length of Transition on U/S of Aqueduct	=	37 m
Length of Transition on D/S of Aqueduct	=	55 m
No. of Foundations Wells on U/S	=	3
No. of Foundations Wells on D/S	=	4
Size of Foundation Wells	=	4 m * 26 m
Depth of Foundation	=	30 m

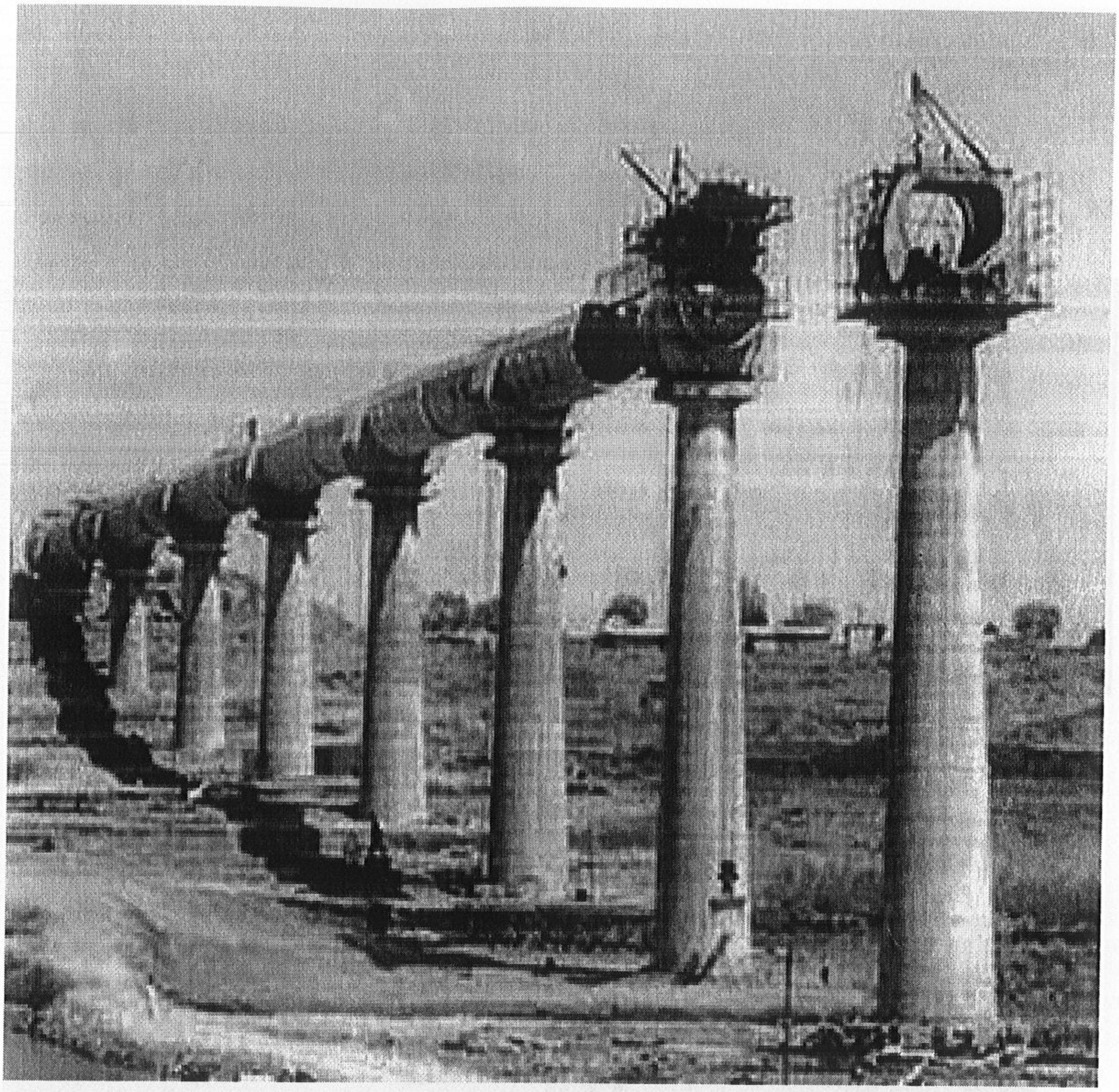


PHOTO 2.1: BHIMA AQUEDUCT

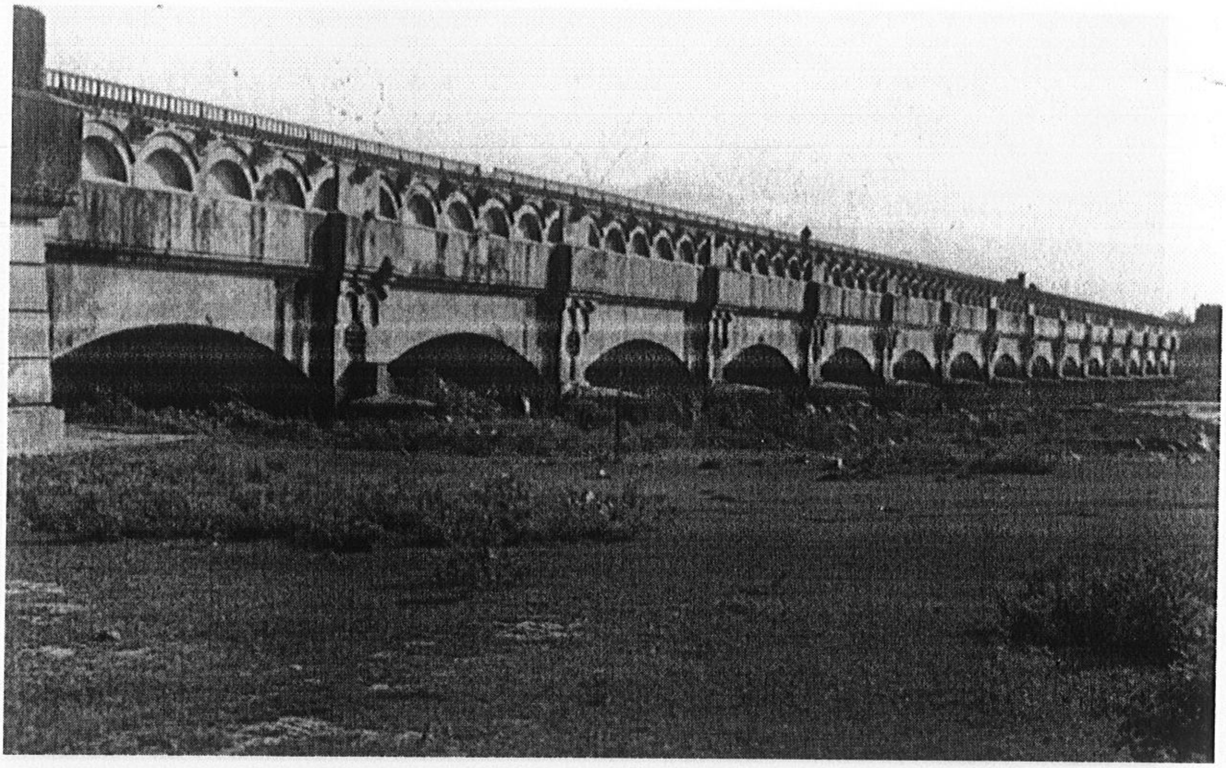


PHOTO-2.2: SOLANI AQUEDUCT

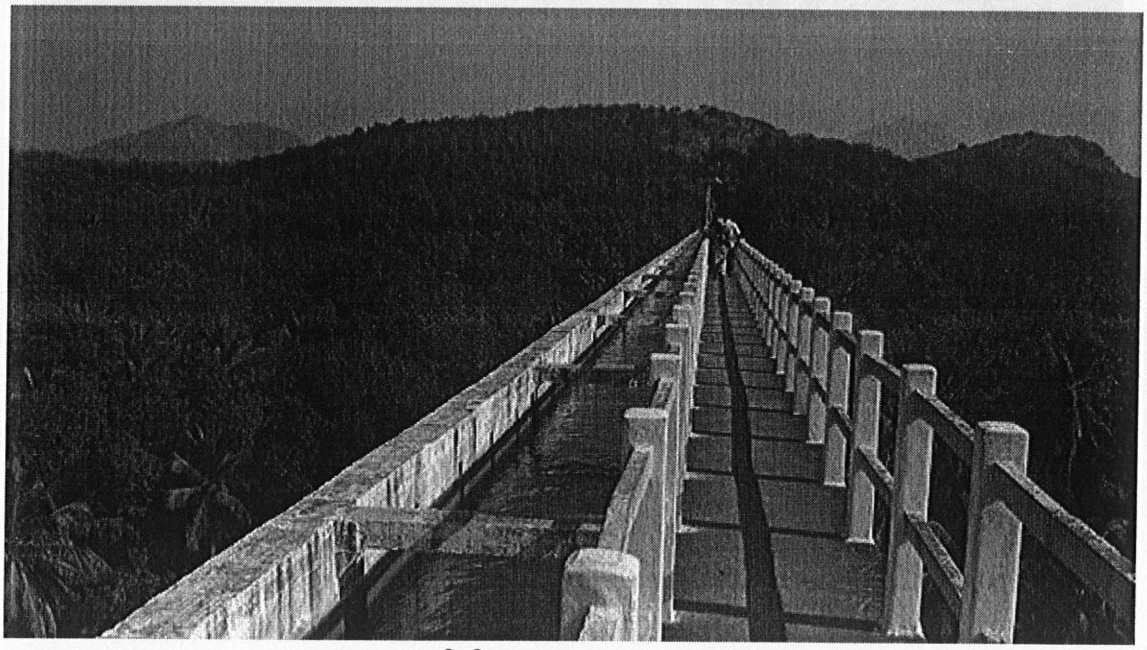


PHOTO -2.3: MATHUR AQUEDUCT

PHOTO -2.4: MATHUR AQUEDUCT (SIDE VIEW)

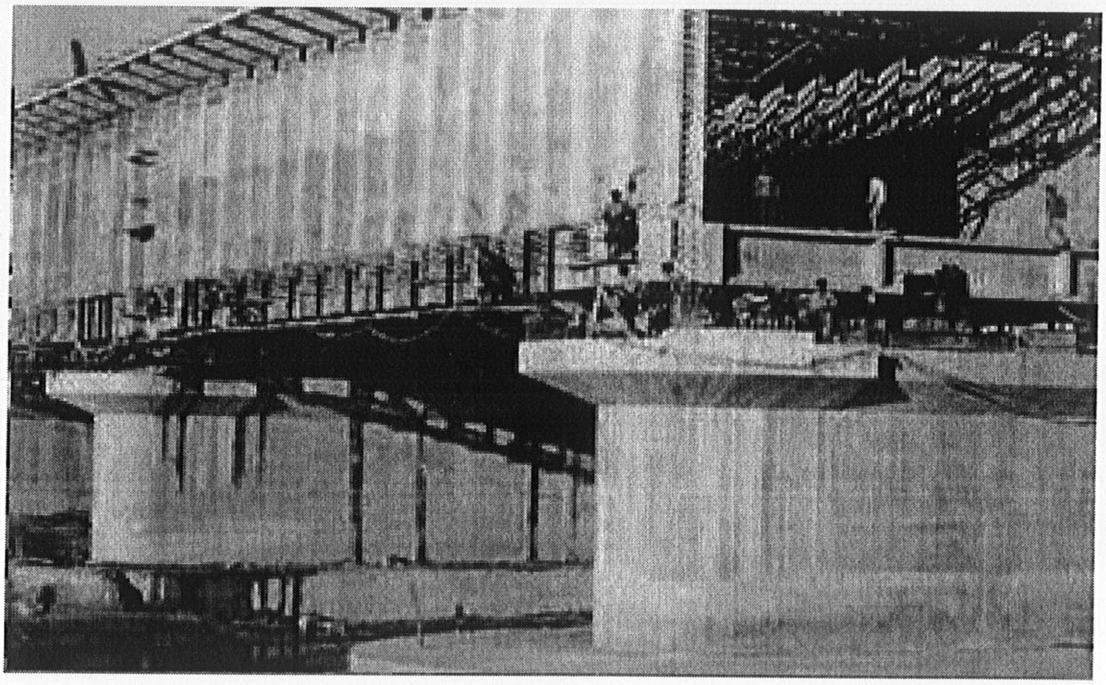


PHOTO- 2.5 (a): GOMTI AQUEDUCT

PHOTO-2.5 (b): GOMTI AQUEDUCT

Chapter 3

DESIGN OF AQUEDUCT AND SYPHON AQUEDUCT

3.1 SITE INVESTIGATION AND FIELD DATA REQUIRED

On the basis of preliminary investigations done, detailed investigations prior submission of the final report is carried as per the following:

I) Topographical Survey

Topographical surveys for these hydraulic structures should be first carried out and discussed. The survey may be conducted with the help of Survey of India which provides the topographical map of that region.

II) Meteorological and Hydrological Investigations

The meteorological and hydrological investigations are required in order to estimate the water availability and magnitude of design flood. The magnitude of flood and its level from the bed is essential to be known for the designing of the structure. Further the characteristic of the soil (such as type of soil) of the catchments area should also be known as it decides the size of the sediment particle and quantity of the sediment particle that may come during the flood time and during the rainy season in the canal and in the river. The sediment reaching at the river and canal will change the characteristics of the flowing water drastically. The sand and gravel comprising the bed of most rivers and canals fall under the general definition of sediment and one of the most importanat practical problems in hydraulic engineering is the computation of sediment transport rates under different sediment, fluid and flow conditions. It is well known that sediment transport behavior in the water is complex phenomenon.

The sediment characteristics within the catchment area should be carefully investigated for at least 3 years.

Various factors like sites of sediment observations, location of Pan Evaporation stations, location of rain gauges station in and around the catchment's area, etc. should be known.

In the hydrological investigations the following should be collected and discussed:

a) Rain gauge discharge observations for at least ten years.
b) Chemical and biological analysis of river water should be examined and maintained.
c) Gauges and discharge measurement during the flood occurrence should be observed at short interval.

The above required data should be completed.

III) Ecological Survey

The ecology of the region is affected by construction and operation of major irrigation and hydrological project. So it is necessary to make survey and investigation of the wild life habitat, fish culture and historical and cultural repercussions. The large hydraulic structure alters the natural flow patterns of the river. Such types of alterations and protection works, if any to retain the structure is needed and should be mentioned in the final report.

IV) Geological Survey

The geologic investigation for these hydraulic structure and site are done in order to check the suitability of foundation for the foundation works of the structure. The characteristics of the soil such as bearing capacity, permeability, consolidation and cohesion property of the soil etc. are the important parameters which should be determined before the construction of the hydraulic structure.

The seismic condition of the region is another important aspect; which should be investigated with reference to the geological map of the area. Evaluation of the seismic status of faults and thrusts and collection of seismological data both in preconstruction as well as post construction stage of the project area is of vital importance because of safety reason.

3.2 DESIGN PARAMETERS FOR THE CROSS DRAINAGE WORK

Determination of Final Flood Discharge

The high flood discharge for smaller drains may be worked out by empirical formula. For large drain we use methods such as hydrograph analysis, ration formula, etc.

Fixing The Waterway Requirement's For Aqueduct And Syphon Aqueduct

Approximate value of required waterway for the drain may be obtained by using Lacey's equation given by.

$$P = 4.75 * (Q_d)^{1/2} \tag{3.1}$$

P = is the wetted perimeter in metres

Q_d = total discharge in cumecs.

Afflux And Head Loss Through Syphon Barrels

The Head loss (h) through syphon and velocity through them related by **Unwin's** formula gives as:

$$h = \left[1 + f_1 + f_2 * \frac{L}{R}\right] * \frac{v^2}{2g} - \frac{V_a}{2g} \tag{3.2}$$

where, L = length of barrel

R = Radius of barrel

V_a = velocity of approach

V = velocity of flow

Fluming of Canal

Three method may be used to design the channel transition;

1) Mitra's method
2) Chaturvedi's method and
3) Hind's method

Design of Pucca Canal Trough: - Pucca canal trough may design by various methods

3.3 TYPE OF FOUNDATION USED FOR CROSS DRAINAGE WORKS

The cross drainage works are somewhat like roads and railway bridges. The load acting on these works is mainly due to the weight of water. However, some portion of the aqueducts and syphons aqueduct is generally used as a road bridge which should be designed for the load specified for the bridges.

The choice of type of superstructure depends upon the most economical arrangement possible at a given site. A common criterion used for the determination of the most economical span is to keep the cost of superstructure equal to that of the substructure. However, while applying this criteion, the foundation conditions should also be considered. If the cost of individual piers and their foundations is high, the minimum number of piers should be provided by increasing the span and vice-versa. The cost of substructure depends upon the type of foundation and the height of piers.

Scour Depth

The type of foundation usually depends upon the depth of scour. If the scour is small, shallow foundations may be adopted. However, if the scour is excessive, deep foundations are required. Generally well foundations are used as deep foundations.

The natural scour depth in alluvial streams is usually determined by **Lacey's Formula,**

$$S = 0.47 * \left(\frac{Q}{f}\right)^{1/2} \tag{3.3}$$

Where S is the scour depth measured below the H.F.L., Q is the discharge and f is the silt factor.

The depth S', of scour around piers of the aqueduct, may be taken as 2 S. (below H. F. L)

For well foundations, the depth of foundation below the maximum scour level should not be less than *S/3* or Rankine's minimum depth, with a minimum of 1m in order to provide adequate ***grip length***.

Shallow foundations may be provided instead of well foundations if the drainage bed is protected against scour by a suitable impervious floor, accompanied by suitable cutoff walls and protection works, on the U/Sand D/S of the floor.

To take into account the excess scour due to restricted waterway and concentration of flood in the span, the actual scour is usually taken as follows:

In a straight reach = 1.25 *S*   At moderate bend = 1.50 *S*

At a severe bend = 1.75 *S*   At a right angle bend or at noses of piers = 2 *S*

In a severe swirls = 2.5 *S*

The type of foundation is usually selected depending upon the depth of scour and the bearing capacity of a soil.

If the depth of scour is small and there is no dewatering problem, open foundations (shallow foundations) maybe used 6 m depth below the drainage bed. However, if the depth of scour is large, deep foundations, such as a well foundation, may be more economical. A deep foundation may also be required even when the depth of scour is small if the bearing capacity of the soil is low and the loads are heavy.

If well foundations are difficult to construct or uneconomical, the bed is protected against scour by providing an impervious floor in conjunction with suitable cutoff walls and protection works at both ends of the impervious floor.

3.4 BANK CONNECTIONS

The bank connections consist of masonry wing walls of the canal and the drainage. These are required to connect the regular section of the canal and the drainage to the modified section at the cross-drainage site.

3.4.1) <u>Canal Wing Walls</u>- the canal wing walls are provided on the U/s side and the D/s side of the aqueduct to guide canal flow and to retain the earth of the canal banks on both sides of the canal trough. The foundation of the canal wing walls should be kept on the sound natural ground. It should not be left on the made up formation. The faces of wing walls are warped from the natural section of the canal (usually, 1:5:1) to the vertical at the trough.

3.4.2) <u>Drainage Wing Walls</u>- The wing walls of drainages are provided on the U/s and D/s of the barrel/culvert to guide the drainage flow ant to retain the natural banks of the drainage. The wing walls should be taken sufficiently deep into the guide banks. The wings should be shaped such that it provides smooth entry and exit at the drainage barrel.

3.5 FLUMING OF CANAL AND ITS TRANSITION

The canal is sometimes flumed to reduce the length of barrels/culverts in an aqueduct or a syphon aqueduct. Suitable canal transitions are provided on the U/s and D/s of the flumed section.

Basically a flume means a narrow gorge through which a stream passes. A channel transition is a gradual change in the cross section of the channel that produces a change of flow from one uniform state to another. This change of flow occurs over the length of transition. A transition avoids excessive loss of energy, eliminates cross-currents and turbulence and thus provides safety to the structure. In a transition, varied flow/non uniform flow occur, and the accelerating or decelerating forces are more predominant than the frictional resistance. Besides aqueducts and syphons aqueducts, fluming of canal is also sometimes done at super passages and canal syphons, falls, regulators, and bridges to reduce the cost of the structure. In all these cases, the transitions are provided to minimize the head loss and to reduce maintenance cost.

The transitions may be classified into two steps:

3.5.1) Contraction Transition

In contraction transition, the cross sectional area is gradually reduced. The design of contraction transition is easy, unless the velocity is very high or the contraction too severe, any suitable streamlined shape like a bell-mouth or a cylinder quadrant of transition can be provided as a contraction transition. The flow through a contraction transition is always stable because of favorable pressure gradients. The length of transition depends upon the degree of contraction adopted, the approach condition, the type of structure and permissible head loss. An average splay of 2:1 is usually provided. However, for important high velocity flumes, an average splay of 3:1 or 4:1 will give better flow conditions. On the other hand, for low hand velocity, the splay may be even 1:1. The transition should be tangential to the walls of the throat of flume where the velocity is high.

3.5.2) Expansion Transition

In expansion transitions, the cross sectional area is gradually increased. In this case, decelerating force tends to increase at the boundary layer effect and the losses are more. The pressure gradient is positive and hence unfavorable. The boundary flow is unsteady and may result in flow separation from the boundary and leads to turbulence and eddies. High the intensity of shear at the separation surface produces appreciable circulation and rollers may be formed. These rollers are the main sources of head loss associated with such flows, since the main flow has to infuse the energy to sustain them. So expansion transitions should effect the change more gradually as compared to that in contraction transitions.

Providing very long transitions are very costly. A splay of usually 3:1 is adopted. Sometimes a splay of 4:1 and 5:1 may also be adopted as in cases of high velocity flumes. In addition, separation control devices, such as splitter vanes, bed deflectors, sills, baffles, etc. are also sometimes used for minimizing separation flow

3.6 DESIGN STEPS FOR CROSS DRAINAGE WORKS

The following steps may be involved in the design of an aqueduct or a syphon aqueduct, design of super passage and a syphon done on same lines as for aqueduct and syphon aqueducts respectively.

3.6.1) <u>Determination of Maximum Flood Drainage</u>

The high flood discharge for smaller drains may be worked out by using empirical formulas; and for large drain other methods such as hydrograph analysis, rational formulas.

3.6.2) <u>Fixing The Waterway Requirements For Aqueduct And Syphon Aqueduct</u>

An approximate value of required waterway for the drain may be obtained by using Lacey's equation (Eq. 3.1).

For the wide drains the wetted perimeter may be approximately taken equal to the width of drain and hence and equal to the water way required.

For smaller drains a smaller figure for the waterway than that given by Lacey's regime perimeter may be chosen. The maximum reduction in waterway from lacey perimeter is 20%. Hence for smaller range (Minimum value=0.8*P*).

Size of barrels, after having fixed the water width and the number of compartments (bays), height of the drain barrel has to be fixed. In the case of an aqueduct the canal trough is carried clear above the drain HFL. And the drain bed is not to be depressed, hence the height of bay openings is automatically fixed in aqueducts as equal to the difference between HFL and DBL of the drain.

The velocity through the barrel is generally limited to 2 to 3 metre per second.

3.6.3 <u>Afflux And Head Loss Through Syphon Barrels</u>

Velocity through siphon barrel is limited to a scouring value of about 2 to 3 meter per second. A higher velocity may cause quick abrasion of the barrel surface by rolling grit etc. and shell definitely results in a higher amount of afflux on the upstream side of syphon or syphon-aqueduct.

The head loss through the siphon barrel and velocity(V) through them are generally related by Unwin's formula (Eq.3.2).

$$h = \left[1 + f_1 + f_2 * \frac{L}{R}\right] * \frac{v^2}{2g} - \frac{V_a}{2g} \tag{3.2}$$

Where L= length of the barrel

R = Hydraulic mean radius of barrel

V = velocity of flow through barrel

Va = ken equal to the velocity of approach and is often neglected

f_1 = Coefficient of head loss at entry

=0.505 for unshaped mouth and 0.08 for Bell Mouth.

f_2 = is a coefficient such that the loss of head through the barrel due to surface

where f_2 is given as f_2=a $(1 + b/R)$

Where the values of a and b for different material may be taken as given in table 3.1

Material of the surface of Barrel	*A*	*b*
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Brick work	0.00401	0.070
Rubble masonry	0.00507	0.205
Steel pipe	0.00497	0.025

Total head loss consists of three losses i.e.-

Entry loss = $f_1 * \frac{V^2}{2g}$, friction loss = $\frac{f_2 L V^2}{2gR}$, exit loss = $\frac{V^2}{2g}$

So a limit placed on afflux with limit the velocity through the barrel and vice –versa hence by permitting higher afflux and therefore a higher velocity through the barrel, the cross sectional area of siphon barrel can be reduced but there is a corresponding increase in the cost of guide banks and marginal bunds and also the length of downstream protection is increased. Hence an economic balance should be worked out and a compromise obtained between barrel area and afflux.

3.6.4) <u>Fluming Of Canal</u>

The contraction in the water way of canal (i.e. the fluming of canal) will reduce the length of barrel or the width. This is likely to produce economy in many cases. The fluming of canal is generally not done when the canal section is in earthen banks.

The maximum fluming is generally governed by extent that the velocity in the trough should remain sub critical (of order of three meter per second). Because if super critical velocity is generated then the transition back to the normal section on the downstream of work may involve possibility of formation of hydraulic jump. The extent of fluming is further governed by economy and permissible loss of head. The greater is the fluming, the greater is the length of the transition wings upstream as well as downstream.

After deciding the normal canal section and flumed canal section the transition has to be designed so as to provide a smooth change from one stage to the other so as to avoid sudden transition and formation of the eddies. For the reason upstream should not be steeper than 2:1 splay. And downstream should not be steeper than 3:1 splay. The normal earthen canal section is trapezoidal while the flumed pucca canal section is rectangular. It is necessary to keep the same depth in the normal and flumed sections.

The following methods may be used for the designing of channel transition:

- Mitra's method of design of transition
- Chaturvedi's method of design of transition
- Hind's method of design of transition

But in India the fluming of canals is done by only two methods only *i.e.* Mitra's method and Chaturvedi's method and hence these two methods are explained here only.

a) Mitra's Method

Shri A.C. Mitra has proposed a hyperbolic transition for the design of channel transition. The channel width at any section X-X at a distance x from the flumed section is given by:

$$B_x = \frac{B_n * B_f * L_f}{L_f B_n - \left(B_n - B_f\right) * x} \tag{3.4}$$

Where B_n = bed width of normal channel section

B_f = bed width of flumed channel section

B_x = bed width at any distance x from the flumed section

L_f = length of transition

b) *Chaturvedi's Semi Cubical Parabolic Transition*

Prof. R.S. Chaturvedi proposed the following Equation for the design of transition.

$$x = \frac{L * B_n^{3/2}}{\left(B_n^{3/2} - B_f^{3/2}\right) * \left[1 - \left\{\frac{B_f^{3/2}}{B_x^{3/2}}\right\}\right]} \tag{3.5}$$

3.6.4) Uplift Pressure on The Roof of Barrel

As the barrel of the siphon aqueduct runs full during floods an uplift pressure acts on the roof of barrel. The uplift pressure at any point of barrel can be obtained from hydraulic gradient line (HGL). The uplift pressure at any point is obviously equal to the ordinate between HGL and the roof of barrel at that point.

Because of any entry loss there is a sudden drop of HGL at the entrance. It is followed by gradual drop due to friction throughout the length of barrel and again there is a sudden drop at exit. The maximum uplift occurs just after the entry point at the upstream end of a barrel and is represented by U.

While designing trough there are two extreme conditions:

The canal trough is carrying full discharge but the barrel is empty.

The barrel is running full but there is no water in trough.

(i) Uplift Pressure on The Floor of Syphon Aqueduct

The floor of siphon aqueduct is subjected to uplift pressure due to the following causes

(a) Rise Of Water Table

The maximum uplift on the bottom of the floor occurs when the barrel is empty and the subsoil water rises upto the drainage bed. The uplift pressure is equal to difference of bed level of drainage and bottom surface of floor.

Uplift head (h_1) = drainage bed level – bottom level of floor

(b) Seepage From Canal

The maximum uplift pressure due to seepage from the canal occurs when the canal is full and the barrel is empty.

The uplift pressure due to seepage from canal is difficult to compute because the subsurface flow is *3D*. The seepage starts from point A in a canal trough upto which the trough base is impervious. It reappears at point *D*. At one end of impervious floor of the barrel of drainage. It is assumed that the flow takes the path ABCD where *C* is the middle of the first span of barrel. Because the flow cannot act as *2D* Khosla theory is not used for the estimation uplift pressure. Bligh's theory is used for small works however for large works it is obtained by electrical analogy method.

Total creep length L =AC+CD = L_1+L_2.

Total seepage head Hs =canal FSL – D/S bed level of drainage

Seepage head drop per unit length = Hs/L

Where h_2 = residual seepage head at C = $\left(\frac{H_S}{L}\right) * L_2$

(ii) Total Uplift Pressure

Total uplift pressure on the floor is obtained from equation

$$h_r = h_1 + h_2$$

Thickness of impervious floor = $\frac{4}{3}\left[\frac{h_r}{(G-1)}\right]$ (3.6)

Where G = Specific gravity of concrete

h_r = Total uplift

If the uplift pressure is very high then it can be reduced by following ways-

1) Creep length is increased by increasing the length of impervious floor at the bed of canal.
2) Drainage holes are drilled in the floor of the barrel's to release the uplift pressure.

3.7 DESIGN OF BARREL

I) In the case of aqueducts, the height of the openings is automatically fixed. It is equal to difference of level of trough and the bed level of drainage.

II) In the case of siphon aqueduct the required area of flow of the barrel is obtained by dividing the design drainage discharge by the permissible velocity in the barrel.

TABLE 3.2: PERMISSIBLE VELOCITY IN BARRELS

S.NO.	Types of surface	Maximum permissible Velocity (m/s)
1.	Steel and cast iron lined face	10
2.	Concrete face	6
3.	Stone masonry with cement pointing	4
4.	Brick masonry with cement plaster	4
5.	Brick masonry with cement pointing	2.5

The total area required when divided by the number of span gives the cross sectional area of each opening.

While designing a syphon aqueduct barrel maintains the minimum scouring velocity required to prevent silting.

3.8 HYDRAULIC DESIGN OF AQUEDUCT

An aqueduct carries the canal over the drainage such that the bottom of the canal trough is above of the HFL of drainage. The design discharge and HFL of the drainage should be estimated. The types of super structures should be selected. The design procedure may be summarized as follows.

i) Determine the required waterway from Lacey's equation, $P = 4.75*\sqrt{Q}$ as per given step in article 3.5.2

ii) Provide the required waterway and select suitable number of span and the number of spans.

iii) Determine the canal waterway after deciding the fluming ratio and the type of aqueduct. Use IS: 7764 for fluming ratio.

iv) Fix the dimension of canal trough and the number of bays.

v) If the canal is flumed; design the transition by suitable methods as per given in article 3.5.4

vi) Determine the height of piers and abutments by using article 3.2.1 for foundation design.

vii) Estimate the maximum scour and design the foundation using Eq-3.3

viii) Provide the suitable bank connection

3.9 DESIGN OF SYPHON AQUEDUCT

It is similar to an aqueduct however, design of barrel is different.

Design of Barrel

The area of flow of barrel is determined from maximum permissible velocity.

Cross-sectional area

$$A = \frac{Q}{V} \tag{3.7}$$

Where V = velocity in the barrel

Q = design discharge

Height of barrel h = A/Clear water way

Height of barrel should not be less than 2 meter. If the drop is greater than 1 meter, a glacis of 3:1 is provided. On the d/s side, a ramp with a slope of 5:1 is provided so that the silt is carried with drainage water. The vertical distance between d/s bed and trough should not be less than 1 meter. And difference between vertical drop wall and u/s end of piers should not be less than 1 meter.

TABLE 3.3: PERMISSIBLE VELOCITY IN BARRELS

S.NO	Types of surface	Maximum permissible Velocity (m/s)
1.	Steel and cast iron lined face	10
2.	Concrete face	6
3.	Stone masonry with cement pointing	4
4.	Brick masonry with cement plaster	4
5.	Brick masonry with cement pointing	2.5

TABLE 3.4: THE MINIMUM VELOCITY TO PREVENT SILTING

S.No.	Material	Particle size (mm)	Minimum velocity (m/s)
1.	Silt	0.002 to 0.02	0.15 to .20
2.	very fine sand	0.02 to 0.1	0.20 to .25
3.	fine sand	0.1 to 0.5	0.25 to .40
4.	coarse sand	0.5 to 2.0	0.40 to .60

Cutoff

Suitable cutoff walls should be provided at the end of the impervious floor on the U/S and D/S sided to prevent scour. The maximum scour of $1.25S$ and $1.50S$ are generally assumed at the U/S and D/S ends respectively, where S is Lacey's normal scour depth.

Impervious Floor

The total length of the impervious floor of the drainage is fixed according to the requirement of the exit gradient and economy. The thickness of the floor is determined for the maximum residual uplift pressure at different points. The maximum uplift pressure occurs under the high flood condition. A minimum thickness of 0.3m is usually recommended.

Concrete Block Protection

At the end of the impervious floor, cement concrete blocks of suitable size are provided so that they that will not get dislodged during floods. The minimum length of the U/S protection is usually taken as d1, where d1 is the depth of the scour below the U/S bed. The minimum length of D/S protection is usually taken as 1.5 d_2, where d_2 is the depth of the scour the d/s bed.

Loose Stone Protection or Pitching

Beyond the concrete block protections, loose stone protection (or pitching) is provided on the U/S and D/S sides. It consists of stones, loose boulders or brick bats. The apron is designed as a launching apron. The quantity of stone is determined as in the case of a weir. Generally, a minimum length of apron of 2 to 5 m on U/S and 3 to 10m on D/S is recommended.

Chapter 4
CASE STUDY ON GOMTI AQUEDUCT

4.1 INTRODUCTION

The Sarda Sahayak Pariyojna was conceived to utilize the irrigation potential of the river Ghagra in various districts of central and eastern Uttar Pradesh. The area was scantily fed by the lower reaches of the Sarda canal system. The necessity for augmenting supplies there from the river Ghagra was felt as a consequence of the requirements of a developing society.

The Pariyojna envisaged the construction of barrages across the rivers Ghagra and Sarda, a link channel connecting the two rivers and 260-km long feeder channel besides the remodeling of different branches of the existing canal system and construction of several new channels. The feeder channel has a discharge capacity of 650 cumecs and it crosses several small and large drainages. This channel crosses the river Gomti about 20 km from Lucknow where the discharge is 357 cumecs. The design discharge of the Gomti river at the aqueduct site is 4350 cumec.

The work of the Gomti aqueduct was started in 1973 and was completed in 1978.

4.2 GENERAL FEATURES

4.2.1) *Main Structure*

Having 12 spans of 31.8 m each, the superstructure consists of 9.9 m deep longitudinal pre-stressed concrete girders spanning 31.8 m between the pier centres. These are connected by pre-stressed concrete ties at the bottom flange and pre-stressed concrete ties and also vertical stiffeners are placed at 1.95 m centres. The top flanges of the main girders are 5 m wide and also serve the purpose of carrying a single lane roadway. A thin R.C.C trough of 12.8 m by 7.45 m supported on bottom cross girders and side stiffeners are provided to carry the water across the aqueduct.

The unique feature of the main girder is that these are pre-stressed in three directions. There are 47 HT cables of 1277mm stressed vertically at each girder. This vertical pre-stressing connects the three lifts of concrete properly. Each main girder is pre-stressed longitudinally with 40 HT cables 1278mm. The girders get their third directional pre-stress through cross girders and top ties.

The superstructure is supported over R.C.C hollow piers and rests on MS rocker and roller bearings designed to carry vertical load of 1300 T.

The foundation wells for the aqueduct are 12m wide, 27 m long double D-type sunk approximately to about 35m.

4.2.2) ***Flumed Approaches***

The canal section on u/s and d/s of the aqueduct is 23m wide and has side slopes of 2:1. This section has been flumed to be the aqueduct section in a length of 37m on the u/s and 55m on the d/s side of the aqueduct. The hydraulic profiles of the flume were decided on the basis of model studies.

The foundation of the flumed approaches consists of wells only. The size of the wells in the approach is 14m by 26m which is still bigger than the wells provided for the main structure. There are three foundation wells on the u/s side and four on the d/s side. One well on both sides are double D-types and the remaining are rectangular in shape.

The gap between holes is covered by simply supported slabs resting on neoprene bearings. The main structure of the aqueduct is connected to the transitions at each end by a mild steel trough which can slide and allow free movement of the main superstructure during earthquakes. The sealing is done in such a way that even after sliding, the structure remains water tight.

4.2.3) ***Preliminary Investigations***

1) Hydrology

No data on the river course and flood hydrographs near the aqueduct site were available. A contour plan for the year 1972 and stage discharge tables near a bridge in Lucknow town as well as about 200 m u/s of this bridge is available. The abnormal floods of 1960 and 1971 provided good hydrological data for fixing and design discharge at this site. The Central Water Commission, the Irrigation Research Institute (IRI) and the UP PWD carried out detailed studies of the river Gomti near Lucknow in order to fix maximum flood discharge for designing flood works and new bridges in Lucknow. The State Flood Control Board, on the basis of these studies, laid down that the flood works be designed for a maximum discharge of 4260 cumec. The catchment area of the river upto Lucknow town id 9441.2 sq km. Adding 158.8 sq km further catchment beyond Lucknow town it was decided to adopt the design discharge for Gomti aqueduct site as a 4530 cumec.

4.3 SITE INVESTIGATIONS AND BEARING CAPACITY

For a work of such magnitude it was necessary to determine the values of he bearing capacity and settlements as precisely as possible. For investigations of various subsoil characteristics about 100 deep borings were done and disturbed and undisturbed samples were collected. These were analyzed by the Irrigation Research Institute, Roorkee, as well as by IIT, Kanpur.

IRI evaluated the unconfined compressive strength of the clayey soil below the foundation level as 1 kg/sq cm which yields allowable bearing capacity of 4.5 kg/sq cm with a factor of safety as 3. The settlement for the load was found to vary from 4-5 cm. The recommendation of IIT, Kanpur was different. The allowable bearing capacity was evaluated as 3.3 kg/sq cm and the anticipated settlement as 32 cm.

As the data was at variance, it was considered necessary to carry out an actual load test at the site by sinking a small well, upto the proposed foundation level. A circular wall of 5 m diameter and 1.25 m thick steining was sunk on the left bank of the river near the proposed site.

For the area of the well, 2400 t of kentledge was required for a bearing capacity of $40t/m^2$ with a factor of safety 3. For ultimate failure 25% extra load was presumed. Hence, concrete blocks of 2.1m* 1.5 m*0.9m weighing about 3000 t were cast. Later on these blocks could be used in the apron of the guide banks.

On performing the load test and taking detailed observation following parameters were evaluated:

i) Skin friction as 1.9 t/m^2

ii) Allowable bearing capacity as 4.5 kg/cm^2

iii) The immediate settlement due to plastic deformation against a load intensity of 5 kg/cm^2 which excludes settlement of excavation of soil. Out of this, a settlement of 3.8 cm will take place during construction of the aqueduct and the rest on application of water load.

iv) Average consolidation settlement against load intensity of 5 kg/cm^2 at 11.5 cm and 5.3 cm with piers founded at river bed levels E.L 97 and E.L 106 respectively. The total average settlement after adding the settlement due to plastic deformation works out to 13.4 and 7.2 cm for the two conditions.

It was decided that one-half of the total settlement may be provided for, by constructing top level of the pier higher by 5 cm above the design level so as to have minimum deviation of the aqueduct floor level from the design level.

Chapter 5

DEVLOPMENT OF PROGRAMME FOR AQUEDUCT AND SYPHON AQUEDUCT

5.1 UTILITY OF COMPUTER PROGRAMMING

Computer aided programme makes easy and accurate way of design of any structure and save the time. C++ language has not lost its importance and popularity in software industry in spite of high level languages. It has slowly become a way of work in Civil Engineering nowadays. The presence of software's like Staad Pro and AutoCAD still makes C and C++ an important aspect in design problems of Civil Engg. C language has excellent support of high level and low level functionality which makes it suitable for various applications. Moreover for most of the Civil Engg works, efficiency of space and time becomes crucial and can be very effectively achieved by C++ language. The inherent flexibility and tolerance of this language, at times, makes it suitable for many different developments environments. Concept of procedure-oriented programming can be well understood and learnt with the help of C language which is also needed for implementation of functions and modules of objects oriented languages like C++ ,Java *etc.*

5.2 DEVLPOMENT OF PROGRAMME USING C++

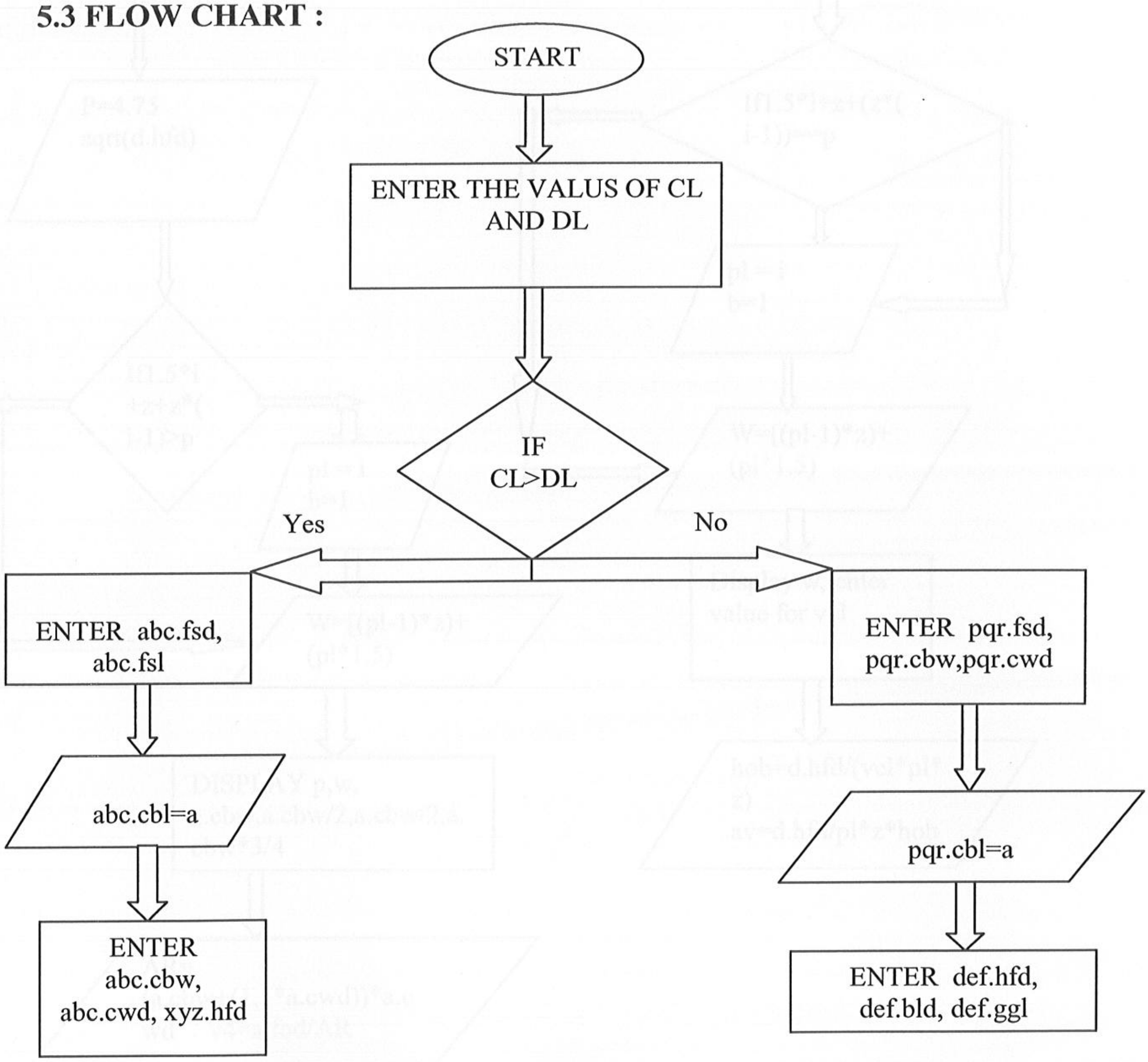
C++ is a versatile language for handling very large problems. It is capable of performing tasks like development of editors, compilers, etc.

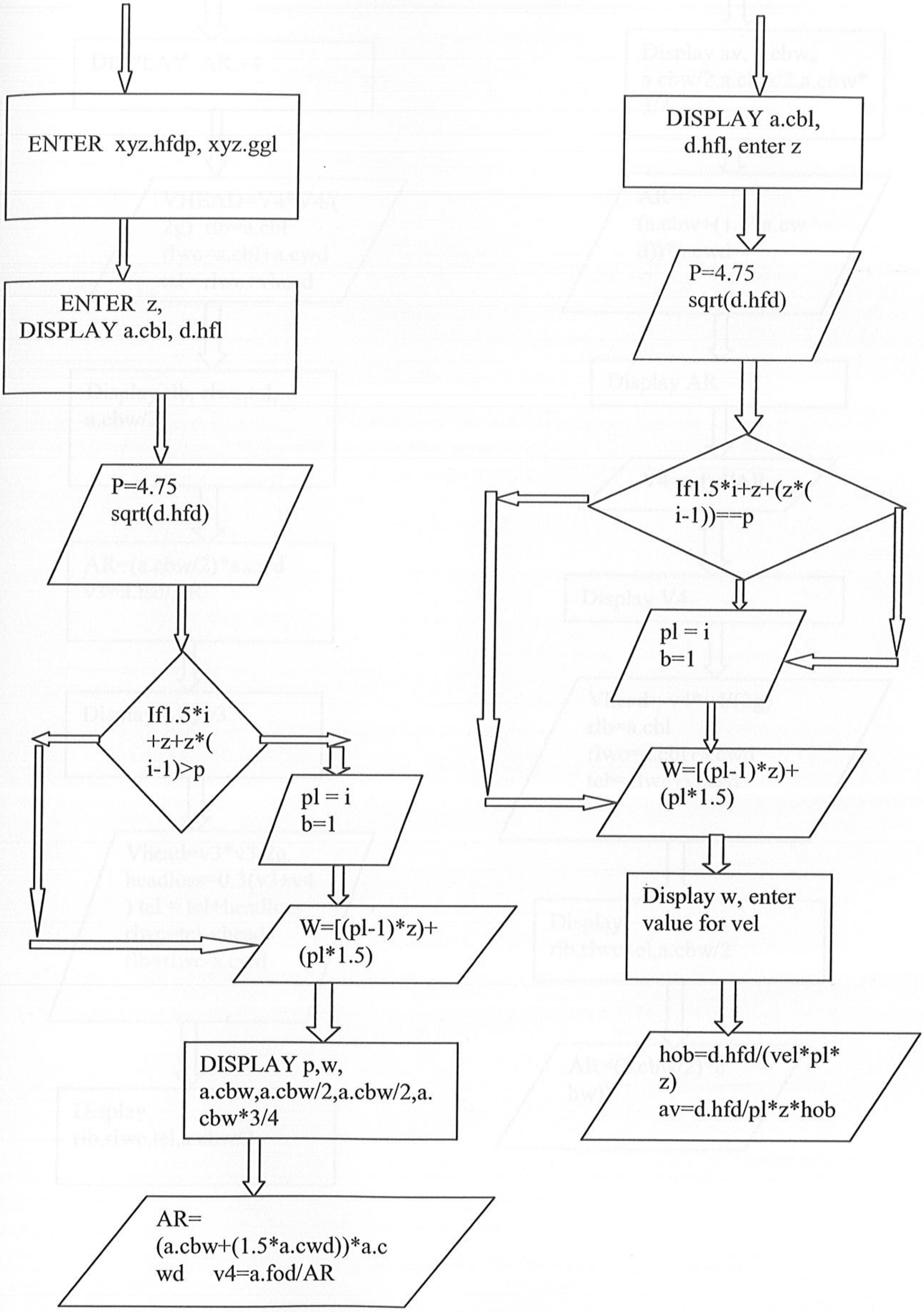
C++ programming languages has been used to develop the above methodology. The basic aim of our project is to design cross drainage works faster and effectively using C++ as programming languages and therefore avoid all tedious manual work that go on during manual designing. With the help of this source code an engineer need no specifically go into the minute details of every step as is needed in manual calculations.This saves the precious time of the engineer.

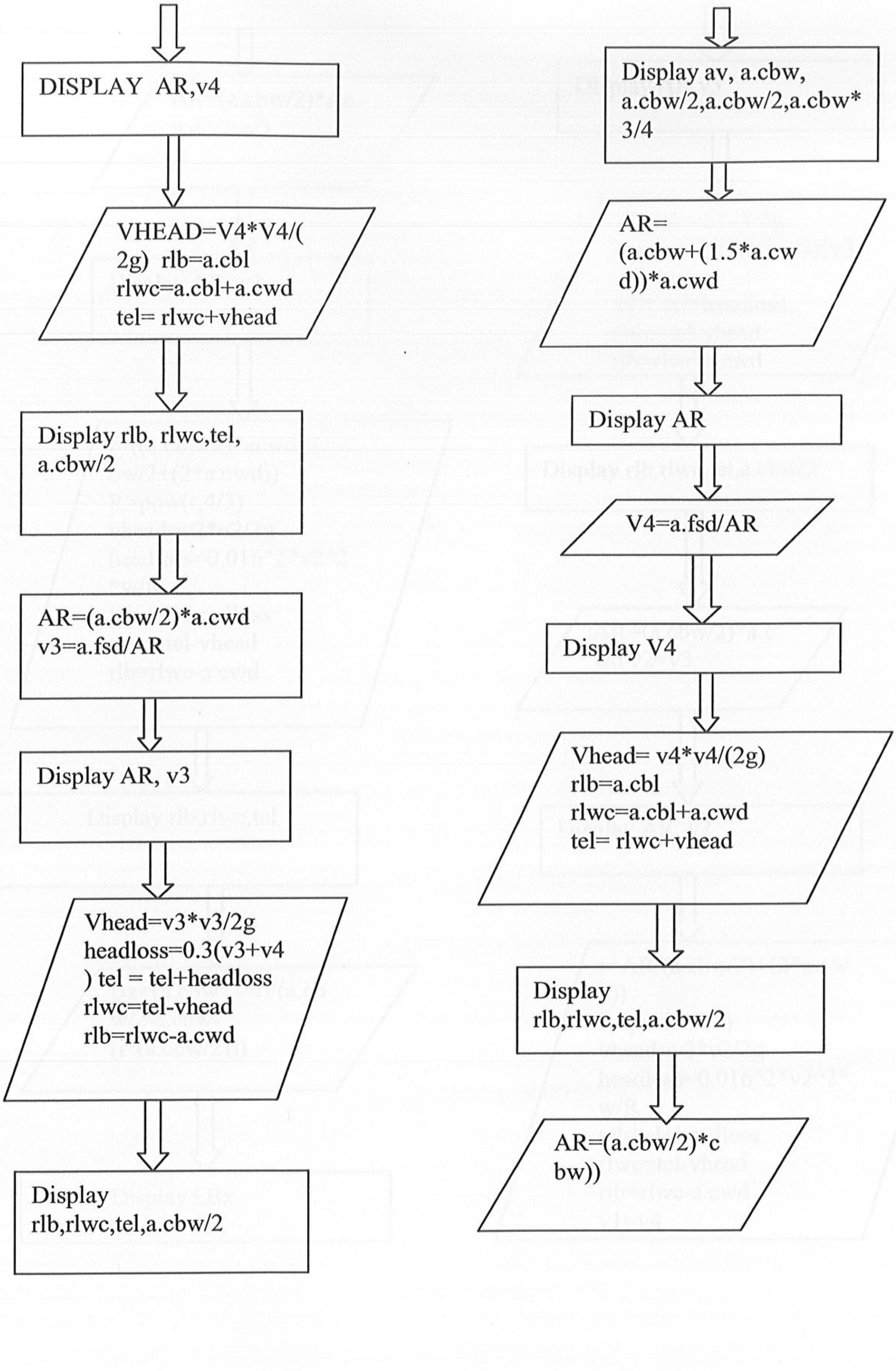
The source code of the programme is given in APPENDIX ~~Annexure~~ A

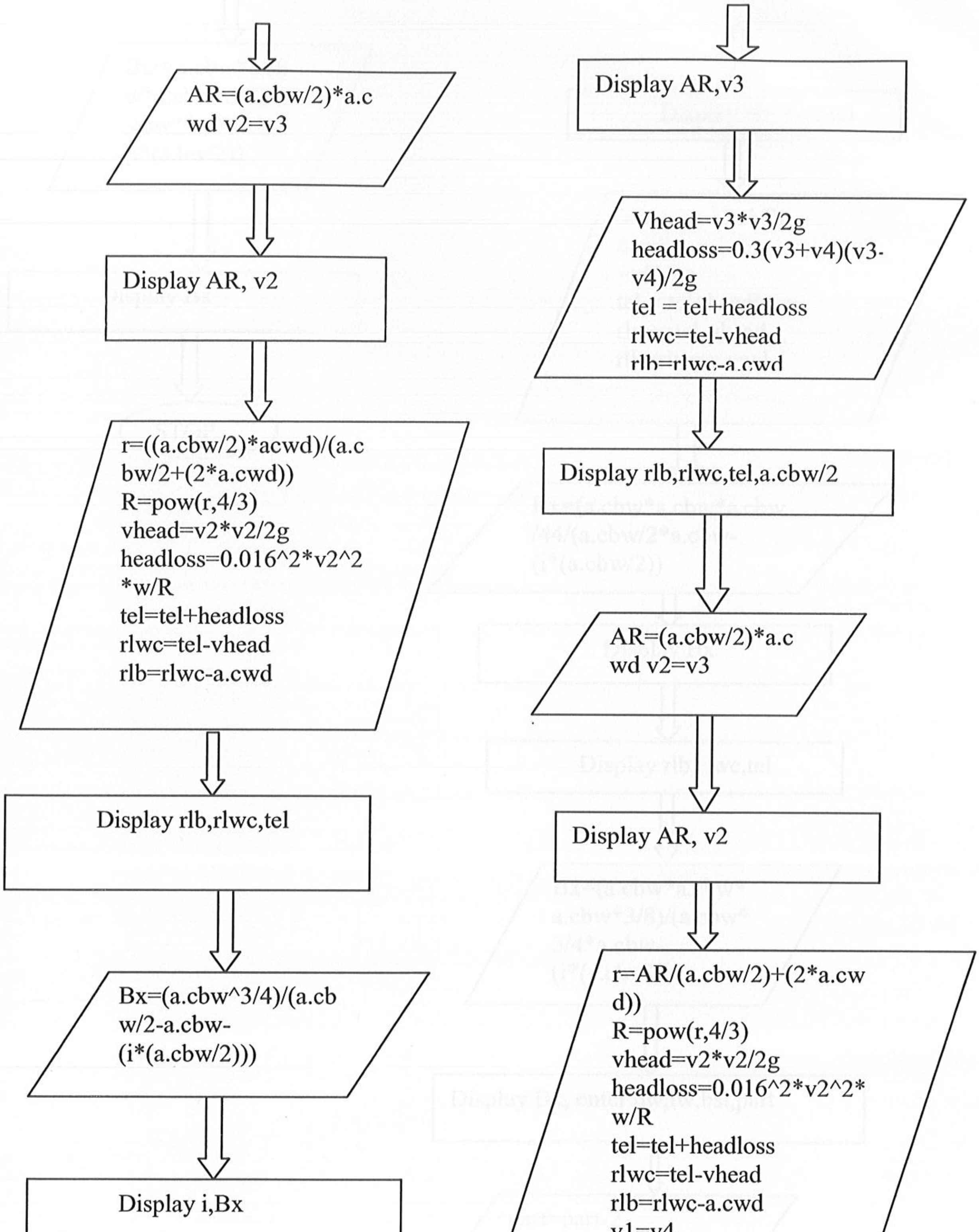
The flow chat of the programme is given below:

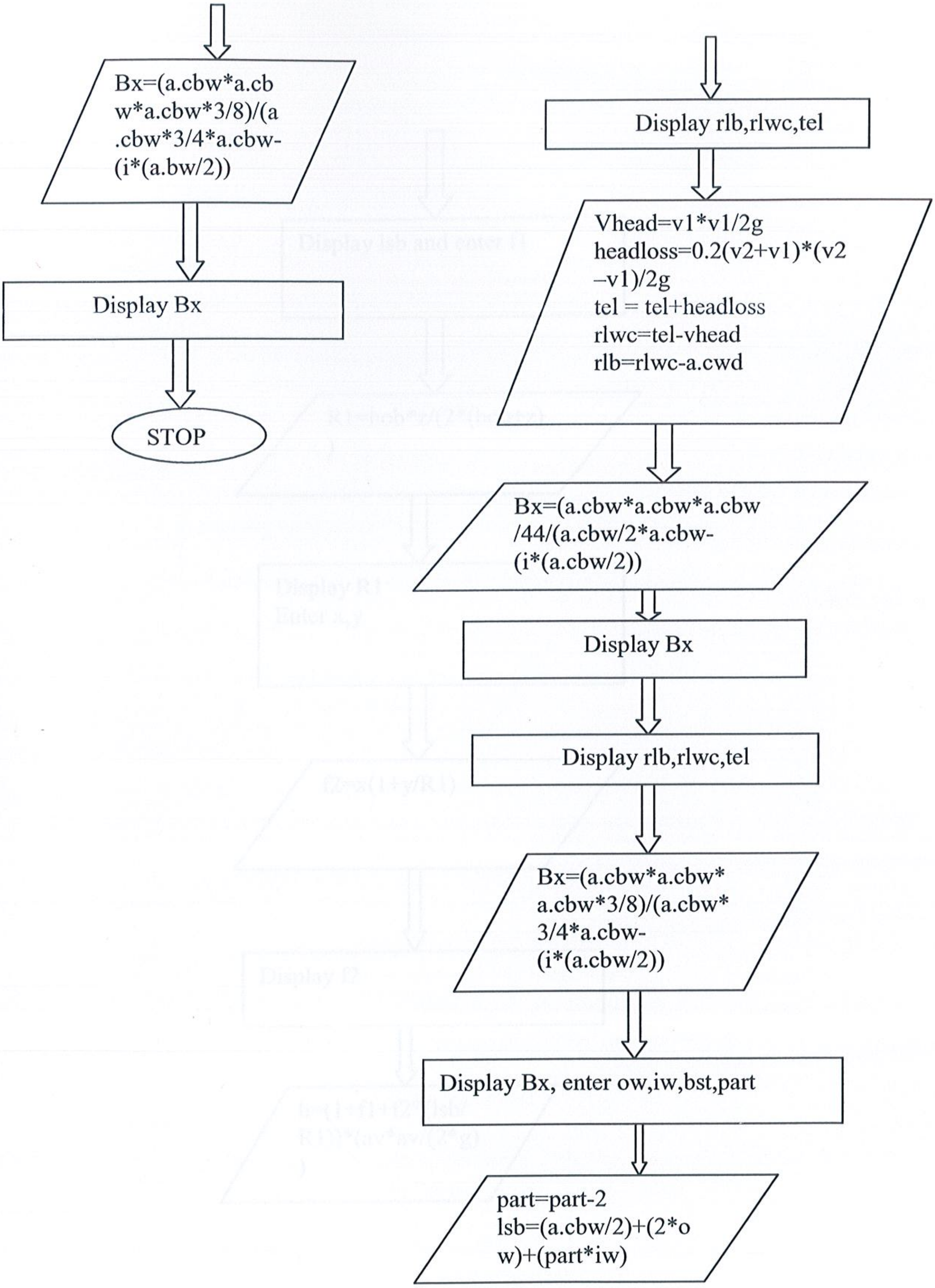
5.3 FLOW CHART :

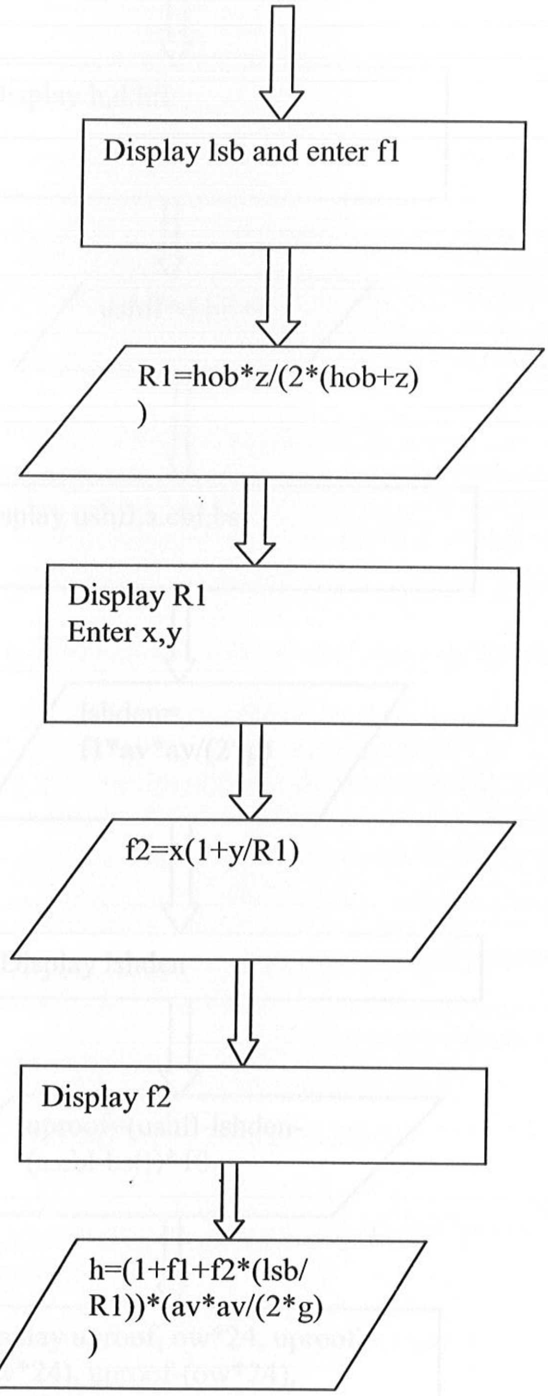


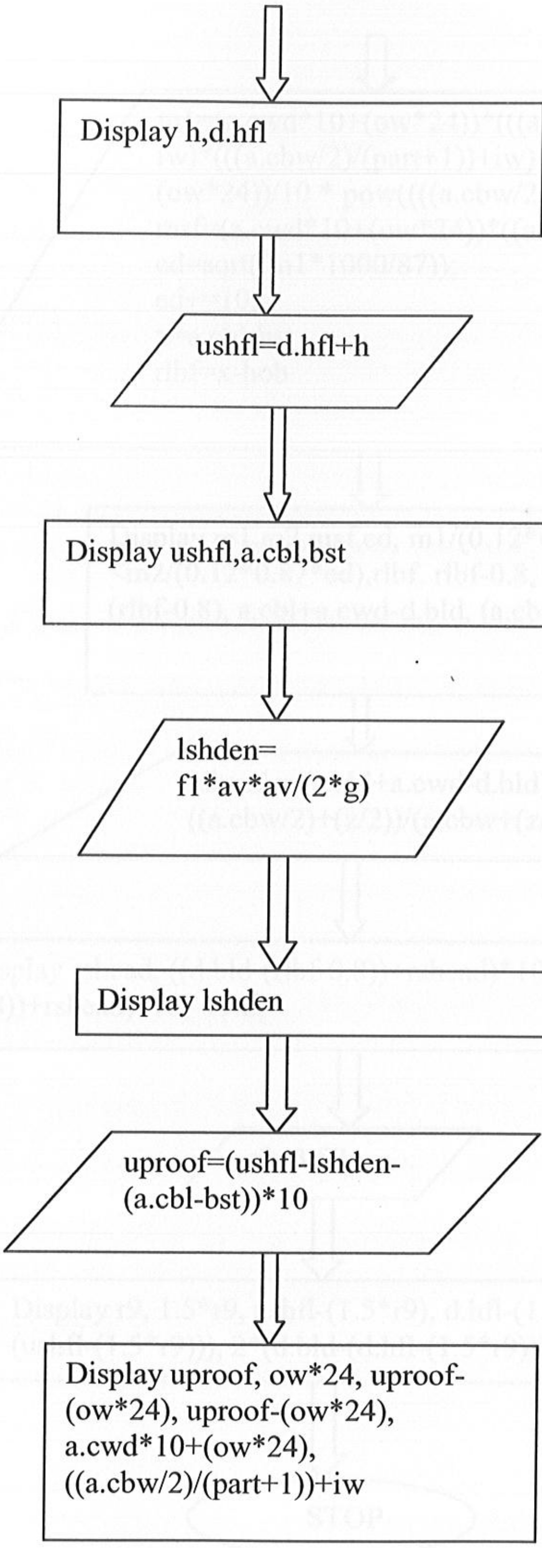

ENTER xyz.hfdp, xyz.ggl
ENTER z,
DISPLAY a.cbl, d.hfl
P=4.75
sqrt(d.hfd)
If1.5*i
+z+z*(
i-1)>p
pl = i
b=1
W=[(pl-1)*z)+
(pl*1.5)
DISPLAY p,w,
a.cbw,a.cbw/2,a.cbw/2,a.
cbw*3/4
AR=
(a.cbw+(1.5*a.cwd))*a.c
wd v4=a.fod/AR
DISPLAY a.cbl,
d.hfl, enter z
P=4.75
sqrt(d.hfd)
If1.5*i+z+(z*(
i-1))==p
pl = i
b=1
W=[(pl-1)*z)+
(pl*1.5)
Display w, enter
value for vel
hob=d.hfd/(vel*pl*
z)
av=d.hfd/pl*z*hob


DISPLAY AR,v4
VHEAD=V4*V4/(2g) rlb=a.cbl rlwc=a.cbl+a.cwd tel= rlwc+vhead
Display rlb, rlwc,tel, a.cbw/2
AR=(a.cbw/2)*a.cwd v3=a.fsd/AR
Display AR, v3
Vhead=v3*v3/2g headloss=0.3(v3+v4) tel = tel+headloss rlwc=tel-vhead rlb=rlwc-a.cwd
Display rlb,rlwc,tel,a.cbw/2
Display av, a.cbw, a.cbw/2,a.cbw/2,a.cbw*3/4
AR=(a.cbw+(1.5*a.cwd))*a.cwd
Display AR
V4=a.fsd/AR
Display V4
Vhead= v4*v4/(2g) rlb=a.cbl rlwc=a.cbl+a.cwd tel= rlwc+vhead
Display rlb,rlwc,tel,a.cbw/2
AR=(a.cbw/2)*cbw))


AR=(a.cbw/2)*a.c
wd v2=v3
Display AR, v2
r=((a.cbw/2)*acwd)/(a.c
bw/2+(2*a.cwd))
R=pow(r,4/3)
vhead=v2*v2/2g
headloss=0.016^2*v2^2
*w/R
tel=tel+headloss
rlwc=tel-vhead
rlb=rlwc-a.cwd
Display rlb,rlwc,tel
Bx=(a.cbw^3/4)/(a.cb
w/2-a.cbw-
(i*(a.cbw/2)))
Display i,Bx
Display AR,v3
Vhead=v3*v3/2g
headloss=0.3(v3+v4)(v3-
v4)/2g
tel = tel+headloss
rlwc=tel-vhead
rlb=rlwc-a.cwd
Display rlb,rlwc,tel,a.cbw/2
AR=(a.cbw/2)*a.c
wd v2=v3
Display AR, v2
r=AR/(a.cbw/2)+(2*a.cw
d))
R=pow(r,4/3)
vhead=v2*v2/2g
headloss=0.016^2*v2^2*
w/R
tel=tel+headloss
rlwc=tel-vhead
rlb=rlwc-a.cwd
v1=v4


Bx=(a.cbw*a.cb
w*a.cbw*3/8)/(a
.cbw*3/4*a.cbw-
(i*(a.bw/2))
Display Bx
STOP
Display rlb,rlwc,tel
Vhead=v1*v1/2g
headloss=0.2(v2+v1)*(v2
–v1)/2g
tel = tel+headloss
rlwc=tel-vhead
rlb=rlwc-a.cwd
Bx=(a.cbw*a.cbw*a.cbw
/44/(a.cbw/2*a.cbw-
(i*(a.cbw/2))
Display Bx
Display rlb,rlwc,tel
Bx=(a.cbw*a.cbw*
a.cbw*3/8)/(a.cbw*
3/4*a.cbw-
(i*(a.cbw/2))
Display Bx, enter ow,iw,bst,part
part=part-2
lsb=(a.cbw/2)+(2*o
w)+(part*iw)


Display lsb and enter f1
R1=hob*z/(2*(hob+z)
)
Display R1
Enter x,y
f2=x(1+y/R1)
Display f2
h=(1+f1+f2*(lsb/
R1))*(av*av/(2*g)
)


Display h,d.hfl
ushfl=d.hfl+h
Display ushfl,a.cbl,bst
lshden=
f1*av*av/(2*g)
Display lshden
uproof=(ushfl-lshden-
(a.cbl-bst))*10
Display uproof, ow*24, uproof-
(ow*24), uproof-(ow*24),
a.cwd*10+(ow*24),
((a.cbw/2)/(part+1))+iw

m1=(a.cwd*10+(ow*24))*(((a.cbw/2)/(part+1))+iw)*(((a.cbw/2)/(part+1))+iw)/10 m2=(uproof-(ow*24))/10 * pow((((a.cbw/2)/(part+1))+iw),2);
msf=(a.cwd*10+(ow*24))*((a.cbw/2)/(part+1))/2
ed=sqrt((m1*1000/87));
ed+=10;
x=a.cbl-bst
rlbf=x-hob

Display m1,m2,msf,ed, m1/(0.12*0.87*ed), <m2/(0.12*0.87*ed),rlbf, rlbf-0.8, d.bld-(rlbf-0.8), a.cbl+a.cwd-d.bld, (a.cbw)+(z/2)

rshead = (a.cbl+a.cwd-d.bld)*(1-((a.cbw/2)+(z/2))/(a.cbw+(z/2)))

Display rshead, ((d.bld-(rlbf-0.8))+rshead)*10, (((d.bld-(rlbf-0.8))+rshead)*10)-19.2,

r9=3.59

Display r9, 1.5*r9, ushfl-(1.5*r9), d.hfl-(1.5*r9), 2*(rlbf-(ushfl-(1.5*r9))), 2*(d.bld-(d.hfl-(1.5*r9)))

STOP

5.4.1 EXAMPLE RUN

```
Enter Clear span between piers - 9

Since the canal bed level 212m is much above the H.F.L of drainage 210m an aqua
duct will be constructed
Lacey's perimetre = 82.2724

Total length of waterway = 85.5m

Bed width of canal = 20m
Let the width to fumed to 10m
Providing the splay of 2:1 contract. length of contract. transition = 10
Providing the splay of 3:1 expansion length of expansion transition = 15
```

```
HEAD LOSS AND BED LEVELS AT DIFFERENT SECTIONS

At section 4-4, the canal returns to its normal section

Area of the Trapezoidal canal section = 33.375sq.m
Velocity = 0.958802m/sec
R.L of bed at 4-4 = 212m
R.L of water surface at 4-4 = 213.5m
R.L of T.E.L at 4-4 = 213.547m
```

```
At section 3-3
Bed width = 10m
Area of channel = 15sq.m
Velocity at 3-3 = 2.13333m/sec
R.L of bed at 3-3 = 211.87m
R.L of water surface at 3-3 = 213.37m
R.L of T.E.L at 3-3 = 213.602m

At section 2-2
Bed width = 10m Area of channel = 15sq.m Velocity at 2-2 = 2.13333m/sec
R.L of bed at 2-2 = 211.957m
R.L of water surface at 2-2 = 213.457m
R.L of T.E.L at 2-2 = 213.689m

At section 1-1Velocity at 1-1 = 0.958802m/sec
R.L of bed at 1-1 = 212.179m
R.L of water surface at 1-1 = 213.679m
R.L of T.E.L at 1-1  = 213.726m

Contraction transition is given as :-
0       10
2       11.1111
4       12.5
6       14.2857
8       16.6667
```

10	20

Expansion transition is given as :-

0	10
2	10.7143
4	11.5385
6	12.5
8	13.6364
10	15
12	16.6667
14	18.75

5.4.2 SYPHON AQUEDUCT EXAMPLE

Since the canal bed level206.4m is slightly below the H.F.L of drainage207m an syphon aquaduct will be constructed
Enter length of span - 8

Total length of waterway = 106
Enter the value of velocity - 2

Actaul velocity through barrels = 2m/s

Bed width of canal = 30m
Let the width to fumed to 15m
Providing the splay of 2:1 contract. length of contract. transition = 15
Providing the splay of 3:1 expansion length of expansion transition = 22.5

HEAD LOSS AND BED LEVELS AT DIFFERENT SECTIONS

At section 4-4, the canal returns to its normal section

Area of the Trapezoidal canal section = 51.84sq.m
Velocity = 0.771605m/sec
R.L of bed at 4-4 = 206.4m
R.L of water surface at 4-4 = 208m
R.L of T.E.L at 4-4 = 208.03m

At section 3-3
Bed width = 15m
Area of channel = 24sq.m
Velocity at 3-3 = 1.66667m/sec
R.L of bed at 3-3 = 206.322m
R.L of water surface at 3-3 = 207.922m
R.L of T.E.L at 3-3 = 208.064m

At section 2-2
Bed width = 15mArea of channel = 24sq.mVelocity at 2-2 = 1.66667m/sec
R.L of bed at 2-2 = 206.379m
R.L of water surface at 2-2 = 207.979m
R.L of T.E.L at 2-2 = 208.121m

At section 1-1Velocity at 1-1 = 0.771605m/sec
R.L of bed at 1-1 = 206.513m
R.L of water surface at 1-1 = 208.113m
R.L of T.E.L at 1-1 = 208.143m

Contraction transition is given as :-

0	15
2	16.0714
4	17.3077
6	18.75
8	20.4545
10	22.5
12	25
14	28.125

Expansion transition is given as :-

0	15
2	15.6977
4	16.4634
6	17.3077
8	18.2432
10	19.2857
12	20.4545
14	21.7742
16	23.2759
18	25
20	27
22	29.3478

```
Enter the thickness of outer walls - 0.4

Enter the thickness of inner walls - 0.3

Enter the thickness of bottom slab - 0.4

Enter total no. of partitions - 4

Length of syphon barrel = 16.4m
Enter the coeff. of loss of head at entry - 0.505
R1 = 0.906344
Enter value of a and b(as per IS 7784)  - 0.00316
0.030
f2 = 0.0032646
Headloss = 0.319198
d/s H.F.L = High Flood Level of Drainage = 207
The u/s H.F.L. = 207.319
R.L. of bottom of trough =  206/n Loss of head at entry of barrel = 0.103061
Uplift on the roof =12.1614 kN/sq.m
Assuming the unit weight of concrete = 24 kN/sq.m
Downward load of concrete slab = 9.6kN/sq.m
Balance uplift pressure = 2.56143kN/sq.m
Downward water load acting = 16kN/sq.m
Total downward load = 25.6kN/sq.m
Effective span = 5.3m
```

```
Maximum downward sagging bending moment = 71.9104kN-m
Maximum Hogging moment due to uplift = 7.19507kN-m
Maximum shear force = 64kN

Using 1:2:4 cement concrete the effective depth is 38.7499cm
Steel reqd. at bottom of slab = 17.7755sq.cm
Steel reqd. at the top of slab = 1.77854sq.cm

Static head on barrel floor = 203.656m
Assuming a thickness of 0.8m slab
R.L of bottom floor = 202.856m
The static uplift on floor = 4.14375m
Seapage head = 0.999994m
Seapage line = abc and traverse total creep length is 34m
Residual seapage head  = 0.441174m
Total uplift  = 45.8492kN/sq.m
Balance to be resisted by r/f due to bending action  = 26.6492kN/sq.m

                  Design of cutoffs and protection work for drainage floor

Depth of Scour = 3.59m
Depth of u/s cut-offs below H.F.L = 5.385m
Bottom of u/s cut-off  = 201.934m
Bottom of d/s cutoff = 201.615m
Length of upstream protection = 3.4441m
Length of d/s pitching = 10.77m
```

Manual Calculations: Manual calculations for the above typical example run is shown in Appendix B.

CONCLUSIONS

Aqueducts are the hydraulic structures and these are needed whenever a canal and River cross each other at different bed levels. The main objective of the present study was to get the clear understanding of the various steps required in the hydraulic design of the aqueduct and siphon aqueduct. Keeping in mind the above objective a thorough knowledge of the subjects was gained.

An attempt was also made to develop the computer aided programme of this hydraulic structure using computer language. The programme thus developed has potential to determine the all hydraulic design parameter of the aqueducts and syphon aqueduct.

The software designed by us satisfactorily designs the following with user defined inputs. Although the project does not put forward the design of Super Passage and Canal Syphon, the software holds the capability of giving design results satisfactory enough of Aqueducts and Syphon Aqueducts, for any engineer.

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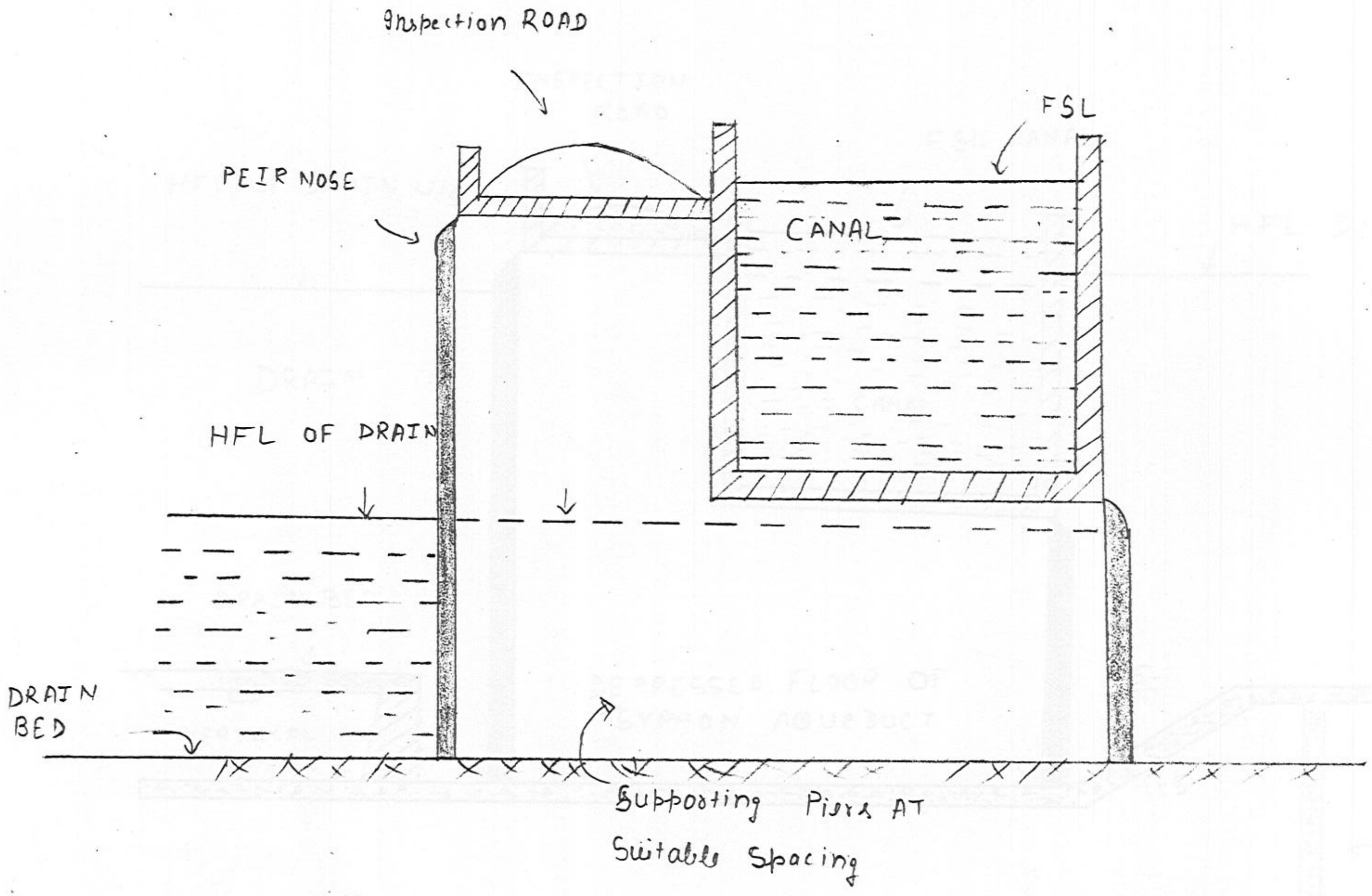
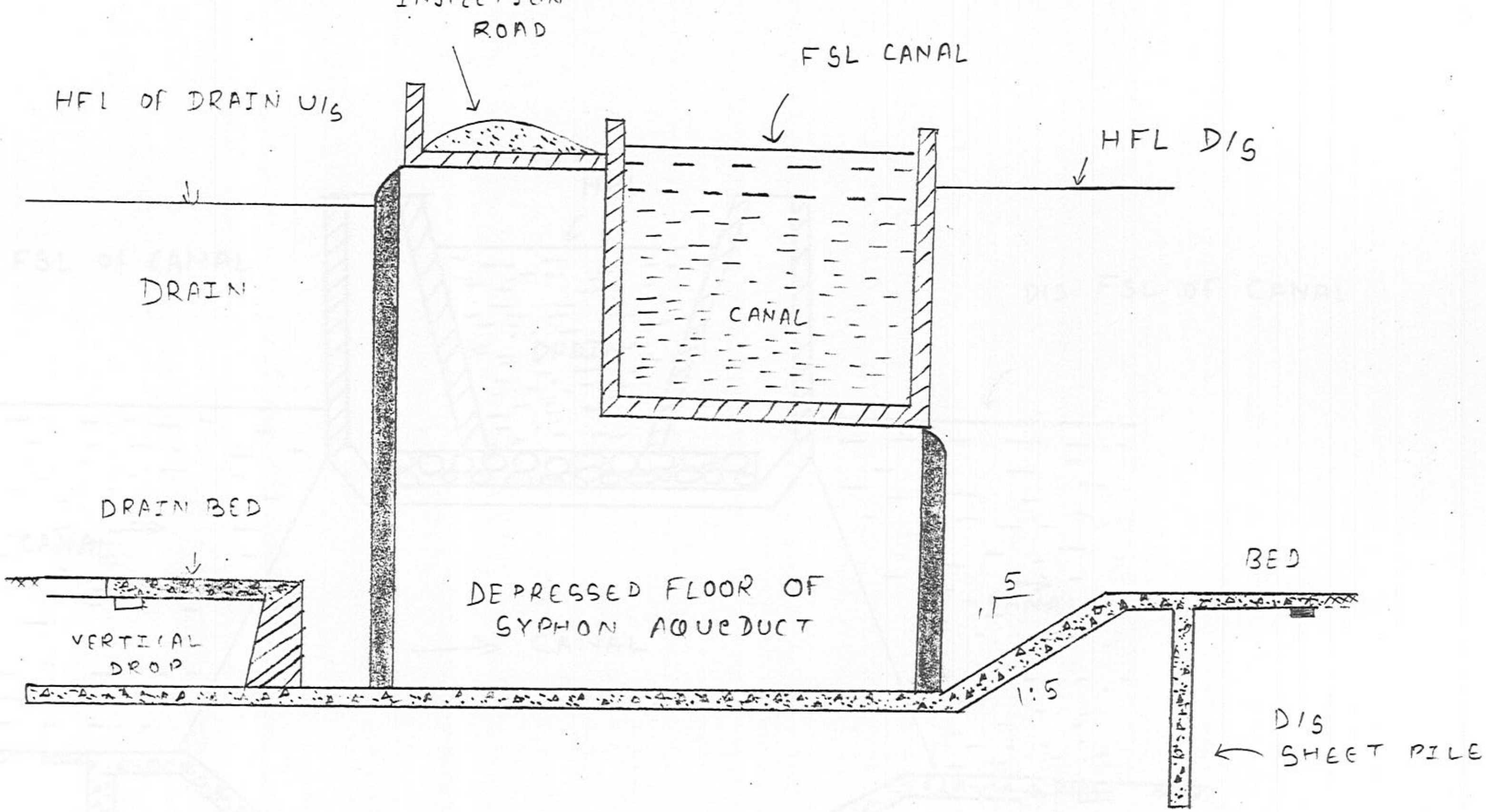


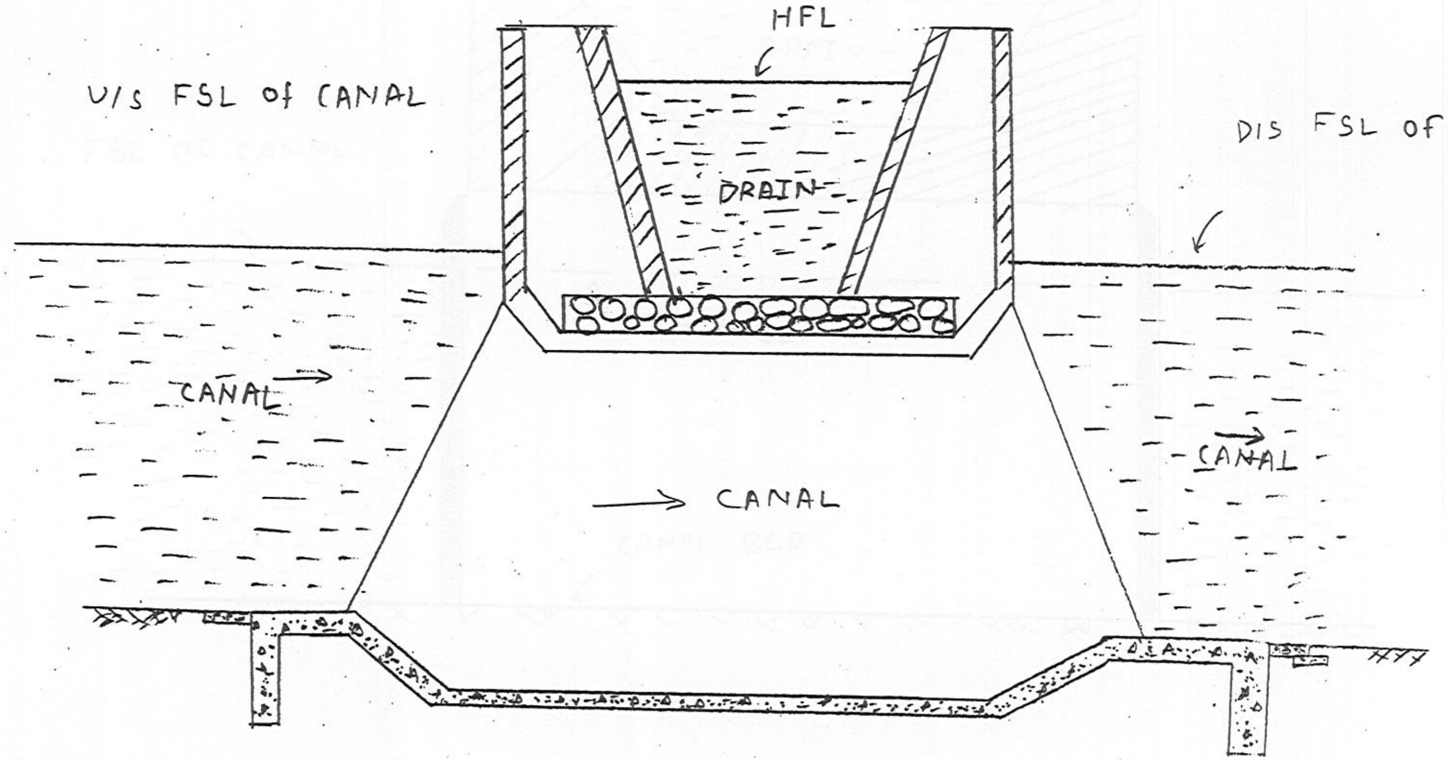
Typical Cross Section of an Aqueduct

FIG 1·1



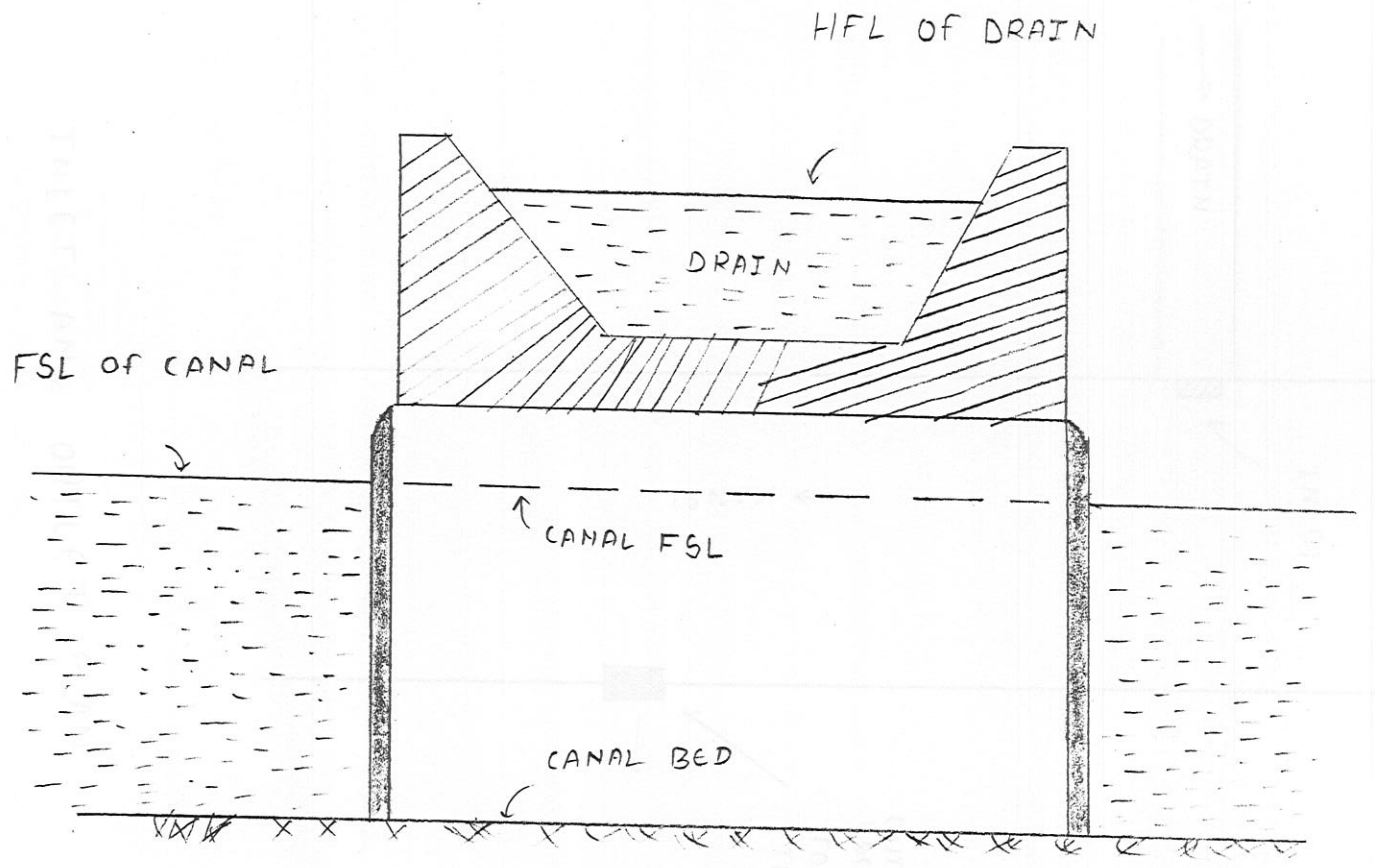
TYPICAL CROSS SECTION OF SYPHON AQUEDUCT

FIG:- 1.2



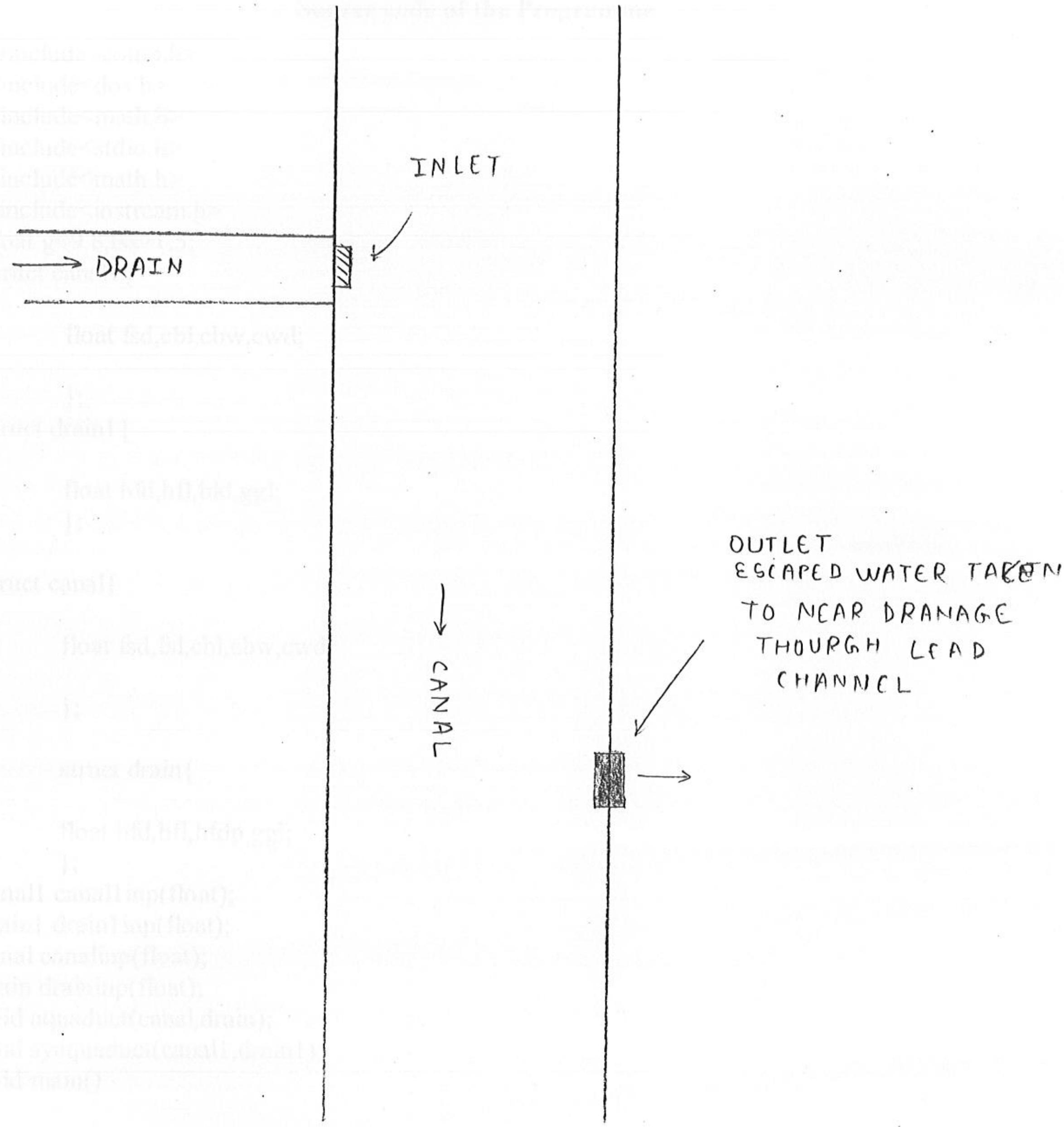
TYPICAL CROSS SECTION CANAL SYPHON

FIG - 1.3



TYPICAL CROSS SECTION OF SUPER PASSAGE

FIG– 1.4



INLET AND OUTLET PLAN

FIG 1.5

APPENDIX A

Source code of the Programme

```
#include<conio.h>
#include<dos.h>
#include<math.h>
#include<stdio.h>
#include<math.h>
#include<iostream.h>
float g=9.8,tss=1.5;
struct canal1 {

        float fsd,cbl,cbw,cwd;

        };
struct drain1 {

        float hfd,hfl,bld,ggl;
        };

struct canal{

        float fsd,fsl,cbl,cbw,cwd;

        };
        struct drain{

        float hfd,hfl,hfdp,ggl;
        };
canal1 canal1inp(float);
drain1 drain1inp(float);
canal canalinp(float);
drain draininp(float);
void aquaduct(canal,drain);
void syaquaduct(canal1,drain1);
void main()

{
clrscr();
                        canal abc;
                        drain xyz;
                        canal1 pqr;
                        drain1 def;
float cl,dl;
```

```
cout<<"\n\n\n\n\n*****************************************************";
 cout<<"**        Design of Cross Drainage Works          **";
 cout<<"**                                                **";
 cout<<"**         Aquaduct / Syphon-Aquaduct             **";
 cout<<"**                                                **";
 cout<<"**                                                **";
 cout<<"**                     BY                         **";
 cout<<"**         Chandresh Kumar - 041601               **";
 cout<<"**         Maushmi Samjhdar - 041604              **";
 cout<<"**                                                **";
 cout<<"****************************************************";
 delay(3000);
 clrscr();
 cout<<"\n\n\n\n Enter the value for Canal bed level - ";
 cin>>cl;
 cout<<"\n\n Enter the value for High flood level - ";
 cin>>dl;
     clrscr();
if(cl>dl)
{
gotoxy(20,10);
cout<<" AQUADUCT CASE SELECTED AS C.B.L > H.F.l";
delay(2000);
abc=canalinp(cl);
xyz=draininp(dl);
aquaduct(abc,xyz);
}

else
{
gotoxy(20,10);
cout<<" SYPHON-AQUADUCT CASE SELECTED AS C.B.L < H.F.L";
delay(2000);
pqr=canal1inp(cl);
def=drain1inp(dl);
syaquaduct(pqr,def);
}
}
void aquaduct(canal a, drain d)
{
clrscr();
float p,W,AR,v1,v2,v3,v4,Vhead,rlb,rlwc,tel,headloss,R,r,Bx;
int pl,b=0,i,z;
cout<<"\n Enter Clear span between piers - ";
                    cin>>z;
```

```
cout<<"\n Since the canal bed level "<<a.cbl<<"m is much above the H.F.L of drainage "<<d.hfl<<"m an aquaduct will be constructed";

            p=4.75 * sqrt(d.hfd);

for(i=1;i<=p&&b==0;i++)
{
if((1.5*i + (z * (i-1)))>=p)
{
pl=i;
b=1;
}
}
W=((pl-1)*z)+(pl*1.5);
cout<<" \n Lacey's perimetre = "<<p;
cout<<"\n\n Total length of waterway = "<<W<<"m";

cout<<"\n\n Bed width of canal = "<<a.cbw<<"m";

cout<<"\n Let the width to fumed to "<<a.cbw/2<<"m";

cout<<"\n Providing the splay of 2:1 contract. length of contract. transition = "<<a.cbw/2;

cout<<"\n Providing the splay of 3:1 expansion length of expansion transition = "<<(a.cbw*3/4);
delay(5000);
cout<<"\n\n HEAD LOSS AND BED LEVELS AT DIFFERENT SECTIONS";//section 4.4

cout<<"\n\n\n At section 4-4, the canal returns to its normal section";

AR=(a.cbw+(1.5*a.cwd))*a.cwd;

cout<<"\n\n Area of the Trapezoidal canal section = "<<AR<<"sq.m";

v4=a.fsd/AR;

cout<<"\n Velocity = "<<v4<<"m/sec";

Vhead= v4*v4/(2*g);

rlb=a.cbl;

rlwc=a.cbl+a.cwd;
```

```
tel=rlwc+Vhead;

cout<<"\n R.L of bed at 4-4 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 4-4 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 4-4 = "<<tel<<"m";
delay(4000);

//section 3-3

cout<<"\n\n At section 3-3\n Bed width = "<<a.cbw/2<<"m";

AR=(a.cbw/2)*a.cwd;

v3=a.fsd/AR;

cout<<"\n Area of channel = "<<AR<<"sq.m";

cout<<"\n Velocity at 3-3 = "<<v3<<"m/sec";

Vhead=v3*v3/(2*g);

headloss=0.3*(v3+v4)*(v3-v4)/(2*g);

tel=tel+headloss;

rlwc=tel-Vhead;

rlb=rlwc-a.cwd;

cout<<"\n R.L of bed at 3-3 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 3-3 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 3-3 = " <<tel<<"m";
delay(5000);
//section 2-2

cout<<"\n\n At section 2-2\n Bed width = "<<a.cbw/2<<"m";

AR=(a.cbw/2)*a.cwd;

v2=v3;

cout<<" Area of channel = "<<AR<<"sq.m";
```

```
cout<<" Velocity at 2-2 = "<<v2<<"m/sec";

r=((a.cbw/2)*a.cwd)/(a.cbw/2+(2*a.cwd));

R=pow(r,4/3);

Vhead=v2*v2/(2*g);

headloss=0.016*0.016*v2*v2*W/R;

tel=tel+headloss;
rlwc=tel-Vhead;

rlb=rlwc-a.cwd;

cout<<"\n R.L of bed at 2-2 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 2-2 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 2-2 = "<<tel<<"m";
delay(10000);
//section 1-1

cout<<"\n\n At section 1-1";

v1=v4;

cout<<"Velocity at 1-1 = "<<v1<<"m/sec";

Vhead=(v1*v1)/(2*g);

headloss=0.2*(v2+v1)*(v2-v1)/(2*g);

tel=tel+headloss;

rlwc=tel-Vhead;

rlb=rlwc-a.cwd;

cout<<"\n R.L of bed at 1-1 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 1-1 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 1-1  = "<<tel<<"m";
delay(5000);
```

```
//design of transition

//contraction

cout<<"\n\n Contraction transition is given as :-";

for(i=0;i<=a.cbw/2;i+=2)

{

Bx=(a.cbw*a.cbw*a.cbw/4)/(a.cbw/2*a.cbw-(i*(a.cbw/2)));

cout<<"\n"<<i<<"\t"<<Bx;

}
//Expansion

cout<<"\n\n Expansion transition is given as :-";

for(i=0;i<=a.cbw*3/4;i+=2)
{

Bx=(a.cbw*a.cbw*a.cbw*3/8)/(a.cbw*3/4*a.cbw-(i*(a.cbw/2)));

cout<<"\n"<<i<<"\t"<<Bx;

}

}

//syphon aquaduct

void syaquaduct(canal1 a, drain1 d)
{
clrscr();
float av,vel,hob,bld,AR,p,W,v1,v2,v3,v4,Vhead,rlb,rlwc,tel,r,R,headloss,Bx;
int pl,i,b=0,z;

clrscr();

cout<<"\n\n Since the canal bed level"<<a.cbl<<"m is slightly below the H.F.L of
drainage"<<d.hfl<<"m an syphon aquaduct will be constructed";
cout<<"\n Enter length of span - ";
cin>>z;
      p=4.75 * sqrt(d.hfd);
```

```
for(i=1;i<=p&&b==0;i++)

{

if((1.5*i + (z * (i-1)))>=p)
{
pl=i;

b=1;
}
}
W=((pl-1)*z)+(pl*1.5);

cout<<"\n Total length of waterway = "<<W;
cout<<"\n Enter the value of velocity - ";
cin>>vel;
hob=d.hfd/(vel*pl*z);
av=d.hfd/(pl*z*hob);
cout<<"\n Actaul velocity through barrels = "<<av<<"m/s";
delay(1000);
cout<<"\n\n Bed width of canal = "<<a.cbw<<"m";

cout<<"\n Let the width to fumed to "<<a.cbw/2<<"m";

cout<<"\n Providing the splay of 2:1 contract. length of contract. transition = "<<a.cbw/2;

cout<<"\n Providing the splay of 3:1 expansion length of expansion transition = "<<(a.cbw*3/4);

cout<<"\n\n\n HEAD LOSS AND BED LEVELS AT DIFFERENT SECTIONS";//section 4.4
delay(1000);
cout<<"\n\n At section 4-4, the canal returns to its normal section";

AR=(a.cbw+(1.5*a.cwd))*a.cwd;

cout<<"\n\n Area of the Trapezoidal canal section = "<<AR<<"sq.m";

v4=a.fsd/AR;

cout<<"\n Velocity = "<<v4<<"m/sec";
```

```
Vhead= v4*v4/(2*g);

rlb=a.cbl;

rlwc=a.cbl+a.cwd;

tel=rlwc+Vhead;

cout<<"\n R.L of bed at 4-4 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 4-4 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 4-4 = " <<tel<<"m";

//section 3-3
delay(1000);
cout<<"\n\n At section 3-3\n Bed width = "<<a.cbw/2<<"m";

AR=(a.cbw/2)*a.cwd;

v3=a.fsd/AR;
cout<<"\n Area of channel = "<<AR<<"sq.m";

cout<<"\n Velocity at 3-3 = "<<v3<<"m/sec";

Vhead=v3*v3/(2*g);

headloss=0.3*(v3+v4)*(v3-v4)/(2*g);

tel=tel+headloss;

rlwc=tel-Vhead;

rlb=rlwc-a.cwd;

cout<<"\n R.L of bed at 3-3 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 3-3 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 3-3 = " <<tel<<"m";

//section 2-2
delay(1000);
cout<<"\n\n At section 2-2\n Bed width = "<<a.cbw/2<<"m";
```

```
AR=(a.cbw/2)*a.cwd;

v2=v3;

cout<<"Area of channel = "<<AR<<"sq.m";

cout<<"Velocity at 2-2 = "<<v2<<"m/sec";

r=AR/(a.cbw/2+(2*a.cwd));

R=pow(r,4/3);

Vhead=v2*v2/(2*g);

headloss=0.016*0.016*v2*v2*W/R;

tel=tel+headloss;

rlwc=tel-Vhead;

rlb=rlwc-a.cwd;

cout<<"\n R.L of bed at 2-2 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 2-2 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 2-2 = " <<tel<<"m";
delay(1000);
//section 1-1

cout<<"\n\n At section 1-1";

v1=v4;

cout<<"Velocity at 1-1 = "<<v1<<"m/sec";

Vhead=(v1*v1)/(2*g);

headloss=0.2*(v2+v1)*(v2-v1)/(2*g);

tel=tel+headloss;

rlwc=tel-Vhead;

rlb=rlwc-a.cwd;
```

```
cout<<"\n R.L of bed at 1-1 = "<<rlb<<"m";

cout<<"\n R.L of water surface at 1-1 = "<<rlwc<<"m";

cout<<"\n R.L of T.E.L at 1-1 = " <<tel<<"m";

//design of transition

//contraction

cout<<"\n\n Contraction transition is given as :- ";

for(i=0;i<=a.cbw/2;i+=2)

{

Bx=(a.cbw*a.cbw*a.cbw/4)/(a.cbw/2*a.cbw-(i*(a.cbw/2)));

cout<<"\n"<<i<<"\t"<<Bx;

}

//Expansion

cout<<"\n\n Expansion transition is given as :- ";

for(i=0;i<=a.cbw*3/4;i+=2)

{

Bx=(a.cbw*a.cbw*a.cbw*3/8)/(a.cbw*3/4*a.cbw-(i*(a.cbw/2)));

cout<<"\n"<<i<<"\t"<<Bx;

}
//design of trough
//    delay(10000);
float ow,bst,cw,iw,part,lsb;
cout<<"\n\n Enter the thickness of outer walls - ";
cin>>ow;
cout<<"\n Enter the thickness of inner walls - ";
cin>>iw;
cout<<"\n Enter the thickness of bottom slab - ";
cin>>bst;
cout<<"\n Enter total no. of partitions - ";
cin>>part;
```

```
part=part-2;
lsb=(a.cbw/2)+(2*ow)+(part*iw);
cout<<"\n Length of syphon barrel = "<<lsb<<"m";
//headloss throufg barrel
delay(1000);
float f1,f2,h,x,y,R1,ushfl;
cout<<"\n Enter the coeff. of loss of head at entry - ";
cin>>f1;
R1=hob*z/(2*(hob+z));
cout<<"R1 = "<<R1;
cout<<"\n Enter value of a and b(as per IS 7784)  - ";
cin>>x>>y;
f2=x*(1+y/R1);
cout<<"f2 = "<<f2;
h=(1+f1+f2*(lsb/R1))*(av*av/(2*g));
cout<<"\n Headloss = "<<h;
cout<<"\n d/s H.F.L = High Flood Level of Drainage = "<<d.hfl;
ushfl=d.hfl+h;
cout<<"\n The u/s H.F.L. = " <<ushfl;
// Uplift pressure on the roof of barrel
delay(1000);
float ed,msf,lshden,uproof,m1,m2,rlbf,rshead;
cout<< "\n R.L. of bottom of trough = "<<a.cbl-bst;
lshden= f1*av*av/(2*g);
cout<<"/n Loss of head at entry of barrel = " <<lshden;
uproof=(ushfl-lshden-(a.cbl-bst))*10;
cout<<"\n Uplift on the roof =" <<uproof<<" kN/sq.m";
cout<<"\n Assuming the unit weight of concrete = 24 kN/sq.m";
cout<<"\n Downward load of concrete slab = "<<ow*24<<"kN/sq.m";
cout<<"\n Balance uplift pressure = "<<uproof-(ow*24)<<"kN/sq.m";
cout<<"\n Downward water load acting = "<<a.cwd*10<<"kN/sq.m";
cout<<"\n Total downward load = "<<a.cwd*10+(ow*24)<<"kN/sq.m";
cout<<"\n Effective span = "<<((a.cbw/2)/(part+1))+iw<<"m";
m1=(a.cwd*10+(ow*24))*(((a.cbw/2)/(part+1))+iw)*(((a.cbw/2)/(part+1))+iw)/10;
cout<<"\n Maximum downward sagging bending moment = "<<m1<<"kN-m";
m2=(uproof-(ow*24))/10 * pow((((a.cbw/2)/(part+1))+iw),2);
cout<<"\n Maximum Hogging moment due to uplift = "<<m2<<"kN-m";
msf=(a.cwd*10+(ow*24))*((a.cbw/2)/(part+1))/2;
cout<<"\n Maximum shear force = "<<msf<<"kN";
ed=sqrt((m1*1000/87));
ed+=10;
cout<<"\n\n Using 1:2:4 cement concrete the effective depth is "<<ed<<"cm";
cout<<"\n Steel reqd. at bottom of slab = "<<m1/(0.12*0.87*ed)<<"sq.cm";
cout<<"\n Steel reqd. at the top of slab = "<<m2/(0.12*0.87*ed)<<"sq.cm";
//uplift on the bottom floor of barrel
x=a.cbl-bst;
```

```
delay(1000);
rlbf=x-hob;
cout<<"\n\n Static head on barrel floor = "<<rlbf<<"m";
cout<<"\n Assuming a thickness of 0.8m slab";
cout<<"\n R.L of bottom floor = "<<rlbf-0.8<<"m";
cout<<"\n The static uplift on floor = "<<d.bld-(rlbf-0.8)<<"m";
cout<<"\n Seapage head = "<<a.cbl+a.cwd-d.bld<<"m";
cout<<"\n Seapage line = abc and traverse total creep length is "<<(a.cbw)+(z/2)<<"m";
rshead = (a.cbl+a.cwd-d.bld)*(1-((a.cbw/2)+(z/2))/(a.cbw+(z/2)));
cout<<"\n Residual seapage head  = "<<rshead<<"m";
cout<<"\n Total uplift  = "<<((d.bld-(rlbf-0.8))+rshead)*10<<"kN/sq.m";
cout<<"\n Balance to be resisted by r/f due to bending action  = "<<(((d.bld-(rlbf-
0.8))+rshead)*10)-19.2<<"kN/sq.m";
delay(1000);
cout<<"\n\n\n\t\t Design of cutoffs and protection work for drainage floor";
float r9;
r9=3.59;
delay(1000);
cout<<"\n\n Depth of Scour = "<<r9<<"m";
cout<<"\n Depth of u/s cut-offs below H.F.L = "<<1.5*r9<<"m";
cout<<"\n Bottom of u/s cut-off  = "<<ushfl-(1.5*r9)<<"m";
cout<<"\n Bottom of d/s cutoff = "<<d.hfl-(1.5*r9)<<"m";
cout<<"\n Length of upstream protection = "<<2*(rlbf-(ushfl-(1.5*r9)))<<"m";
cout<<"\n Length of d/s pitching = "<<2*(d.bld-(d.hfl-(1.5*r9)))<<"m";

}
canal canalinp(float a)
{
canal abc;
cout<<"Enter the data for the canal ";
cout<<"\n\n\n\n\n \t\tEnter full supply discharge - ";
cin>>abc.fsd;

cout<<"\n\t\tEnter full supply level - ";
cin>>abc.fsl;

abc.cbl=a;

cout<<"\n\t\tEnter Canal bed width - ";
cin>>abc.cbw;

cout<<"\n\t\tEnter canal water depth - ";
cin>>abc.cwd;
return abc;
}
canal1 canal1inp(float a)
```

```
{
canal1 pqr; clrscr();
gotoxy(12,5);
cout<<"Enter the data for the canal  ";
cout<<"\n\n\n\n\n\n \t\tEnter full supply discharge - ";
cin>>pqr.fsd;

pqr.cbl=a;

cout<<"\n\t\tEnter Canal bed width - ";
cin>>pqr.cbw;

cout<<"\n\t\tEnter canal water depth - ";
cin>>pqr.cwd;
return pqr;
}
drain draininp(float b)
{
drain xyz;
clrscr();
gotoxy(12,5);

cout<<"Enter Data  For The Drain";
cout<<"\n\n\n\n\n\n\t\t Enter high flood discharge - ";
cin>>xyz.hfd;

xyz.hfl=b;
cout<<"\n\t\tEnter high flood depth - ";

cin>>xyz.hfdp;

cout<<"\n\t\tEnter general ground level - ";
cin>>xyz.ggl;
return xyz;
}
drain1 drain1inp(float b)
{
drain1 def;
clrscr();
gotoxy(12,5);

cout<<"Enter Data  For The Drain";
cout<<"\n\n\n\n\n\n\t\t Enter high flood discharge - ";
cin>>def.hfd;

def.hfl=b;
```

```
cout<<"\n\t\tEnter bed level of drainage - ";
cin>>def.bld;

cout<<"\n\t\tEnter general ground level - ";
cin>>def.ggl;
return def;
}
```

DESIGN PROBLEM - 1

2) Design a suitable cross drainage work, given the following data at the crossing of a canal and a drainage.

CANAL:

(1) Full supply discharge = 32 m^3/s
(2) Full supply level = R.L 213.5 m
(3) Canal bed level = RL 212 m
(4) Canal bed width = 20 m
(5) Trapezoidal canal section with 1.5H : 1V slopes
(6) Canal water depth = 1.5 m

DRAINAGE:

1) High flood discharge = 300 m^3/s
2) High flood level = 210 m
(3) High flood depth = 2.5 m
(4) General ground level = 212.5 m

Ans) As the drainage is of large size and canal bed level (212 m) is much above the H.F.L of drainage (210 m) an Aqueduct will be constructed. For effecting economy, the canal shall be flumed.

1) DESIGN OF DRAINAGE WATERWAY

According to Lacey's Regime Perimeter

$$P = 4.75\sqrt{Q}$$

$\therefore \quad P = 4.75\sqrt{300}$
$= 82.3m$.

Let the clear span between piers be 9m & pier thickness be 1.5m

Using 8 bays of 9m each, clear waterway $= 8 \times 9 = 72m$

using 7 piers of 1.5 each, length occupied by piers $= 7 \times 1.5$
$= 10.5m$

Total length of waterway $= 72 + 10.5$
$= 82.5m$.

2) DESIGN OF CANAL WATERWAY:

Bed width of Canal $= 20m$

Let the width to be flumed $= 10m$

Providing a splay of 2:1 in contraction, the length of transition

$= \frac{20-10}{2} \times 2 = 10m$

Providing a splay of 3:1 in expansion, the length of transition

$= \frac{20-10}{2} \times 3 = 15m$

Length to be flumed of rectangular canal portion between abutments $= 82.5m$

3) HEAD LOSS AND BED LEVEL AT SECTIONS:

> SECTION 4-4

At Section 4-4, where the Canal returns to its normal section, Area of trapezoidal Canal section

$= (B + 1.5y)\,y$

$= (20 + 1.5 \times 1.5)\,1.5$

$= 33.75\,m^2$

$$\text{Velocity} = V_4 = \frac{Q}{A} = \frac{32}{33.75} = 0.947 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.947)^2}{2\times 9.81} = 0.046\text{m}$$

R.L of bed at 4-4 = 212.0 (given)

R.L of water surface at 4-4 = 212 + 1.5 = 213.5 m

R.L of T.E.L = 213.5 + 0.046 = 213.546 m

AT SECTION 3-3

Bed width = 10 m

Area of channel = 10 × 1.5
= 15 m²

$$\text{Velocity} = V_3 = \frac{32}{15} = 2.13 \text{ m/s}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.13)^2}{2\times 9.81} = 0.232\text{m}$$

Assuming loss of head in expansion from 3-3 to 4-4 is taken as

$$= 0.3\left[\frac{V_3^2 - V_4^2}{2g}\right]$$

$$= 0.3\,[0.232 - 0.046]$$

$$= 0.0558\text{m}$$

$$= 0.056\text{m}$$

R.L of T.E.L at section 3-3 = R.L of T.E.L at 4-4 + loss in expansion

= 213.546 + 0.056

= 213.602 m

∴ R.L of water surface at 3-3 = R.L of T.E.L at 3-3 − Velocity Head

= 213.602 − 0.232

= 213.307 m

R.L of bed at 3-3 = 213.370 − 1.5
= 211.87 m

> SECTION 2-2

From section 2-2 to 3-3, the trough is constant, so Area & velocity is same as that of 3-3 but friction loss between 2-2 & 3-3 occur which can be computed by the

Mannings Formula $H_L = \dfrac{n^2 . V^2 . L}{R^{4/3}}$

n = Rugosity coeff = 0.016 for concrete trough

L = Length of trough

$A = 10 \times 1.5 = 15\ m^2$

P = wetted Perimeter $= 10 + 2 \times 1.5 = 13m$

R = Hydraulic mean depth $= \dfrac{A}{P} = \dfrac{15}{13} = 1.16m$

V = Actual velocity in trough $= \dfrac{Q}{A} = \dfrac{32}{15} = 2.13\ m/sec$

$$\therefore\ H_L = \frac{(0.016)^2 \times (2.13^2) \times 82.5}{(1.16)^{4/3}} = 0.0787\ m$$
$$= 0.079m$$

R.L of TEL at 2-2 = R.L of TEL at 3-3 + Friction Loss
= 213.602 + 0.079
= 213.681 m

R.L of water surface = R.L of T.E.L at 2-2 − Velocity Head
= 213.681 − 0.232
= 213.449 m.

R.L of bed at 2-2 = 213.449 − 1.5
= 211.949 m.

> SECTION 1-1

Loss of head in contraction from 1-1 to 2-2

$$= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$= 0.2\left[\frac{(2.13)^2 - (0.947)^2}{2\times 9.81}\right]$$

$$= 0.2\,[0.232 - 0.046]$$

$$= 0.037\,m$$

R.L of T.E.L at 1-1 = R.L of T.E.L at 2-2 + Loss in contraction

= 213.681 + 0.037

= 213.7180 m

R L of water surface at 1-1 = 213.718 − 0.046

= 213.672 m.

R.L of bed at 1-1 = 213.672 − 1.5

= 212.172.

4) DESIGN OF TRANSITIONS :

(a) CONTRACTION TRANSITION : following Mitra's Hyperbolic Method

$$B_x = \frac{B_n \cdot B_f L_f}{L_f B_n - x(B_n - B_f)}$$

B_n = Bed width of normal channel

B_f = Bed width of flumed channel

B_x = Bed width at any distance x

L_f = Length of transition

$B_f = 10m$ $B_n = 20m$ $L_f = 10M$.

$$\therefore B_x = \frac{20\times 10\times 10}{10\times 20 - x(20-10)} = \frac{2000}{200 - 100x}.$$

For various values of x lying between 0 to 10 m, the respective B_x values have been worked out as shown in the table below. The distance x is measured from the flumed section 2-2.

x (m)	0	2	4	6	8	10	
$B_x = \frac{2000}{200-10x}$	10	11.11	12.15	14.29	16.67	20.0	

> EXPANSION TRANSITION

$B_n = 20$ m

$B_f = 10$ m

$L_f = 15$ m

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f \cdot B_n - x(B_n - B_f)}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x(20-10)}$$

$$= \frac{3000}{300 - 10x}.$$

The values of x lying between 0 to 15 are plotted in the table below

x (m)	0	2	4	6	8	10	12	14	15
$B_x = \frac{3000}{300-10x}$	10	10.71	11.54	12.5	13.64	15	16.67	18.75	20

(5) DESIGN OF TROUGH : - The trough shall be divided into two equal compartments of 5m each. seperated by inner walls of 0.3m thickness. The inspection road shall be carried on the top left compartment.

A freeboard of 0.6m above normal water depth of 1.5m is kept over trough and thus the bottom level of bridge slab is

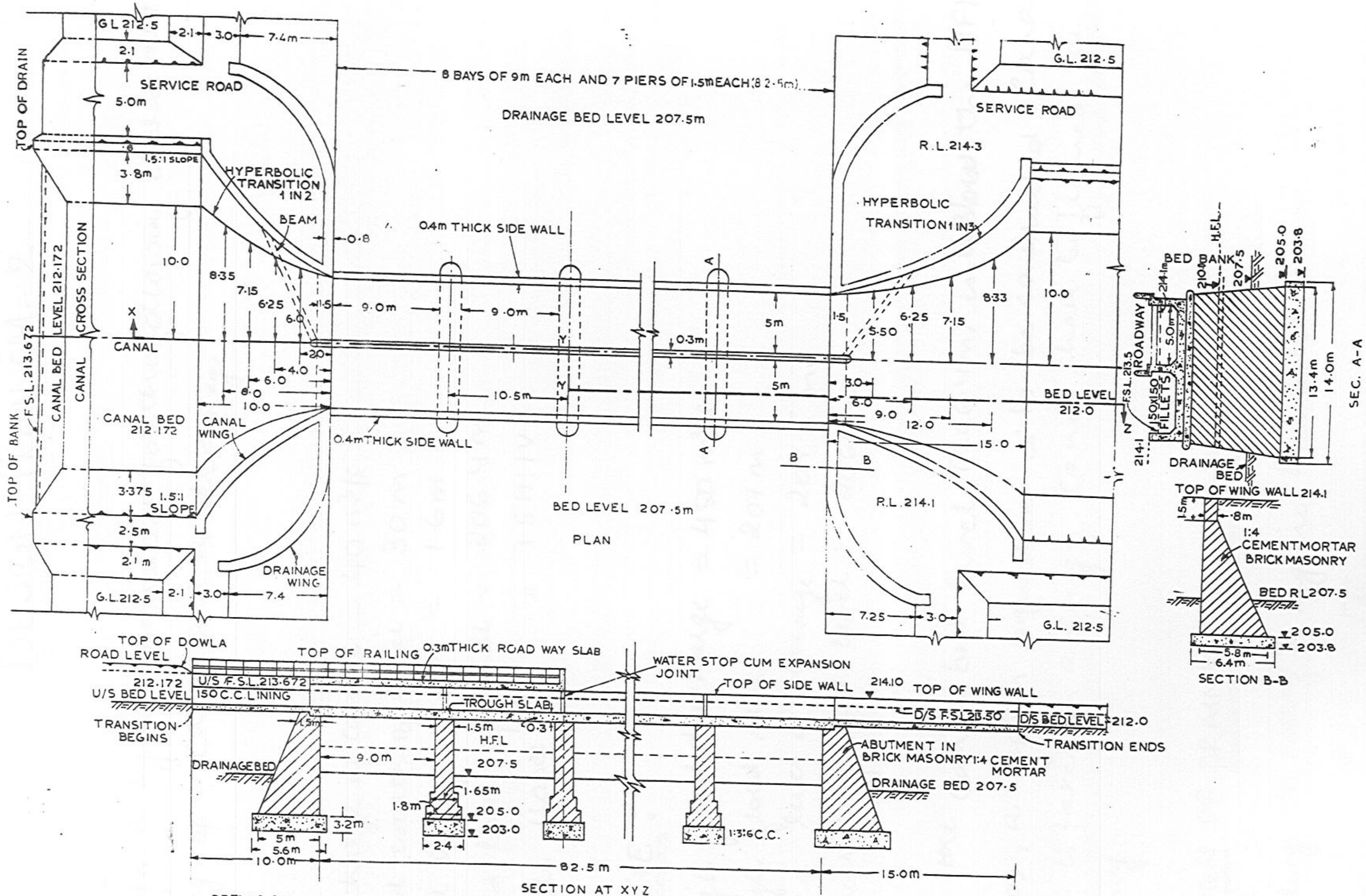
$1.5+0.6=2.1m$ the bed level of trough.

Hence height of trough = 2.1m.

Entire trough is a concrete monolith of RCC

Outer walls = 0.4m thick

Bottom slab = 0.4m thick



DETAILS OF THE AQUEDUCT (FIG. 14.24.)

DESIGN PROBLEM-2

1) Design a Syphon Aqueduct for the following data at the crossing of a canal and a drainage.

CANAL:

(1) Discharge of canal = 40 m^3/s
(2) Bed width of canal = 30 m
(3) Full supply depth = 1.6 m
(4) Bed level of canal = 206.4 m
(5) Side slopes = 1.5H:1V

DRAINAGE:

(1) High flood discharge = 450 m^3/s
(2) High flood level = 207 m
(3) Bed level of drainage = 204.5 m
(4) General ground level = 206.5 m.

Sol) As the canal bed level (206.4m) is below the HFL (207.0), a syphon Aqueduct is to be constructed. Canal water is taken in a trough. Canal shall be flumed for economy.

① DESIGN OF DRAINAGE WATERWAY:

According to Lacey's Regime Perimeter

$$P = 4.75\sqrt{Q}$$
$$= 4.75\sqrt{450}$$
$$= 100.8 \text{ m}$$

Providing 11 clear spans of 8 m each with pier thickness = 1.5m.

$\therefore$ The length occupied by 11 bays of 8m = 11X8 = 88m

Length occupied by 10 piers of 1.5 m = 10x1.5 = 15 m

$\therefore$ Total length of waterway = 88+15 = 103m.

According to IS-7784, the limiting velocity through syphon cement barrel = 2m/sec.

$$\therefore \text{Height of barrels required} = \frac{Q}{\text{Velocity} \times \text{clear width of waterway}}$$

$$= \frac{450}{2 \times 88}$$

$$= 2.56 m.$$

Hence provide 11 rectangular barrels of 8m each and a height of 2.5 m.

$$\therefore \text{Actual velocity through barrel} = \frac{450}{11 \times 8 \times 2.5} = 2.05 \, m/s.$$

(2) DESIGN OF CANAL WATERWAY:

Normal bed width of canal = 30 m

Let width to be flumed = 15 m.

Providing a splay of 2:1 in contraction, then the length of contraction = $\frac{30-15}{2} \times 2$ = 15m

Providing a splay of 3:1 in expansion, the length of the transition is $= \frac{30-15}{2} \times 3$

$= 22.5m.$

(3) DESIGN OF BED LEVELS AT DIFFERENT SECTIONS:

> SECTION 4-4

Area of trapezoidal canal section $= (B+1.5y)y$

$= (30+1.5\times1.6)1.6$

$= 51.84\ m^2$

Velocity of flow $= V_4 = \frac{Q}{A} = \frac{40}{51.84} = 0.77 m/s$

Velocity head $= \frac{V_4^2}{2g} = \frac{(0.77)^2}{2\times9.81} = 0.030 m$

R.L of canal bed at 4-4 $= 206.4$ m (given)

R.L of water surface " $= 206.4 + 1.6$

$= 208.0 m$

R.L of T.E.L at 4-4 $= 208.0 + 0.03$

$= 208.03 m.$

> SECTION 3-3

Assuming constant depth of 1.6m throughout the canal at section 3-3.

Bed width $= 15m$.

Depth $= 1.6 m$

Area $= 15\times1.6 = 24 m^2$

$$\text{Velocity} = V_3 = \frac{Q}{A} = \frac{40}{24} = 1.67 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(1.67)^2}{2 \times 9.81} = 0.142 \text{m}$$

Assuming loss of head in expansion is taken as the following between 3-3 & 4-4.

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g} \right]$$

$$= 0.3 \,[0.142 - 0.030] = 0.3 \times 0.112$$

$$= 0.0336 \text{m}$$

$$\cong 0.034 \text{m}.$$

R.L of T.E.L at 3-3 = R.L of T.E.L + loss in expansion

= 208.03 + 0.034

= 208.064 m

R.L of water surface at 3-3 = 208.064 − 0.142

= 207.922 m

R.L of bed at 3-3 = 207.922 − 1.6

= 206.322 m.

> <u>SECTION</u> <u>2-2</u> – Trough is constant throughout section 2-2 & 3-3 so velocity & Area is same as that of 3-3.

The friction loss is computed as per Manning's Formula

$$H_L = \frac{n^2 . V^2 . L}{R^{4/3}}.$$

where n = rugosity coefficient = 0.016 as per IS:456

L = length of channel = 103m

A = Area of trough = 15×1.6 = 24 m²

P = wetted Perimeter = 15+2×1.6 = 18.2m

R = Hydraulic mean depth $= \frac{A}{P} = \frac{24}{18.2} = 1.32m$

Velocity = velocity in trough $= \frac{Q}{A} = \frac{40}{24} = 1.67\ m/s$

$$\therefore H_L = \frac{(0.016)^2 \times (1.67)^2 \times 103}{(1.32)^{4/3}} = 0.051m$$

R.L of T.E.L at 2-2 = R.L of T.E.L at 3-3 + H_L

= 208.064 + 0.051

= 208.115m

R.L of water water surface = 208.115 − 0.142

= 207.973m

R.L of bed at 2-2 = 207.973 − 1.6

= 206.373m

> SECTION 1-1

Loss in head due to contraction between 2-2 & 1-1

$$= 0.2\left[\frac{V_2^2 - V_1^2}{2g}\right] = 0.2\left[\frac{(1.67)^2 - (0.77)^2}{9.81 \times 2}\right]$$

$$= 0.022m.$$

R.L of T.E.L at 1-1 = R.L of T.E.L + Head loss

= 208.115 + 0.022

= 208.137m

R.L of water surface at 1-1 $= 208.137 - 0.030$
$= 208.107 m$

R.L of bed at 1-1 $= 208.107 - 1.6$
$= 206.507 m$.

(4) DESIGN OF TRANSITIONS:

> CONTRACTION TRANSITION - By Mitra's Hyperbolic method

$$B_x = \frac{B_n \cdot L_f \cdot B_f}{B_n \cdot L_f - x(B_n - B_f)}$$

$B_f = 15 m$
$B_n = 30 m$
$L_f = 15 m$

$$= \frac{30 \times 15 \times 15}{30 \times 15 - x(30-15)}$$

$$= \frac{450}{30 - x}$$

For values of x lying between 0-15 m, Bx values are plotted in the table below.

x (m)	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30-x}$	15	16.04	17.27	18.72	20.42	22.5	25	28.1	30

> EXPANSION IN TRANSITION :

$B_n = 30$ m

$L_f = 22.5$ m

$B_f = 15$ m $\quad \therefore B_x = \dfrac{B_n \cdot L_f \cdot B_f}{B_n \cdot L_f - x(B_n - B_f)}$

$$= \frac{30 \times 15 \times 22.5}{30 \times 22.5 - x(30-15)}$$

$$= \frac{675}{45 - x}$$

For values of x lying between 0-22.5m, corresponding Bx values are plotted.

x(m)	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45-x}$	15	15.7	16.46	17.3	18.25	19.3	20.4	21.75	23.3	25	27	30

5) DESIGN OF TROUGH : - The trough shall be devided into 3 equal parts of 5 m width, seperated by 0.3 m thick partition walls. A free board of 0.6m above normal water depth is enough. The bottom level of bridge slab is kept at 1.6+0.6 = 2.2m above bed level of trough.

Outer walls = 0.4m thick

Bottom slab = 0.4m thick

Trough is a monolith structure of RCC

The intermediate wall shall be extended into transitions, to provide a clear width of 15m.

$\therefore$ Length of syphon barrel $= 15 + 2 \times 0.3 + 2 \times 0.4$
$= 16.4m.$

(6) HEAD LOSS THROUGH SYPHON BARRELS:

Using Unwin's Formula.

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g}$$

where V = velocity in barrels $= 2.05 m/s$

f_1 = coeff of loss of head at entry.
$= 0.505$ for unshaped mouth.

$f_2 = a\left(1 + \frac{b}{R}\right)$

a & b are taken from table for plastered cement barrels.

$a = 0.00316$

$b = 0.030$

R = Hydraulic mean depth
$= \frac{A}{P} = \frac{8 \times 2.5}{2(8+2.5)} = 0.953$

L = Length of barrel $= 16.4m$

$$f_2 = 0.00316\left[1 + \frac{0.030}{0.953}\right]$$
$$= 0.00326$$

$$\therefore h = \left[1 + 0.505 + 0.00326\left[\frac{16.4}{0.953}\right]\right] \frac{\times (2.05)^2}{2 \times 9.81}$$
$$= 0.333m$$

H.F.L $= 207m$ (given).

$\therefore$ d/s H.F.L $= 207.0m$

Afflux (h) $= 0.333m$.

$\therefore$ u/s H.F.L $= 207 + 0.333$
$= 207.333m.$

7) UPLIFT PRESSURE ON ROOF OF BARRELS

R.L of bottom of trough = R.L of canal bed − Slab Thickness
$= 206.4m - 0.4$
$= 206m$

Loss of head at entry to barrel $= 0.505 \dfrac{V^2}{2g}$

$= 0.505 \times \dfrac{(2.05)^2}{2 \times 9.81}$

$= 0.108 \, m$

Uplift on roof = U/S H.F.L − Loss at entry − R.L of trough bottom

$= 207.333 - 0.108 - 206$

$= 1.225$ m of water

Unit wt of water $= 10 \, kN/m^3$.

$\therefore$ uplift on roof $= 12.25 \, kN/m^2$

Downward load exerted by concrete trough slab of 0.4m

$= 0.4 \times$ unit wt of concrete

$= 0.4 \times 24$

$= 9.6 \, kN/m^2$

$\therefore$ The balanced uplift pressure $= 12.25 - 9.6$
$= 2.65 \, kN/m^2$

The balanced uplift pressure is to be resisted by R/F to be provided on top in roof of slab. The roof slab has to be designed for full canal water load (1.6m of water) + self wt when drainage water is low & not exerting any uplift. R/F for bottom slab is worked as below.

8) <u>DESIGN</u> OF <u>ROOF</u> OF <u>BARREL</u> :

Uplift to be balanced by top R/F $= 2.65 \, kN/m^2$

Downward load of water when no uplift $= 1.6 \times 10 = 16 \, kN/m^2$
16

Load due to self wt of slab $= 0.4 \times 24$
$= 9.6 kN/m^2$

Total downward load $= 9.6 + 16$
$= 25.6 kN/m^2$

An inner wall of 0.3m thickness has been provided of clear span 5m. Effective span $= 5 + 0.3 = 5.3m$.

Considering 1m wide strip of slab.

Max. Sagging BM in slab due to downward loads

$$= \frac{wL^2}{10} = \frac{25.6 \times (5.3)^2}{10} = 71.7 \text{ kN-m}$$
$$= 71.7 \times 10^5 \text{ N.cm}$$

Max. Hogging B.M due to residual uplift

$$= \frac{w_2 l^3}{10} = \frac{2.65 \times (5.3)^3}{10} = 7.42 \text{ kN-m}$$
$$= 7.42 \times 10^5 \text{ N.cm.}$$

Max shear force $= \frac{w_1 l}{2} = \frac{25.6 \times 5}{2} = 64 kN$

using 1:2:4 cement concrete.

Effective depth $d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{71.7 \times 10^5}{8.7 \times 100}} = 28.8 cm.$

Provided overall thickness $= 40 cm$ & effective depth $= 37.5 cm$.

Steel required in bottom of slab $= \frac{71.7 \times 10^5}{12000 \times 0.87 \times 37.5} cm^2$
$= 18.3 cm^2$

Provide 16mm ϕ bars @ 10cm c/c in bottom of slab.

Steel required at top $= \frac{7.42 \times 10^5}{12000 \times 0.87 \times 37.5}$

$= 1.9 \; cm^2$.

Provide 10mm ϕ bars @ 15cm c/c.

7) UPLIFT ON THE BOTTOM FLOOR OF BARREL

a. STATIC HEAD: R.L of barrel floor = R.L of trough bottom − Height of barrel

$= 206.0 - 2.5$

$= 203.5$ m

Assuming a thickness of 0.8m is provided.

∴ R.L of bottom of floor $= 203.5 - 0.8 = 202.7$m

Bed level of drain $= 204.5$ m.

Assuming bed level has gone up to water table, the static uplift on the floor

$= 204.5 - 202.7 = 1.8$m of water.

b. SEEPAGE HEAD: − This will be maximum when the canal is running full & drain is dry.

Total seepage head = F.S.L of canal − bed level of drain

$= 208.0 - 204.5$

$= 3.5$m.

The residue seepage at point 'a' in barrel as calculate by Bligh's theory

Assuming the total length of drainage floor = 30m

The seepage line abc will traverse creep lengths as follows:

ab = Length of U/s transition + Half of barrel span
$= 15+4 = 19m$.

bc = 15m (Half of total length of 30m = assumed)

Total creep length $= 19+15 = 34m$.

Residual seepage head at $b = 3.5\left[1 - \frac{19}{34}\right]$
$= 1.55m$.

Total uplift = static head + seepage head
$= 1.8 + 1.55 = 3.35$ m of water
$= 33.5\ kN/m^2$

The provided 0.8m thickness of slab will resist due to its own weight, an uplift $= 0.8 \times 24 = 19.2\ kN/m^2$

∴ Balance to be resisted by R/F due to bending
$= 33.5 - 19.2 = 14.3\ kN/m^2$

(10) DESIGN OF CUTOFFS AND PROTECTION WORK FOR DRAIN FLOOR

The scour depth $R = 0.47\left[\frac{Q}{f}\right]^{1/3}$ assuming $f = 1.0$.
$= 0.47\left[\frac{450}{1}\right]^{1/3}$
$= 3.59m$.

Provide depth of cutoff for scour hole of 1.5 R on both sides.

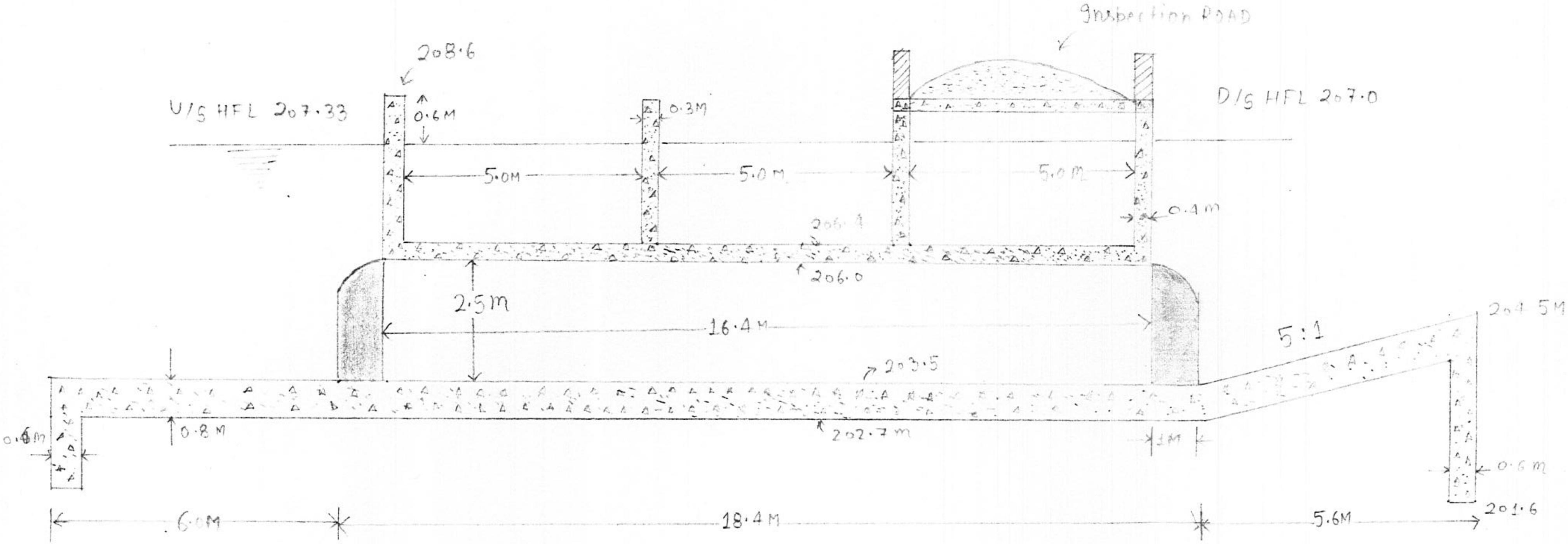
Depth of U/s cut off below H.F.L $= 1.5R$
$= 1.5 \times 3.59$
$= 5.4m$

R.L of bottom of U/S cutoff $= U/S\ H.F.L - 5.4$
$= 207.333 - 5.4$
$= 201.93\,m$

R.L of bottom of d/s cutoff $= d/s\ H.F.L - 5.4$
$= 207 - 5.4$
$= 201.6\,m$

Length of U/S protection = 2 [R.L of U/S bed − R.L of bottom of U/S cutoff]
$= 2\,[203.5 - 201.93]$
$= 2 \times 1.57 = 3.14\,m$

Length of d/s protection = 2 [R.L d/s bed − R.L of bottom of d/s cutoff]
$= 2\,[204.5 - 201.6]$
$= 5.8\,m$



SECTION of SYPHON AQUEDUCT