HYDRAULIC DESIGN OF CIVIL STRUCTURES OF SMALL HYDRO-PLANTS WITH SPECIAL REFERENCE TO POWER PLANTS IN UTTRANCHAL

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I do hereby declare that the work, which is being presented in this dissertation titled **"HYDRAULIC DESIGN OF CIVIL STRUCTURES OF SMALL HYDRO-PLANTS WITH SPECIAL REFERENCE TO POWER PLANTS IN UTTRANCHAL"** in partial fulfillment of the requirement for the award of the Degree of **MASTER OF TECHNOLOGY** in **WATER RESOURCES DEVELOPMENT (CIVIL)**, at the Department of Water Resources Development Training Centre, Indian Institute of Technology, Roorkee, is an authentic record of my own work carried out during the period from July 2002 to February 2003 under the guidance of **Dr. B.N. Asthana** Emeritus Fellow, **Dr. Rampal Singh,** Professor, **Prof. Devadatta Das**, Professor, Water Resources Development Training Centre, Indian Institute of Technology, Roorkee, Roorkee.

The matter presented in this dissertation has not been submitted by me for the award of any other degree.

Dated: February 17, 2003
Place: Roorkee

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This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

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SYNOPSIS

India's hydropower potential is of the order 84044MW at 60% load factor. Out of this about 24000 MW has been harnessed so far. Because of high capital cost long gestation period and various environmental and rehabilitation issues involved in developing large hydro emphasis has been shifted to development of small hydro as an alternative. Small hydro plant offers several advantages in today's energy development scenario. It has little or no adverse environmental impact on ecology. Ideally a small hydro may serve other purposes in addition to generation of power such as water supply, irrigation and recreation to the people living in rural areas. The population in rural areas particularly in the hilly regions of The Himalayas are in need of electrical energy for their sustainability and there is no better option other than small hydro which is economically viable. India has a sizable potential for establishing small hydro stations of the order 10,000 to 15000 MW out of which more than 50% exists in hilly regions of Himalayas and about 5000 MW can be harnessed from existing canal falls .The small hydropower technology is proven and simple. It involves relatively simple and small civil structures for harnessing small power generation. In order to make these schemes techno-economically viable the design of civil structures should be such that the total cost of power plant is less. The power generation and control equipments should also be standardized to make the schemes less costs.

Keeping economic considerations in view elaborate works of diversion structures, desilting arrangements, water conductor systems etc are not advisable in small hydropower projects. So in this study an attempt has been made to develop cost-effective hydraulic design of civil structures involved in small hydro plants. To achieve this aim prevailing design practices have been critically studied/reviewed with reference to the proposed power plant on river Asiganga a tributary of Bhagirathi in Uttarkashi District of Uttaranchal state.

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CHAPTER 1
INTRODUCTION

1.1 GENERAL

The world is currently in the throes of a recession and energy crisis. The depletion of non-renewable energy resources reorients our perception and research towards the developing of alternate and renewable sources of energy. Most of the conventional energy resources come from the natural reserves of fossil fuels like coal, oil, and gas. The demand of energy for human developmental activities result an over exploitation of those natural resources and that makes an adumbration of their early exhaustion. The impacts of energy crisis are more serious for less developed or developing countries, where an ever-increasing percentage of national budgets earmarked for the purchase of petroleum product or fossil fuels. To reduce this dependency on imported fuels with high price volatility, most countries have their effort to develop the alternative sources of energy based on domestic renewable resources. Out of different options development of small hydro is most viable and adorable one in the political, economic as well as environmental front. The exploitation of this primordial and renewable energy for human development is not only the result of present crisis but also has its own story of historical perspective. The use of running water for mechanical power such as milling, pumping, agriculture and other functions are narrated in the history of early civilization. The water wheels of various types are still used for the above purposes in many countries in village and remote areas, as a source of economical and low-level waterpower for human sustenance. The advent of industrial revolution transfers these low level systems to the present day's sophisticated hydroelectric generating plants. The ability of hydroelectric plants to provide electricity for the larger population centers was recognised as early as 18^{th} centuries but could be not succeeded due to poor transmission technology. The development of high voltage transmission line in the early part of twentieth century made a shift from small hydro plants serving the local electricity markets to large-scale plants feeding into extensive distribution grids. The drastic hike in petroleum prices and the economies of scale in the current century, requirement of attention towards the more lucrative and least environmental damaging scheme. Hence once again the SHP schemes become popular and more relevant in the era of environmental awareness and privatisation. Besides other benefits small hydro in particular caters the energy needs and socio-economic upliftment of far flung, scattered rural areas away from grid and as such has a greater role in the scenario of rural development. Although the adoption of small hydro is the call of the day, its increased capital cost per KW power generation make them economically unviable in the first sight. So the reduction of capital cost from different components of the scheme is of vital importance to gear up it's developmental prospective. The civil components among them have a scope to reduce its cost by adaptation of different scientific technique and readily locally available low cost construction material. Visualising these aspects a detailed study is made on the hydraulic design of civil structures in small hydros and their optimisation and standardisation.

1.2 DEFINITION AND CLASSIFICATION OF SMALL HYDRO ELECTRIC PROJECT

The definition of small hydro is somewhat arbitrary from technical standpoint. It has a wide range in usage, covering schemes having installed capacities from a few KWs to 50,000 KWs. In India according to the classification of Central Electricity Authority (CEA) and recent legislation of Government of India, the small hydro projects (SHP) are defined as plants of 25MW or less capacity. This capacity limit has been adopted in view of special handling in licensing, loans, incentives and other supports for infrastructure development of state and local programs and above all attracting private sector participation in power development, keeping in tune with the Government's overall thrust on liberalization of economy.

The SHP schemes are further classified into three categories based on capacity of individual machines as well as total installed capacity [Table.1.1].

Table 1.1: Classification of SHP Schemes According to Plant Capacity

SL.NO.	HYDRO ELECTRIC SCHEME	INDIVIDUAL MACHINE CAPACITY	TOTAL PLANT CAPACITY
1	Micro Hydro Electric Scheme	A few KW - 100 KW	Upto 100 KW
2	Mini Hydro Electric Scheme	101 KW – 1000 KW	101 KW - 3000 KW
3	Small Hydro Electric Schemes	1001 KW – 5000 KW	3001 KW - 25,000 KW

From technical point of view the small hydro schemes can be classified into three categories by available head (Difference in elevation between plant intake and discharge) for power generation. Each of these categories requires a different approach of design. According to A. Ludin [33] and other authors the general classification that has been adopted based on arrangement of the plant, hydraulics and foundation works is given in Table 1.2. These suggested limits are not rigid but are merely a means of categorizing sites.

Table 1.2: Classification of SHP According to Available Head

SL.NO.	CLASSIFICATION	HEAD
1	Low Head Plant	H < 15m
2	Medium Head Plant	H = 15m to 50m
3	High Head Plant	H > 50m

Note: H = Available head for power generation.

A more realistic classification can be made keeping in view the available head for the scheme below

Canal Drop Schemes: The schemes are characterised by low heads. Here the drop of the existing canal along its course is utilized for power generation.

Dam-toe-Schemes: These are characterised by low and medium heads. Here the power plant is built just at the toe of a dam.

Schemes in hilly and remote areas: These schemes are characterised by high heads and small discharges and considered as a scaled down version of any typical major hydroelectric installation.

The type of schemes and considerations that weigh in planning of SHP schemes close to the grid and in the remote areas are very different. Hence on these considerations the SHP schemes may be classified differently for planning purposes such as

(i) Schemes connected to the existing grid.

(ii) Schemes serving isolated and remote hilly areas.

1.3 The Promises of Small Hydropower Scheme

Small hydropower is the most promising among the other sources of renewable energy such as solar, biogas, biomass gasifier, wind, ocean, hydrogen, and geothermal etc. Although the use and knowledge of waterpower, as a source of conventional energy is as primitive as human civilization, but its abundant existence at different sphere of natural and man made landscape has made it more appropriate to define as a nonconventional source of energy. It is a clean and environmentally benign source of energy when utilized for conversion of its potential energy to mechanical and in turn electrical energy. The small hydropower popularly known as SHP provides electricity essential to economic and social development. The importance of SHP increases manifold particularly in the environment of high rate of inflation, cost of maintenance in other sources of power plants, cost of fossil fuel in thermal plants, environmental and political concerns. In one time planner's perception of large hydro-elelectric scheme with the advent of electrical transmission technology was supposed to be the best option during the peak demand along with base load thermal power plants.

But due to the growing environmental concern, high cost of power transmission, high construction cost, long delays in planning, construction and operation of the plant, reduces the attractiveness of large-scale hydro projects. On the other hand the recent technical advances in water turbine design, construction and efficiency, combined with increasing petroleum fuel costs have enhanced the competitive position of SHP in relation to diesel-powered generators of similar size. Unlike large hydro, SHP can be planned and built in less time. They are reliable and, within the limits of the water resources available, can be tailored to the needs of the end user market. Due to its lower investment, less gestation period along with quick return, the financing is often easier to obtain, hence encourage private sector participation. A comparison of benefits of small hydro and large hydro, and economics of various conventional and non-conventional sources of energy are given in Table 1.3 & 1.4, which evaluate the relative importance of SHP [36].

Ellis Armstrong [21], former Head of the U.S.Bureau of reclamation, enumerated the advantages of small hydropower in the following ways. Small Hydro:

1) is a non-consumptive generator of electrical energy, utilizing a renewable resource which is made continually available through the hydrologic cycle by the energy of the sun.

2) is essentially non-polluting and releases no heat. Adverse environmental impacts are negligible, and, for small installations, may be totally eliminated.

3) plants can be designed and built within one or two years time. Licensing requirements are minimal, equipments are readily available, and construction procedures are well known.

Table 1.3: Comparison of Small Hydro and Large Hydro Schemes.

SL.NO.	ITEMS	SMALL HYDEL	LARGE HYDEL
1	Construction period	2 to 3 years	5 to 10 years
2	Operation	Less hazardous and environment friendly	Require careful reservoir operation and mass scale rehabilitation during construction
3	Stability of power generation supply	Stable generation due to less silting and short water conductor system	Stability is disrupted without storage schemes
4	Types of works and construction techniques	Simple	Sophisticated construction techniques for tunneling, barrage etc.
5	Development of resources/ men/ materials/ machinery	Labour oriented	Machine oriented
6	Power supply to remote areas	Suitable to perennial stream available in the area	Difficult to maintain power and higher cost for longer transmission line
7	Cost per KW of installed capacity	Much higher	Much less
8	Operation and maintenance	This can be done by less number of trained personnel / local people / beneficiaries	Skilled personnel are required for O&M. Establishment cost is high.

SL.NO.	ITEM OF COMPARISON	MAJOR HYDROPOWER	THERMAL POWER	SMALL HYDROPOWER	WIND POWER	SOLAR PHOTO-VOLTAIC	BIOMASS GASIFIER	BIOGAS
1	Cost per KW installed (Rs)	12,000-15,000	15,000-18,000	25,000-35,000	16,000-18,000	2,33,000	9,000	23,100
2	Assessed life in years	40	40	40	20	20	8	10
3	Annual generation (KWh) per KW installation	3000	4000	4500	1000-1200	720	2500	2500
4	Annual operation & Maintenance per KW (% of capital investment)	13.6	15.6	3	3	3	31	8
5	Rs/KWh generated with 10%interest on capital	0.54/.58	0.58-0.7	0.87-1.2	2.04-3.24	58	1.72	1.74
6	Degree of reliability	Proven and time tested	Proven	Proven	Proven	Yet to be proved	Yet to be proved	Yet to be proved
7	Energy conversion efficiency	High	High	High	Low	Low	Low	Low
8	Technical concept	Simple & indigenous	Simple & indigenous	Simple & indigenous	Not simple	Not simple	Not simple	Not simple
9	Predictability	Certain	Certain	Certain	Uncertain	Uncertain	Uncertain	Uncertain
10	Input	Potential energy of water	latent heat of fuel	Potential energy of water	Kinetic energy of wind	Solar radiation	Wood	Animal dung
11	Output	Electrical energy	Electrical energy	Electrical energy	Electrical energy	Electrical energy	Electrical energy	Electrical energy

(**Source:** **Indian Journal of Power and River Valley Development, July-August 1991, No. 748.**)

TABLE.1.4:Comparative Economics Of Various Conventional And Non-Conventional Sources Of Energy

4) in remote areas using relatively simple technology can be a catalyst in mobilizing productive resources and creating enhanced economic opportunities for local residents.

5) is characterized by reliability and flexibility of operation, including fast start up and shut-down in response to rapid changes in demand. It thus becomes a valuable part of any large electrical system, increasing over all economy, efficiency, and reliability.

6) Technology is well developed and proven, with turbine efficiencies running as high as 90 percent. Small units ranging from a few kilowatts to several megawatts have been in operation since the turn of the 20^{th} century.

7) facilities have a long life. As a rule, dams and control works perform for a century or more with little maintenance.

8) requires few operating personnel. Some small- scale installations are operated entirely by remote control.

9) development can make maximum use of local materials and labor. When compared to thermal facilities, small hydros usually provide more local employment in the construction of civil works.

10) economic feasibility is improving when compared to other energy sources that use fossil fuels. With more realistic methodologies for economic evaluation, including full recognition of the value of non-consumptive water use, freedom from fuel dependence and minimum environmental impact, small hydropower has become increasingly desirable.

11) potential in the industrialized countries can be developed to augment hydropower capacity at existing powerhouses and dams. The possibility of retrofits and additional turbines and generators makes the upgrading of present installations attractive.

Apart from these benefits some more benefits particularly true to underdeveloped and developing countries are mentioned below

- In small hydro, civil structures are simple and easy to construct by using the local and rural skill.
- Power output is stable and continuous.
- Repairs are easy.
- Transmission and distribution loss is minimum, as the power will be supplied within a small area.
- Once constructed these are immune to inflationary price rise in comparison to power stations, which utilise exhaustible fossil fuel.
- Suitable for peaking support to the local grid as well as for stand alone applications in isolated remote areas where it is not economically prudent to extend the national grid.
- It can reduce the use of wood as a domestic fuel in rural areas and check the rate of forest degradation.
- Reduce the use of diesel generators in rural areas for agriculture purpose if the scope of its development exists.
- For economic development of isolated areas small quantities of hydro energy can solve the problem. A cluster of such projects can meet the power requirement of the region to a greater extent. Extensive forecasts, planning, capital expenditure is not required for immediate implementation.

1.4 TRENDS IN SMALL HYDROPOWER DEVELOPMENT AND ITS SCOPE IN INDIA

Small hydropower technology is not new in India. It has a history of 100 years in installation of small hydro. The beginning of hydropower development in India was marked way back in 20th century when various SHPs were installed in different parts of the country. The first hydro installation of 130 KW capacity was commissioned at Darjeeling in 1897 followed by 4500 KW capacity plant at Sivasamudram, in 1902. Since then its development (hydropower) steadily increased upto 1963. Thereafter the share decreases with development of other sources of energy. Although the country starts its hydropower development with SHP schemes due to its economical exploitation in various agro-industrial and mining works but had switched early to large hydros after independence, keeping in view a fast pace of green and industrial revolution in the country. Upto now the total estimated hydropower installation is of order 24,000 MW from various water resources schemes with only a share of around 1500 MW from small hydros. Due to the increased awareness of environmental concern, more thrust towards rural development and lack of financial resources shifts India's attention again towards the development of SHPs, owing to the fact that it needs not the mass production of hydro energy but production for masses from easily implemented small hydro schemes. Coming to the Indian small hydro potential it can be easily visualised with its great diversity in physical, geographical and geological feature. India has one of the world's largest irrigation canal networks with thousands of dams. It has monsoon fed, double monsoon fed as well as snow fed rivers and streams particularly in Himalayas with perennial flows. So SHP units can easily be installed at toes of existing small irrigation dams, canal drops and run-of-river, high head schemes in remote areas. As a statistical data the global installed capacity of small hydro is around 50,000 MW against the estimated potential of 180,000 MW. With the current definition of SHP the India's estimated potential share is about 10,000 MW. Table 1.5 shows a database created by Ministry of nonconventional energy [48] for statistics of the state wise identified number of SHP sites with its total installed capacity.

Table 1.5: State Wise Details of Identified Small Hydel Sites Upto 25MW Capacity in India.

SL.NO.	Name of State	Identified number of sites	Total capacity (MW)
1	Haryana	22	30.05
2	Himachal Pradesh	323	1624.78
3	Jammu and Kashmir	201	1207.27
4	Panjab	78	65.26
5	Rajasthan	49	27.26
6	Uttar Pradesh and Uttaranchal	445	1472.93
7	Gujarat	290	156.83
8	Madhya Pradesh and Chhatisgarh	125	410.13
9	Maharastra	234	599.47
10	Andhra Pradesh	286	254.63
11	Karnataka	230	652.61
12	Kerala	198	466.85
13	Tamil Nadu	147	338.92
14	Bihar and Jharkhand	171	367.97
15	Orissa	161	156.76
16	Sikkim	68	202.75
17	West Bengal	145	182.62
18	Arunachal Pradesh	492	1059.03
19	Assam	46	118.00
20	Manipur	96	105.63
21	Meghalaya	98	181.50
22	Mizoram	88	190.32
23	Nagaland	86	181.39
24	Tripura	8	9.85
25	Andaman and Nicober Island	6	6.40
26	Goa	3	2.60
	Total	**4096**	**10071.81**

(Source: Indian Journal of Power and River Valley Development, 2001[48].)

A statement of implementation of SHP upto March 2001 is given in Table 1.6 for a quick look of its increasing popularity in the current economic scenario.

TABLE 1.6: Installed Capacity (MW) of Small Hydro in India.

SL.NO.	PROJECT STATUS	TOTAL NO OF PROJECTS	TOTAL CAPACITY (MW)
1	Commissioning	387	1341.05
2	Under Implementation	170	498.28
	Total	**557**	**1839.33**

(Source: Indian Journal of Power and River Valley Development, 2001[48].)

1.5 ROLE OF SHP IN RURAL DEVELOPMENT AND GOVERNMENT INCENTIVES.

About 70 percentage of Indias' population live in rural areas where its economy mainly dependent on agro-production and related industries. Although a good percentage of agricultural area of India is satisfactorily under flow irrigation system still more than 60% of agricultural fields depends upon rain God. To cater these problems, Government of India had started the rural electrification program to decentralise the power sector. But the program has not shown any significant achievement due to low power demand and inherent scatterness of village settlement. The grid connection to the villages in inaccessible remote hilly areas with low load requirement is not only a costly but also a difficult affair. Similarly the economics of other alternative to provide diesel-generating set for sustainable development is subjected to inflationary economy and environmentally degradable. On the other hand the official statistics of low per capita energy consumption in rural areas shows the poor economic growth of the region. This reveals that the areas are backward in terms of human resources developmental parameters, like poverty, medical facilities, scope for higher education and above all poor income generation capacity. Here, in such locations if the scope of establishing SHP exists, it can play a greater role in rural as well as sustainable development. It is helpful to the rural mass for their poverty alleviation program in the following ways

1. The generated power serves as an essential input for improvement of the socio-economic condition with low generation and transmission cost.
2. A cheap source of power and can be affordable to village community.
3. Once the rural consumer is able to generate surplus income from electricity based activities it will lead to sustainable economic development.
4. Effectively stop rural migration to cities due to the emergence of electricity-based employment generating enterprises in the locality.
5. An alternative solution to techno-economically unviable grid extension and protect the right of rural citizen.
6. An energy substitution to fuel wood consumption thus reducing forest degradation.

Visualising the effectiveness and usefulness of SHP schemes in rural development as well as overall increased scenario of power in the country, the Government of India has announced different incentive schemes. The Ministry of Non-conventional energy sources have the responsibility to promote SHP development upto 25 MW capacities. The Ministry's incentives and responsibility to SHP development shall be of great benefit to promote these schemes not only in public sector but shall also encourage private sector participation.

1.6 ECONOMICS OF SMALL HYDRO

It is a fact that the per KW capital and generating cost is more in case of small hydro as compared to large hydro and thermal power plant. But if one considers other benefits mainly its role of sustainable development in remote, inaccessible hilly rural areas, then one cannot question for its promotion. However, before going for any scheme one has to examine the relation between the cost of electricity generation and its tariff with the income of the village population. Planner has to think about the income generating activities that are bound to be promoted in the locality by using electricity based activities so that the consumers are able to pay the tariff.

Considering the technical approach for reduction of capital cost of SHP system one has to deeply analyse various costs involved in a SHP scheme. The capital cost varies considerably in case of SHP because of the fact that no standardized is possible and site conditions available differs from project to project along with other shortcomings such as

1. Redundancies in key areas such as layout of civil works, selection of turbine–generator equipment and specification of electrical switching and protection system.
2. Use of complex layout resulting in increased gestation period escalating the capital cost and interest payment.
3. Over manning the small hydel stations with technical staff results in increased operation and maintenance and overhead charges.

The capital cost of small hydel scheme consists of the following

(i) Civil works- Intake, gates, penstocks/pressure shaft, powerhouse, tailrace etc.

(ii) Hydro-mech-equipment - Turbine with its auxiliaries, main inlet valve etc.

(iii) Electrical equipments – Generator and its auxiliaries, transformers, capacitors, breakers, switches, etc.

(iv) [illegible] lines.

Among the above components the first three constitute a major portion of the total cost. There is considerable scope for cost reduction in each component as follows.

(a) Adopting Optimum Design Head And Fixation Of Installed Capacity

The design head depends to a large extent on the flow and head data available. Hence the flow duration curve has to be plotted very accurately and optimum head and unit rating to give optimum energy productivity should be selected. The installed capacity of the scheme should be so fixed that the maximum return on capital cost can be achieved within a stipulated period.

(b) Selecting the Optimum layout

The layout should be carefully considered so that it will avoid any elaborate intake structure, require shorter length of water conductor system, proper location of penstock intake, avoiding any reorientation of power house, and above all taking into account the over all geological condition of the landscape. It shall primarily be aimed at reducing civil works.

(c) Standardising As Well As Optimizing The Designs And Specifications

This is mainly aimed at reducing the construction and manufacturing cost and the procurement time. The construction of civil structure can be minimised by optimal hydraulic and structural design. The thought of safety deters man to be economic. So in small hydro there must be a compromise between measures of safety and economics of a project. This can be achieved by providing simple type of structures, which could be built mostly with local labour and material. In case of turbine and generator the design should be standardised by classifying different heads and output groups and develop runners for each group. The specification should also be standardized keeping in view the economy of SHP. CBIP [11] has already set some standard specifications for civil works as well as turbine and generator.

1.7. SCOPE OF THE STUDY

Before implementing any SHP project, the layout, hydraulic and structural design of civil structures, suitability and costing of electromechanical equipment, layout and costing of transmission lines, economics between investment and fixation of power tariff etc. have to be finalised. Out of it the cost of civil works is a major component of capital cost. Any deficiency in assessment of hydrology of the stream, redundancy in layout, failure in judging appropriate civil structures at proper location with reference to hydraulics and geology of site, lacking in appropriate design considerations in civil as well as electromechanical components will lead to an over all failure of the project. So it is very much essential to examine each and every component of the scheme before giving it to a final shape. In the SHP scheme water is the input. Its quality, conveyance and regulation is of utmost importance for successful commissioning of the scheme and that requires sufficient knowledge of hydraulics. In this study a systematic attempt has been made to review hydraulic design procedure of different civil structures in SHP adopted in

India and other countries under varying site conditions. A critical analysis has also been made for suitability of these structures to the proposed power plant on river Asiganga a tributary of Bhagirathi in Uttarkashi district of Uttaranchal State. The study of civil structure of powerhouse is not included as it depends on the type of equipment and it is more related to structural aspect.

1.8 ORGANISATION OF DISSERTATION REPORT

CHAPTER 1 Introduction giving the brief background of the SHP developments, scope of study, and the organization of report.

CHAPTER 2 Describes the planning and layout of civil structures in small hydropower scheme.

CHAPTER 3 Deals with various types of Diversion and Intake structures used for small hydropower project, their suitability to the site and hydraulic design parameters.

CHAPTER 4 Describes different methods adopted for desilting the intake water meant for the power generation with their possible locations.

CHAPTER 5 Contains hydraulics of water conductor system and its design parameters.

CHAPTER 6 Deals with hydraulics and design parameters of forebay, balancing reservoir and surge tank.

CHAPTER 7 Describes the hydraulics of penstock design.

CHAPTER 8 Explains the hydraulic design of tailrace channel.

CHAPTER 9 It deals with the case study of design of hydraulic structures of a Small Hydro Power project on river Asiganga a tributary of the river Bhagirathi in Uttarkashi district of Uttaranchal.

CHAPTER10 It gives conclusions and suggestions for further study.

CHAPTER 2

PLANNING, SURVEY AND LAYOUT OF SMALL HYDROPOWER SCHEME

2. 1 GENERAL

For the success of any hydropower scheme the project planning, which is done before going in details of design and construction aspect of power plant, is of great importance. The planning of a small hydro power plant (SHP) is made keeping in view certain aspects of the projects such as

a) Resource availability.
b) Power potential.
c) Demand of power – Local or in the grid.
d) Accessibility to site.
e) Infrastructural facilities.
f) Power evacuation arrangement.
g) Techno-economic feasibility.

The process of planning small hydro schemes is more or less the same as in other hydropower schemes involving identification, various investigations, engineering studies and preparation of project reports. This chapter describes the various aspects of planning, survey and alignment of the scheme with different civil structures in a small hydro power plant.

2.2 IDENTIFICATION OF SITE AND POWER POTENTIAL STUDY.

The scope of this preliminary study is to identify potential sites for micro/mini/small hydropower development [30]. Generally there are four types of hydropower development to be investigated, namely:

a) Run - of - river type.
b) Reservoir type.
c) Hydropower installation at existing irrigation storage dams.
d) Low head schemes in irrigation canal.

The activities involved in preliminary studies are briefly described as follows.

2.2.1 Site Selection

The site selection of run-of river and reservoir projects is based on the available topographic maps of 1: 50,000-scale [30], [38]. Some of the basics of each type are discussed below.

The run-of-river projects should be located in hilly areas in order to achieve high heads. However the best location is selected on the basis of detailed knowledge of the area considering aspects like, nearness to load center, perennial flow in the stream with sufficient discharge in lean season, availability of adequate drop with short length of water conductor system and possibility for economically providing diurnal pondage.

The reservoir type scheme is adopted where the topographic and hydrologic conditions are favourable for providing sufficient storage volume for flow regulation.

The existing storage dams of medium scale irrigation projects, which normally have head and storage capacity, can be economically and potentially be utilised for small hydropower development. The site of irrigation canals having larger discharge and small fall can be utilised/selected for harnessing hydropower development.

2.2.2 Project Layout And Configuration

The project components are laid out based on topographic maps of 1:50,000 scales. The project configuration of run-of–river type comprises a diversion weir, intake structure, flow control structure, head race, forebay, penstock, power house, tail-race channel/tunnel, transmission line and access road [Fig.2.8]. Basic components of reservoir project are similar to those of run-of-river project. The dam type is selected based on the topographic and geologic conditions of the site as well as the available construction material .The hydropower installation at the existing irrigation storage dam [Fig.2.9] consists of the modification of dam outlet for the inclusion of power components. Here either water releases from the powerhouse outlet before flowing into irrigation canals or a river will be diverted to powerhouse through power conduits. The hydropower scheme over the irrigation canal [Fig.2.10] consists of a bypass canal, close to the existing canal and the incorporation of a small hydropower station in the system.

2.2.3 Hydrologic Study

Five aspects are taken into consideration in hydrologic study.

- Ten daily/weekly, Monthly and mean annual flows at the proposed diversion site
- Flow duration curve analysis;
- Estimation of flood discharge;
- Sedimentation; and
- Other water requirements.

For the study 10 years data of river flow should be available at the proposed site or at the site having identical geological and hydrological characteristics. For pre feasibility study the discharges at site for $1\frac{1}{2}$ years (2 non-monsoon seasons and one monsoon season) are essential

2.2.4 Power and Energy Potential

Energy potential of a run-of-river scheme depends on the stream flow characteristics, available head and design discharge. For planning purposes the 50% dependable flow on 90% available year basis using flow duration curve is generally taken as design discharge. For reservoir based plant reservoir simulation study is conducted to estimate the capacity and energy output of the reservoir project. It is operationally based on the release requirements of the primary purpose of the project. Energy potential for irrigation canal based plant can be estimated by available discharge and canal fall height.

2.2.5 Project Cost Estimate

Preliminary quantity estimate is based on typical design and sizing of each project component as well as topographic conditions of the site. Using information from the existing hydropower projects and those under construction, the unit price and cost of electro-mechanical equipment is derived. As a matter of fact, the present per Mega Watt cost of SHP is higher as compared to large size hydro project. Minimizing the cost of electro-mechanical and civil works can optimize the cost of SHP.

2.2.6 Economic Viability

The concept of least cost of generation per unit (KWH) at the bus bar is applied to select the alternatives for those SHP schemes intended to serve isolated communities and/or to replace the existing diesel generation units, the least cost alternative between investment for grid extension and diesel generation unit is used for comparison with small hydropower projects. On benefit and cost basis, the hydropower projects are considered to be economically viable if the internal rate of return exceeds 7 percent.

Considering the economical aspect and to increase the internal rate of return the reduction in cost of Electro-mechanical equipments and cost of civil works is very much essential. Electro-mechanical equipments contribute about 40% of the total project cost. The cost of these equipments could be reduced by

a) Suitable innovation in the design by simplifying some of the requirements of the conventional hydropower systems for cost effectiveness and can be achieved by following ways [38] :
    i. Use of fixed vane turbines instead of full Kaplan turbine.
    ii. Use of induction generator instead of synchronous generator.
    iii. Elimination of governor system.
    iv. Using aluminum bronze instead of stainless steel to fabricate the turbine.
b) Standardisation of auxiliary and control system:
c) The reduction in cost of civil works can be achieved by the following ways.
    i. Simplification in the design of conventional hydro station. Costly diversion structure and desilting arrangements should be simplified. The time of construction of civil works shall be reduced by using prefabricated structures.
    ii. Use of locally available materials for constructions.
    iii. Right choice of water conductor system to reduce the operation maintenance cost.
    iv. Careful site selection of different civil component of the SHP scheme.
    v. In case of irrigation based SHP, the structural modifications to the existing irrigation facilities should be reduced to a minimum with a judicial exploration of suitable site for intake of SHP. The layout of civil structural and electrical switching systems be streamlined so that the scheme could be completed within a few irrigation seasons.
d) The reduction in operation and maintenance cost can be achieved by automation so that continuous watch over the control is not required.

2.3 SURVEY

After the project site has been identified and selected during preliminary planning and study, various field investigations are required to be carried out for formulating the schemes. The investigation and survey task include [16], [30],[38].

2.3.1 Collection Of Hydrological Data And Water-Flow Measurement

Minimum of 10 years of data of river flow should be available at the proposed site or at the site having identical, geographical and hydrological characteristics. If the hydrological data are not available for the proposed stream then the field measurement of stream flow at selected site should be measured and compared with that of nearby gauging station.

2.3.2 Topographical Survey

The topographic survey is conducted based on the map from preliminary study with the following objectives.

- To mark the alignment of total hydropower scheme.
- To mark the alignment of possible project route under the existing field conditions.
- To select the location of each structural component along the project route.
- To check the gain head from the existing conditions.
- To check road conditions or an access to the site.
- To investigate the alignment of transmission lines etc.

2.3.3 Geological Investigations

After determining the possible project alignment, the geological conditions of the project site are investigated to confirm the suitability of the civil work constructions such as:

- Whether the foundation of the headwork and powerhouse will be on sound rock or soil.
- Material on hill slope and their stability.
- Type of strata along the headrace, penstock and access road alignment.
- Type of strata for location of forebay, surgetank, powerhouse, tailrace etc.

Test pit and drifts are used by geologist for these investigations. The extent of investigation should be judiciously reduced to a minimum as these are expensive and time consuming and may substantially add to the cost of project.

2.3.4 Socio-Economic Survey

The surveys comprise geographic location, service area, community structure, number and size of households, population and its growth rate, the existing and future electricity consumption demand etc.

2.3.5 Construction Material Survey

The survey of construction material for different civil structures is of utmost important, for overall construction cost estimation and the suitability of the type of civil structure. This survey includes the following

- Distance between the project site and material source.
- Unit prices of material.
- Transportation cost
- Quantity and quality of the materials.

2.4 LAYOUT OF SCHEME

The different civil components of a small hydropower scheme are diversion works, intake structure, desilting chamber, water conductor system, cross drainage works, forebay, surgetank (if necessary), tailrace etc. These components are designed based on the data collected and compiled after various investigations already discussed in the previous para. The detailed site investigation of each component requires a proper and judicious layout with respect to the topographical characteristics of the project area. Depending upon the topographical and existing geological condition different types of layout are possible [24], [26] [27]. A general lay out for a run-of-river type project of SHP is shown in Fig.2.1. The judicious choice of diversion and intake structure, the desilting arrangement, the headrace conduit, forebay and flow-controlling structure at intake of powerhouse, penstock and tailrace are vital for overall economy of the project unlike large hydropower scheme. The layout of the diversion scheme generally depends upon the length of the water conductor system, the existing topography and geology of the site. When the length of the canal is comparatively short, say less than 0.5-0.8 Km, on a flatter terrain then the water conductor system may be a storage canal or canal forebay [Fig.2.2]. If the topographical conditions are suitable and side streams cross the alignment of the canal, an intermediate pond or storage forebay may be created as is shown in Fig. 2.3 or in Fig 2.4. Similarly if the canal in its alignment cross several small streams and gullies it may require more cross drainage structures such as inverted siphon, under tunnel, and aquaduct. During the alignment of the canals the hill slopes and slope wash along the canal, as well as potential slides zones should be properly considered. To avoid the open channel water conductor system from possible damage by surface erosion and to have flexibility with respect to vertical alignment, the headrace may be made in the form of a closed conduit. The conduit may be either a pipe, non-pressurised or pressurised, or a tunnel as per the available topographical and geological condition of the site. The alternative layouts of closed conduit with surge tank/forebay are shown in Fig. 2.5, 2.6, and 2.7. River bends and good geological conditions, as well as steep slopes along the river are generally favourable to a tunnel alternative or tunnels combined with canals. Short tunnels can detour unfavorable areas of canals, such as old slides, taluses, potential slides, local steep slopes. Before considering the alternative of a tunnel due weightage should be given to the availability of skilled labour locally for the job, otherwise the cost may increase.

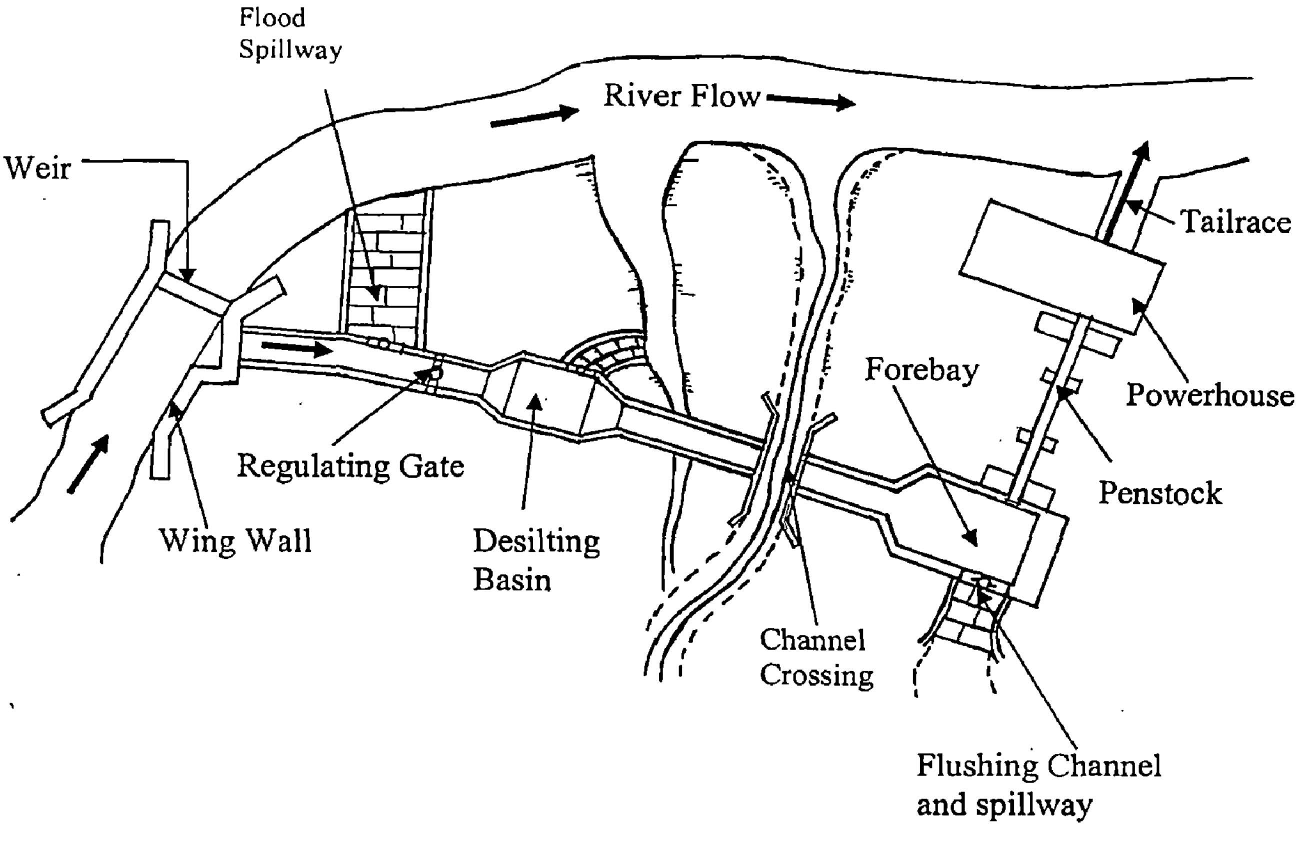


Fig.2.1: PLAN AND LAYOUT OF SMALL HYDROPOWER SCHEME

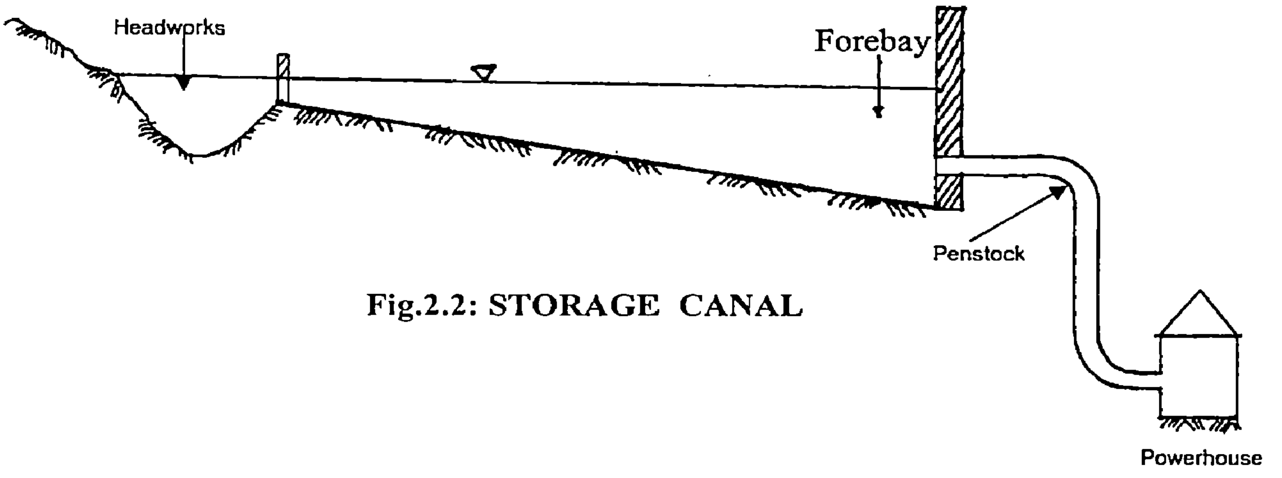


Fig.2.2: STORAGE CANAL

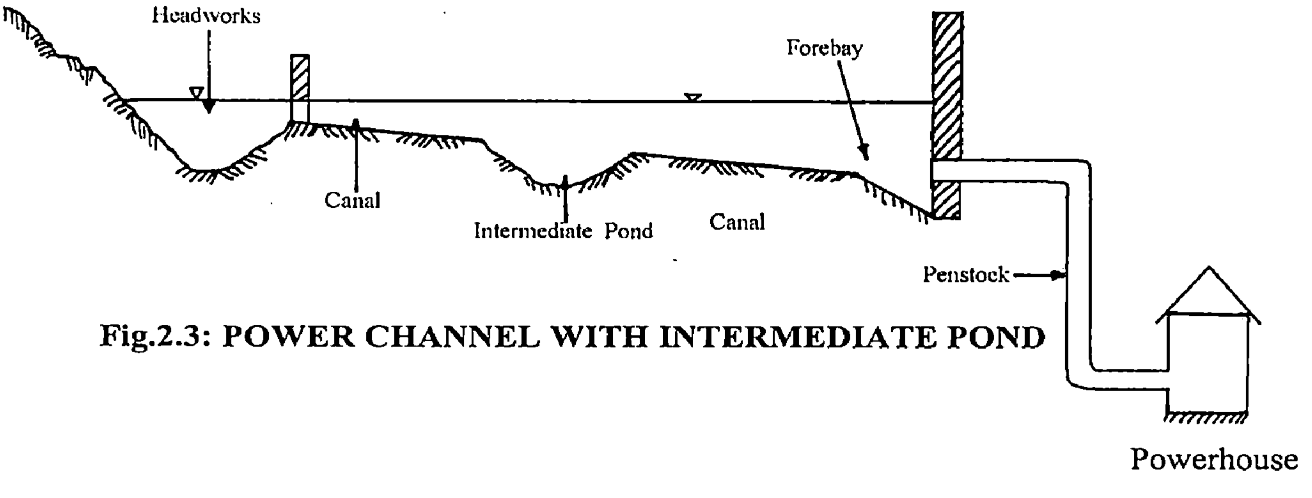


Fig.2.3: POWER CHANNEL WITH INTERMEDIATE POND

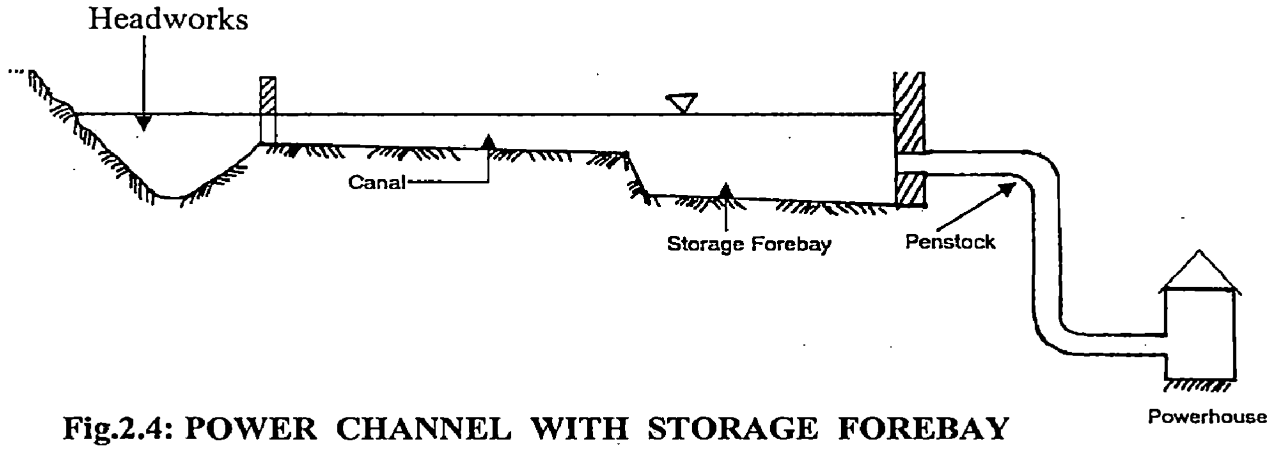


Fig.2.4: POWER CHANNEL WITH STORAGE FOREBAY

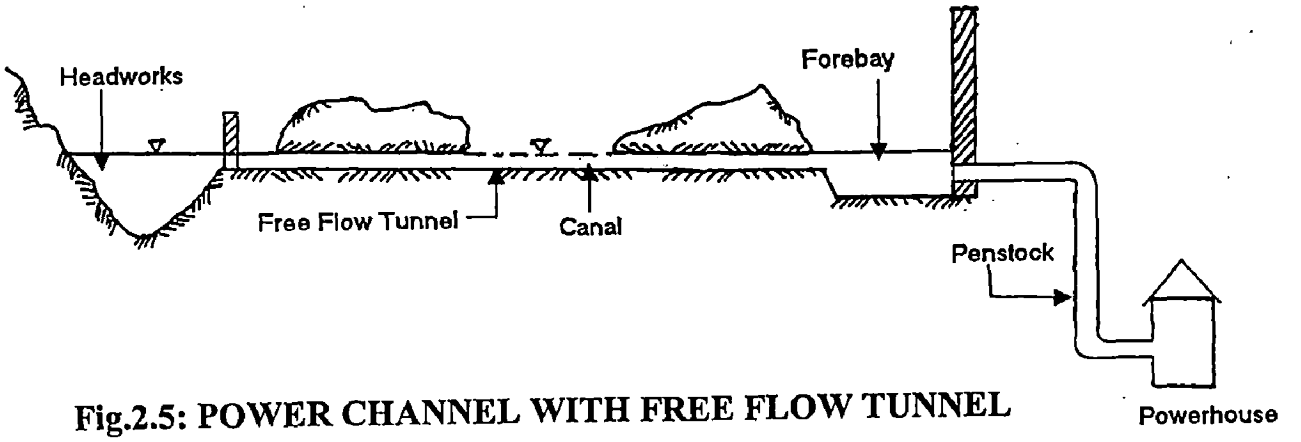


Fig.2.5: POWER CHANNEL WITH FREE FLOW TUNNEL

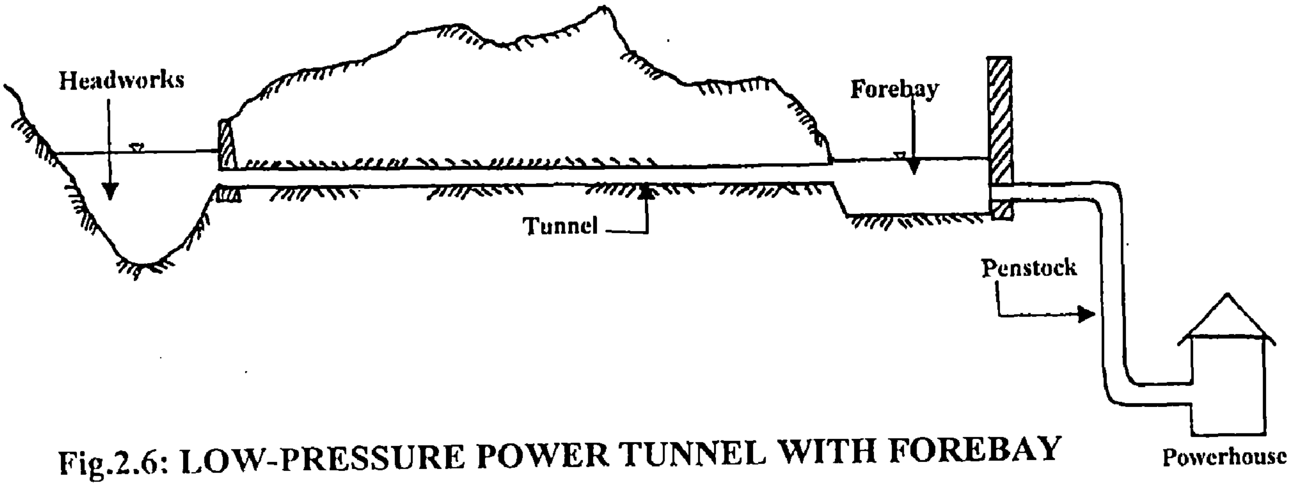


Fig.2.6: LOW-PRESSURE POWER TUNNEL WITH FOREBAY

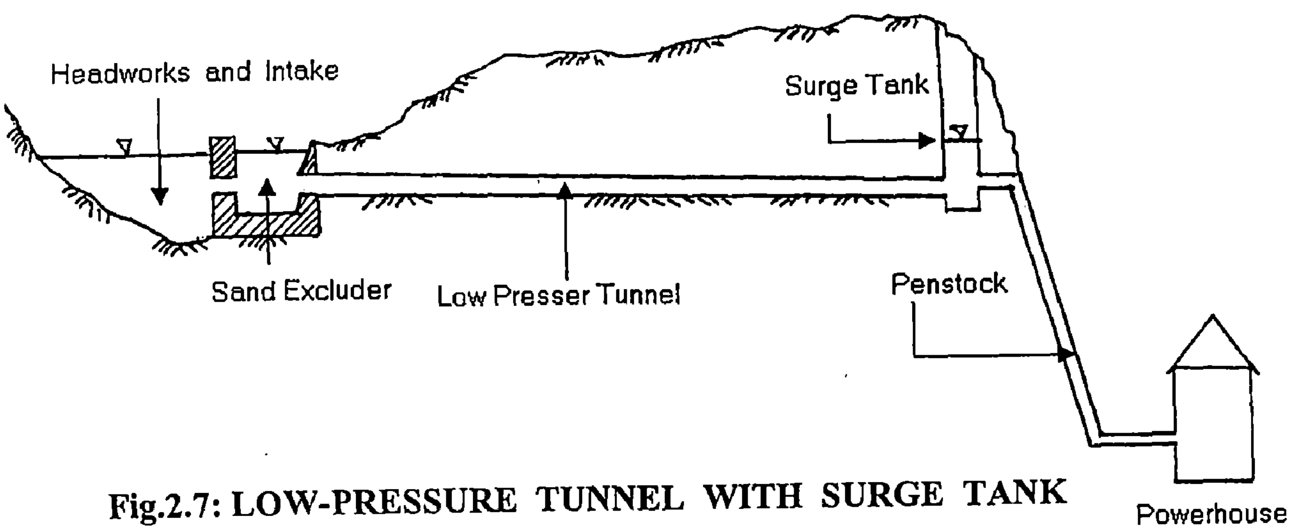


Fig.2.7: LOW-PRESSURE TUNNEL WITH SURGE TANK

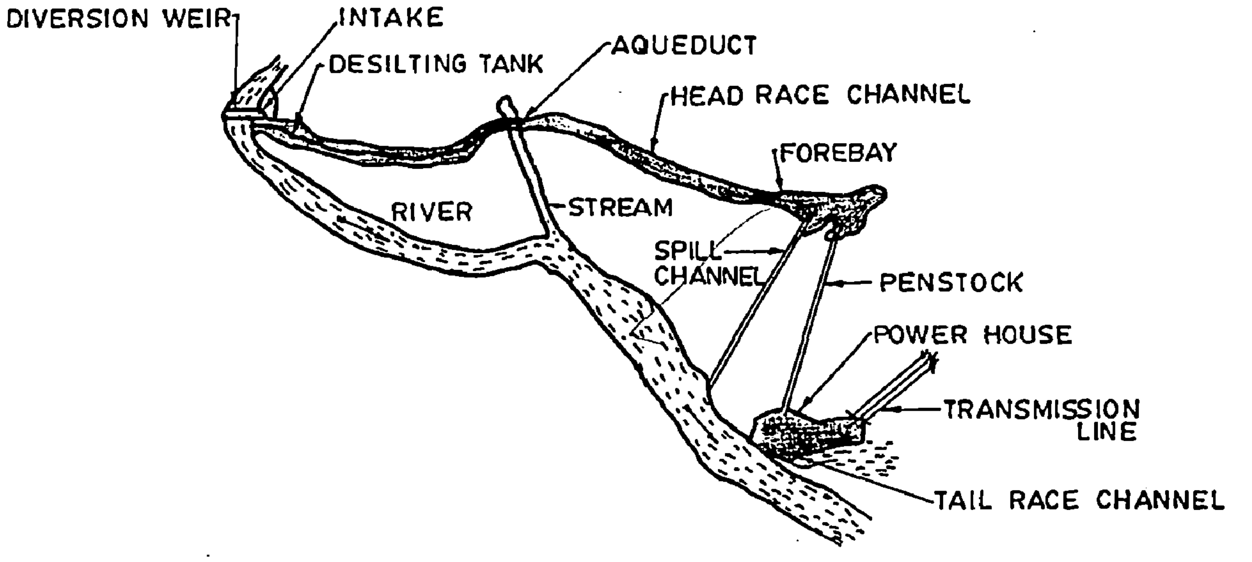


Fig.2.8: TYPICAL LAYOUT OF RUN-OF-RIVER SMALL HYDRO -POWER SCHEME

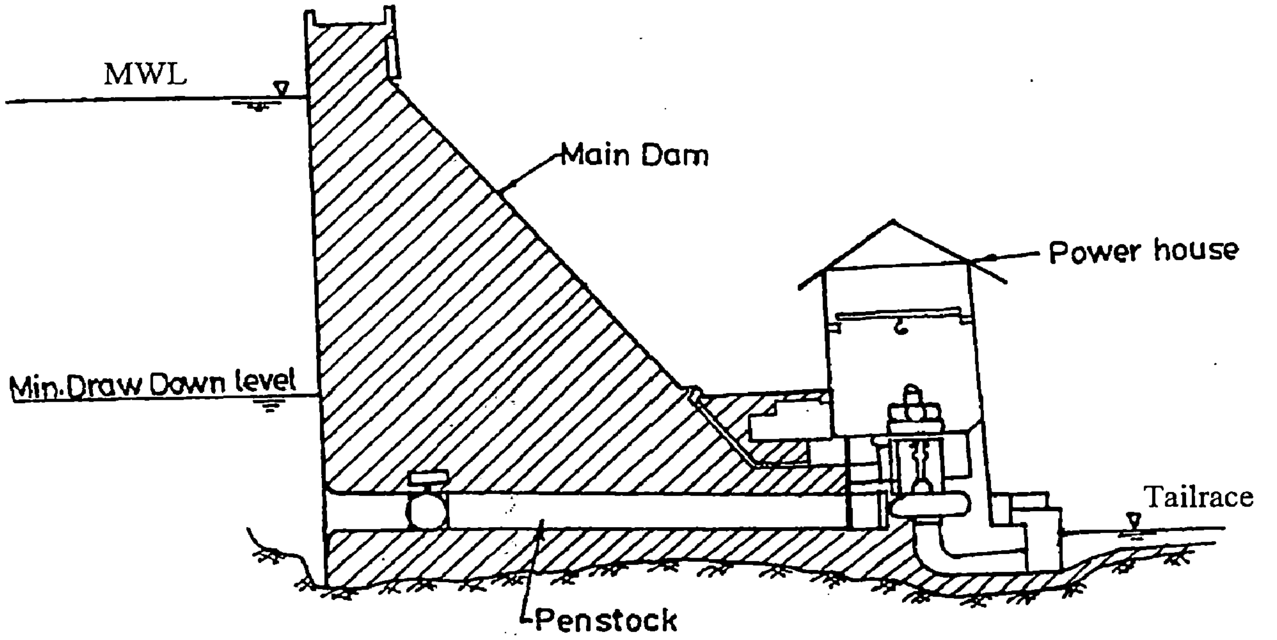


Fig.2.9: TYPICAL LAYOUT OF DAM TOE SHP SCHEME

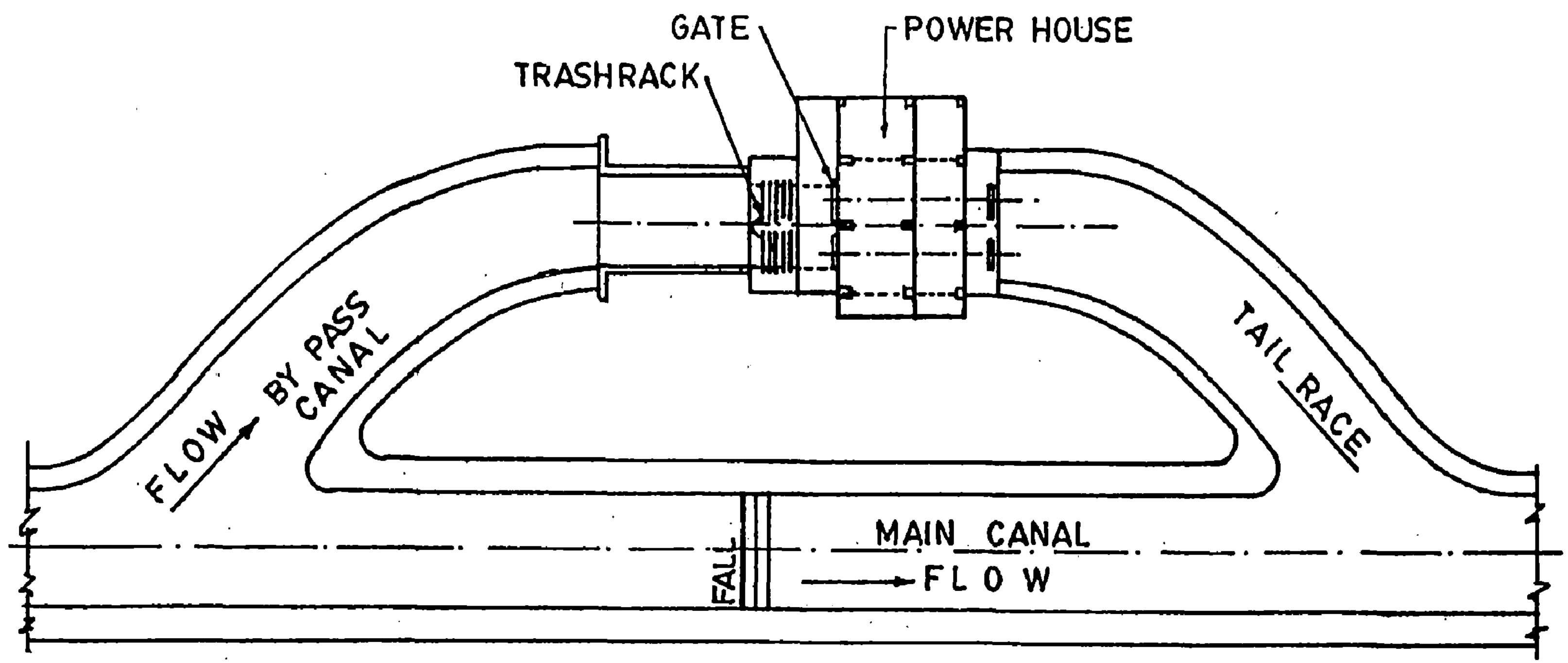


Fig.2.10: TYPICAL LAYOUT OF CANAL FALL SMALL HYDRO POWER SCHEME

CHAPTER 3

DIVERSION AND INTAKE STRUCTURES

3.1 GENERAL

A diversion head works (or a weir) is a structure for diverting a specific amount of flow of water from the river or stream toward the intake channel for generation of hydropower. Diversion head works are generally constructed on the perennial rivers that have adequate flow throughout the year and hence it eliminates the construction of storage reservoir. Sometimes it has a small storage. It raises water level in the river so that it can be diverted into the offtaking canals. In general a diversion headwork serves the following functions.

(i) It raises the water level on its upstream side.

(ii) It regulates the supply of water into off taking channel.

(iii) It controls the entry of silt into canals/channels/conduits.

(iv) It creates a small pondage on its upstream.

(v) It helps in controlling the vagaries of the river.

Canal diversion-headworks consist of different components such as: weir/barrage, under sluices, divide-wall, fish ladder, canal/channel head regulator, silt excluder, guide banks, marginal bunds etc. The diversion head works are different for low, medium and high head developments .The diversion head works for low and medium head development can be of

(i) Diversion canal type.

(ii) Run-of-River type with head water level well above the highest original flood level.

Similarly the headworks of high head developments can be of

(i) Diversion canal type,

(ii) Diversion tunnel type, and

(iii) Dam type.

In this study only the diversion works that is hydraulically and economically suitable for high head small hydro power plants are discussed and others are out of the scope of the study. However the design features and structural arrangements of diversion for high head and low head power plants are similar. Generally the free flow conduit developments are adopted in small hydro plants. The most conspicuous difference between the high and low head developments is the elaborate desilting arrangement at the diversion head works.

For diverting flow of streams for power generation a weir with raised crest is usually required for an adequate flow into the intake located at the abutment. The weir shall be designed to feed the intake in low flows and allowing high floods to pass in the down stream unhindered. Various types of structures have been adopted for the purpose. These have been discussed in this chapter highlighting their limitations.

3.2 DIFFERENT TYPES OF DIVERSION STRUCTURES

Based on the principle of water abstraction, Scheuerlein (1979)[20] has identified four basic intakes types as given below [Fig. 3.1].

(i)	The lateral intake.	(ii)	The bottom intake,
(iii)	The frontal intake	(iv)	Siphon intake

According to Scheuerlein, suitability of an intake type depends on size, slope, state and pattern of the river, sediment transport rate and the water abstraction ratio. Basing upon these principles normally in small hydro, five common types of head works are adopted keeping in view the economy and suitability to the site conditions such as:

(i) Canal diversion type head works.
(ii) Tyroler or bottom intake head works.
(iii) Siphon type intake.
(iv) Intake through holes in Ogee type weir using self cleaning screen
(v) Head works with canvas/rubber gate.

3.2.1 Canal diversion type head works

This type of diversion structure is commonly adopted between boulder and trough stage of the river having small size boulders as bed load. In most of the small hydro power projects the diversion structure is a small dam or weir with sand sluice, desilting forebay, training wall, abutment, apron, side channel and other appurtenances as shown in the Fig. 3.2

3.2.1.1 Design Principles

The following design principles are generally followed [1, 24, 32, 33].

(a) The weir should be aligned at right angles to the direction of flow of the river, as far as possible. Such an arrangement ensures the minimum length of the weir, good discharging capacity and the least cost. However, if riverbed consists of gravel and shingle, an oblique alignment is sometimes preferred. In case, if the alignment is at right angles, gravels and shingles may enter the main canals through the head regulator and get deposited in the head reaches of the canal. The best alignment is usually decided after conducting model studies.

(b) The crest elevation of the overflow weir should be higher than that of the side channel weir or open intake by a height (H_0) enough to divert the desired discharge (Q_0). When the river flow is less than (Q0), all the river flow is diverted into the canal, without attendance. During the flood season the desilting forebay can be periodically cleaned through scouring gates and the excess flow of the canal be discharged back to the river through side spill way [Fig. 3.3].

(c) The highest level (relative to the forebay bottom) of the side channel weir or the guide sill should be greater than 1-1.5m to accommodate the silt deposition desired. The ratio of weir or sill height to the water depth in the forebay be greater than 0.5 and less than 0.7

(d) Generally the desilting forebay width is taken as 0.05 to 0.2 of the total width of the river depending upon the diversion rate required and should be so determined that the average velocity (0.3 to 0.5 m/s) in the forebay is as less as possible to settle out the harmful particles. During scouring, the mean velocity in the forebay should be greater than 1.5 – 3 m/s depending on the particle size of the deposits. The threshold / incipient velocity can be approximately estimated by the formula.

$$V_t = \sqrt{g d_{max}} \tag{3.1}$$

where, V_t = threshold velocity (m/s)

g = 9.81 m/s^2

d_{max} = maximum size of gravel or cobble (m).

(e) The design principles of scouring gates are as follows:

(i) The opening size should be large enough to create the desired scouring velocity.

(ii) Scouring gate should be non-submersible.

(iii) The opening size should be large enough to pass through the maximum size of particle deposited.

(f) The base floor of the forebay may be horizontal or slightly sloping towards the downstream. But the elevation of the base floor should be set at the average riverbed during dry season.

(g) The passage of the scouring gate or gallery as well as the apron, should be properly protected against abrasion due to rolling gravel or cobble.

(h) The guide wall should be stretched upstream from the intake by several metres, depending on the width of the forebay.

3.2.1.2 Types Of Weir

The following types of weirs or small dams of overflow type are usually adopted in small hydropower scheme depending upon the construction material used. As a matter of fact these weirs are used due to low-cost and easy availability of material at site, these are:

- **(a) Concrete and masonry dam**
- **(b) Crib dam**
- **(c) Earth dam**
- **(d) Rock fill dam**
- **(e) Boulder bar**

Concrete and Masonry Dam

These dams are generally of height less than 3.5m with base width 1.5 times height .The d/s apron is given a shape such that the water flow will be turned slightly upward to dissipate energy and protect the bed from eroding. The foundation must be on sound rock. These are generally designed as over flow weirs. Preventive measures shall be taken against sliding [Fig.3.4]

Crib Dam

Crib or timber dams are economical in places where timbers are available in plenty. About 10 to 15cm thick tree trunks are placed 0.6 to 0.9m apart at right angles and nailed to each other. Stones fill the space between the trunks. The upstream and down stream face of the crib dam is covered with planks. The face is sealed with clay to prevent leakage. Down stream planks [Fig. 3.5] used as an apron to guide the water that overflows the dam into the streambed. The down stream apron should be provided in order to prevent damage from scour. These need good rock foundation. These dams are widely used in USSR, CANADA and BRAZIL.

Earth Dam

Earth dams are provided where concrete/building materials are expensive and scarce. Earth dams are made of soils with flat slops 3:1 (H:V) both in upstream and down stream satisfying the criterion of stability. Generally in small hydro schemes a homogeneous earth fill dam [Fig. 3.6] of uniform impervious material is adopted for small heights. These are suitable for all types of foundations but care should be taken to check the seepage through foundations. A positive cutoff of concrete anchored into rock should be provided to check piping in the body or in foundation .To guard against overtopping sufficient free board with masonry spillway section is provided. The difference in level between the crest and riverbed level in the d/s should be negotiated through sloping apron of concrete or masonry to protect the riverbed from scour.

Rock fill Weir

This is generally suitable where riverbed is rocky and boulders are available in the riverbed. Here boulders or crushed rocks are laid in the intervening masonry walls [Fig. 3.7]. The down stream slope is flatter than upstream. These types of weir are constructed economically in large numbers in CHINA and USSR [24].

Boulder Bar

This type is suited at the place where boulders are available in the river and rock is met in the riverbed within 1.0m depths. The boulder bars [Fig. 3.8] are constructed by encasing boulders in GI wires, piling them up to form the weir and covering the same with 10cm thick rich cement concrete reinforced by 10mm diameter mild steel bars provided at 100mm c/c on the faces. The base of the dam is taken equal to the height for stability of the structure.

3.2.2 Tyröler Or Bottom Intake Head Works

The bottom intakes popularly known as trench weir are extensively used in small hydropower scheme on mountainous streams. It is ideally suited to small sized, straight or meandering rivers having bed slopes not less than 0.1 %, water abstraction ratio (Qe/Q) more than 25% with sediment transport rates low to medium and subjected to flash floods thereby causing movement and rolling of big boulders. They are generally constructed where the riverbed is not rocky. Such structures are techno economically attractive because these are not affected by rolling big size bed material.

3.2.2.1 Types Of Bottom Intake

Literature review indicates that two typical versions of bottom intakes or trench weirs are being commonly used to abstract water from streams for various purposes in run-of-the-river schemes [Fig.3.9]. These as defined in Draft Indian standard [6] as:

(a) Trench in the riverbed, placed across the stream having a trash rack structure over it.

(b) Trench in the weir raised above the riverbed, placed across the stream and covered with a trashrack structure.

3.2.2.2 Trench Weir

Trench weir is a device provided at the bottom of the river channel for the purpose of diverting a part of the original flow. The device consists of an opening trapezoidal or rectangular in shape made up of masonry or concrete, in the channel bottom covered with steel rack to prevent the entry of unwanted bed load and floating material through the opening [Fig. 3.10]. The trash racks are of different types such as

(a) Transverse bar bottom racks, where bar racks are placed transverse to the direction of flow.

(b) Longitudinal bar bottom racks, where bars are laid parallel to the flow direction.

(c) Perforated plate bottom racks in which a plate with uniformly spaced circular openings forms the rack.

The above types of trash racks can either lay horizontal or inclined with reference to the bed slope of approaching channel. Generally in intake structure of mountainous streams especially for micro, mini, small hydro projects the longitudinal racks made up of trapezoidal shape steel bars are in use for better result. The trash rack is given a slope of about 1 in 10 in the flow direction so that stones and pebbles do not settle on the rack, but roll down with the stream flow. The trash rack is periodically cleaned after monsoon season to clear any deposited material. The trench itself is provided with a bed slope between 1 in 10 to 1in 20 in the flow direction of diverted water so that sufficient velocity is generated to carry away small stones and heavy silt that may find entry into the trash rack openings. The concrete apron is provided and extended beyond the wing wall so that the stream flow during the heavy floods will pass over the trench without damage to the intake works. As a protective measure to such weirs, bed pitching in concrete blocks or random stones of proper size with stone /concrete retaining walls on the sides is provided for at least 15 to 20m upstream and downstream of the trench weir. Besides a horizontal trash rack, additional opening with trash bars is provided on the sidewall of intake weir to allow some flows to the intake in flood season if the horizontal trash rack is chocked [24].

The intake well at the end of a trench weir may be made of R.C.C or stone masonry. It is provided with two gate openings, one for flushing of settled silt through a circular closed conduit, desilting pipe and other for the power channel [Fig. 3.11.]. The opening of the flushing off pipe is kept about 60 to 75 cm lower than the opening for the water conductor for flushing the silt and coarser material through the desilting pipe. The minimum flushing velocity in the desilting pipe should be 2 m/sec to 2.5 m/sec. It should be aligned in such a way that it negotiates the flood height in shortest possible length and discharging into the same river or nalah near by. The power channel / water conductor in the initial reach could be in the form of a concrete hume pipe to a distance where it is above high flood level and embedded in the concrete to provide protection from scouring.

3.2.2.3 Layout and Site Selection of Grated Intakes

The layout and site selection of headworks of grated intake are very important in tapping silt free, assured quantity of stream/river water for hydropower generation. Following are some recommendations [24].

(a) During site selection the natural river regime should not be disturbed as far as possible i.e. the construction of headworks at the site should give as less influence on the natural river regime as possible as shown in the Fig. 3.12, which is the layout of a mini hydro head works in Shingshan, Hubei province China. The crest elevation should be even with the mean water level during dry season and length should be 70-80% of river width. The site can be selected on a straight reach of the river where the channel bed is comparatively stable and the main current in the middle of the river.

(b) The provision of sand guide sill, upstream of the trench weir and scouring sluices should be explored which not only effective for silt free diverted flow towards the trench weir and delay the clogging of grate but also guide the flood flow with bed load to the scouring sluices Fig. 3.13. The sill can be straight or curved, but its relative height (ratio of sill height to the water depth) should be 0.5 - 0.7 and no less than 0.5, the greater its relative height, the better its effectiveness.

(c) The intake weir had better be sited before a rapid or waterfall to gain a head to facilitate the silt sluicing from intake well. From the viewpoint of silt-prevention the intake-weir, as well as the scouring gate should be non-submergible.

(d) The use of cross circulation flow by means of natural river bend or artificial curved approached channel can effectively prevent the silt from entering the gallery.

3.2.2.4 Sediment Exclusion and Dimensioning Of Grated Intake

The usual problem with trench weir is clogging of the racks or grates. Some of the sediment exclusion devices and dimensioning of grates in India and different countries are discussed below.

(a) The additional feature like sand-guide sill, scouring gate, slopping of grate can have remarkable silt exclusion feature from intake water and less chance of clogging of grates. Regarding slope of grate experimental results in USSR and CHINA shows that when the grate slope is 0.2, the self-cleaning effect is better but its discharge capacity would be significantly reduced. So a slope between 0-0.2 is recommended [24] in design.

(b) The provision of a sand collector channel, upstream of trench weir greatly reduces the silt concentration of diverted flow [Fig. 3.14]. The width of the sand collector should be (0.5-0.7) times the width of the collector channel .The slot width of the sand collector should be 1.5 times that of grated intake. The slope of sand collector had better be separately inclined to both sides. Its slope should be steep enough to sluice off the bed loads to the downstream. The diameter or minimum width of sluicing tubes is generally 0.3-0.4m and their exit should not be submerged.

(c) The use of perforated plate, settling well is applicable to mini hydel. Perforated plate discharge efficiency is less than grated intake, but less clogged by bed loads and floating debris [24].

(d) For small hydro power stations the harmful sand-particles are those greater than 0.3- 0.5mm depending on the hardness of the sand and type of turbine. As for some foreign codes [51] if the silt concentration is less than 0.5 kg/m^3 and the content of the harmful particles are less than 0.2 kg/m^3 settling basin can be avoided.

As far as the dimensioning of grates are concerned different countries has recommended different slot width [24], [11] such as:

India	30mm c/c
China	12-20mm c/c
Russia	06-15mm c/c

Coming to the form of grates, three forms are generally available as below.

Round bar - Greatly used in India.
Trapezoidal bar - Used in China.
Rectangular bar - Used in China and Russia

Out of them, trapezoidal form is the best, though its discharge coefficient is little less than that of round bars and commercially not available. Another advantage is that these can be cleaned more easily than any other forms. A trapezoidal grate commonly used in China is shown in Fig. 3.15 with its dimensions. A comparison of different grates [24] is given in Table 3.1 for reference. The grate length is determined by the width of the gallery. Soviet researchers recommended [24] that the grate length should be shorter for better discharge coefficient and structural strength.

TABLE 3.1: Comparision Of Various Forms Of Grates

SL.NO	FORM OF GRATES	DISCHARGE COEFFICIENT	CLOGGING	AVAILABILITY
1	Round bar	Slightly greater than trapezoidal	Heavy	Commercially available
2	Trapezoidal	Moderate	Light	To be processed
3	Rectangular	Smallest	Moderate	Commercially available

3.2.2.5 Hydraulic Design Principles

The flow of water both over the trash rack and in the trench below is spatially (gradually) varied. During the passage of stream flow over the trash rack the discharge in the stream gradually diminishes as same water enters into the trench due to the gravity and is a case of steady nonuniform flow with a decreasing discharge. Similarly the flow in the trench (channel) below gradually increases as the water is added along the trench from the original stream flow and is a case of steady nonuniform flow with an increasing discharge. These addition and diminution of water will cause disturbance in the energy or momentum content of the flow. As a result, the hydraulic behavior of flow, over and within the trench is complicated. Several researchers proposed different empirical formula to describe the hydraulic principles of flow over and below the grated trench weir.

3.2.2.5.1 Flow over Trench weir

The flow over trench weir is spatially varied with decreasing discharge. The following assumptions are made to have a dynamic equation of the flow of this type.

(a) The flow through grate slots or perforated plate is hydraulically considered as slotted orifice or round orifice.

(b) The specific energy is constant along the flow and the angle of inclination of rack with the horizontal (θ) equal to zero.

(c) The entrance to the reach of the rack may be regarded as a broad-crested weir

Consider a stream channel of width, b with a bottom rack of length L across it. Referring Fig. 3.16, Q_1 represents the total main channel flow of which certain quantity; Q_3 is diverted through the slot and the residual discharge Q_2, flows down the channel. The depths of flow at the beginning and end of the opening are, respectively y_1 and y_2; the corresponding velocities are v_1 and v_2. The specific energy [14] at any section of the channel is

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gb^2y^2} \tag{3.3}$$

where, E = specific energy

y = depth of flow.

g = acceleration towards gravity.

$\frac{V^2}{2g}$ = velocity head.

Q = discharge through the channel before the flow entering the rack.

As per initial assumption $\frac{dE}{dx} = 0$

Differentiating the equation 3.2 we have the following equation

$$\frac{dy}{dx} = \frac{Qy\left(-\frac{dQ}{dx}\right)}{gb^2y^3 - Q^2} \tag{3.4}$$

where, $-\frac{dQ}{dx}$ is the discharge withdrawn through a length dx of the rack. The equation 3.4 is the general dynamic equation for the flow under consideration. The discharge through the rack depends upon the effective head on the rack. Since the direction of flow through the trash rack composed of parallel bars is nearly vertical, the energy loss in the process is negligible. So the effective head on the rack is practically equal to specific energy. E. Mostkow, USSR [14] analysed the flow through the rack by assuming the specific energy of flow to be constant all over the rack and the assumption was found to agree with the experiment [14]. He expressed the discharge through the length dx of the rack by

$$-\frac{dQ}{dx} = \in C_d b\sqrt{2gE} \tag{3.5}$$

where, $\in$ = Opening ratio

= Ratio of the opening area to the total area of the rack surface

C_d = Coefficient of discharge through the openings [Table 3.2][24].

Rearranging the equation 3.3 the discharge can be obtained by the equation

$$Q = b\,y\,\sqrt{2g(E-y)} \tag{3.6}$$

Substituting the value of Q and –dQ/dx as above in equation 3.4 and simplifying, we get

$$\frac{dy}{dx} = \frac{2 \in C_d \sqrt{E(E-y)}}{3y-2E} \tag{3.7}$$

Integration of this equation gives the equation of the flow profile as

$$x = \frac{-E}{\in C_d}\left(\frac{y}{E}\right)\sqrt{1-\frac{y}{E}} + C$$

Again considering fig. 3.18, for x = 0, y = y_1 and by substituting the values the integration of constant will be

$$C = \frac{E}{\in C_d}\left(\frac{y_1}{E}\right)\sqrt{1-\frac{y_1}{E}}$$

So,

$$x = \frac{E}{\in C_d}\left[\frac{y_1}{E}\sqrt{1-\frac{y_1}{E}} - \frac{y}{E}\sqrt{1-\frac{y}{E}}\right] \tag{3.8}$$

For y=0 equation 3.8 gives the length of the rack required for a complete withdrawal of the main flow through the rack, i.e.

$$x = L_0 = \frac{E}{\in C_d}\left(\frac{y_1}{E}\sqrt{1-\frac{y_1}{E}}\right) \tag{3.9}$$

Again from equation 3.6, we have,

$$Q_1 = b y_1 \sqrt{2g(E-y_1)} = b y_1 \sqrt{2gE}\sqrt{1-\frac{y_1}{E}}$$

$$\Rightarrow \qquad \sqrt{1-\frac{y_1}{E}} = \frac{Q_1}{b y_1 \sqrt{2gE}}$$

Substituting the above value in equation 3.9, we have

$$L_0 = \frac{Q_1}{\in C_d b \sqrt{2gE}}$$

If we introduce a term the clogging coefficient or percentage of opening in the trash rack likely to be clogged with a symbol $\in_c$ then we have

$$L_0 = \frac{Q_1}{\in \in_c C_d b \sqrt{2gE}} \tag{3.10}$$

As per initial assumption (c)

$$Q_1 = C'_d b E^{1.5} \quad \text{(Discharge formula for broad crested weir)} \tag{3.11}$$

where, C'_d = Coefficient of discharge for broad crested weir and have average value of 2.8

So specific energy at any section of stream over the rack

$$E = \{Q_1/(C'_d b)\}^{(2/3)} \tag{3.12}$$

Thus E may be computed if the incoming discharge Q_1, b and C'_d are given. The discharge of a partial withdrawal from the main flow through the rack [Fig.3.16] is

$$Q_3 = Q_1 - Q_2$$
$$= b\, y_1\, \{2g(E-y_1)\}^{0.5} - b\, y_2\, \{2g(E-y_2)\}^{0.5}$$
$$= b(2g)0.5\,\{y_1(E-y_1)^{0.5} - y_2(E-y_2)^{0.5}\}$$
$$= b\, y_1\, (2g)^{0.5}(E-y_1)^{0.5}[1 - y_2(E-y_2)^{0.5}/\, y_1(E-y_1)^{0.5}]$$
$$= Q_1\, [1 - \{y_2\, (E-y_2)^{0.5}/\, y_1\, (E-y_1)^{0.5}\}]$$

by substituting the value of Q_1 as in equation 3.12 , we have

$$Q_3 = C_d b E^{1.5}\left[1 - \frac{y_2\sqrt{E-y_2}}{y_1\sqrt{E-y_1}}\right] \tag{3.13}$$

The width of rack along flow in different conditions can be computed from equations 3.10, 3.13 and 3.8, by substituting the value of E.

TABLE 3.2: Recommended Value of C_d

SL.NO.	GRATE FORM	GRATE SLOPE	OPENING RATIO (∈)			
			0.3	0.4	0.5	0.6
1	Trapezoidal	0.00 0.20	0.52 0.48	0.48 0.44	0.44 0.40	0.40 0.38
2	Rectangular	0.00 0.20	0.46 0.44	0.42 0.40	0.39 0.36	0.36 0.34

3.2.2.5.2 Flow Inside the Trench

The flow inside the trench weir is spatially varied with increasing discharge. The design developed by Julian Hinds [15] for side channel spillway has been adopted for calculating the parameters of the trench. Here an appreciable portion of the energy loss is due to the turbulence by mixing of the added water and the water flowing in the channel. Because of the resulting high and uncertain losses, the momentum equation will be found more convenient than the energy equation in solving the problem.

Consider the longitudinal section of a trench weir as in Fig. 3.17

The equation $$y = \frac{1}{g}\int_0^x \left[V\frac{dV}{dx} + \frac{q}{Q}V^2 \right] dx \qquad (3.14)$$

is the basic equation for the water surface profile in the side channel, when water flowing over the crest of the weir and enters the channel.

where , y = the theoretical ordinate y of the water surface curve, measured downward from the line B-P

V = average velocity of water particles in direction of channel

q =inflow in a unit distance

Q = total flow at point corresponding to y.

The solution of the above basic equation is based on the two assumptions.

(i) The discharge/ inflow into the trench in the entire length of waterway is uniform, i.e if q is the inflow per meter length of waterway then total discharge at a section distance x from the upper end of the trench is given by $Q = qx$

(ii) The velocity- distance relation may be expressed by an exponential relation

$$V = ax^n \qquad (3.15)$$

where, 'a' and 'n' are arbitrary constants and x denote respectively the average velocity in direction of channel and distance from the upper end of the trench. Differentiating the equation 3.15, we have,

$$\frac{dV}{dx} = n\,a\,x^{(n-1)}$$

So substituting the values of $\frac{dV}{dx}$, V and Q in the equation 3.14, we have

$$y = \frac{1}{g}\int_0^x \left[ax^n\left(nax^{n-1}\right) + \frac{q}{qx}\left(ax^n\right)^2 \right] dx$$

$$=\frac{1}{g}a^2\left[x^{2n}\frac{n+1}{2n}\right]$$

$$=\frac{n+1}{n}\frac{V^2}{2g}$$

$$=\frac{(n+1)}{n}.h_V \qquad (3.16)$$

where , $\dfrac{V^2}{2g}$ = h_v = the velocity head.

The trench weir channel design is completely determined by the above formulae if the cross-sectional shape and the values of a and n are known. The cross sectional shape is generally governed by economy of excavation. Taking the condition of minimum excavation the following derivation gives the relation between a, n, area A, width of the channel at the water surface T.

For minimum excavation at any section the sum of the depth of flow (d) and the ordinate 'y' should be a minimum.

So, D = y+d should be minimum for minimum excavation.

$$D=\frac{n+1}{n}\frac{V^2}{2g}+d=\left(\frac{n+1}{n}\right)\frac{Q^2}{2gA^2}+d$$

Differentiating both sides with respect to d we have,

$$\frac{dD}{dd}=-\frac{n+1}{n}\frac{Q^2}{gA^3}\frac{dA}{dd}+1=-\frac{n+1}{n}\frac{Q^2}{g}\left(\frac{T}{A^3}\right)+1 \quad \left(\text{Since } \frac{dA}{dd}=T\right)$$

For D to be a minimum $\dfrac{dA}{dd}=0$

Therefore $\dfrac{n+1}{n}\dfrac{Q^2T}{gA^3}=1$

$$\Rightarrow \quad \frac{n+1}{n}\frac{Q^2}{gA^2}=\frac{A}{T}$$

$$\Rightarrow \quad \frac{n+1}{n}\frac{V^2}{2g}=\frac{A}{2T}=y$$

$$\Rightarrow \quad y=\frac{a^2}{2g}\left(\frac{n+1}{2n}x^{2n}\right) \qquad (3.17)$$

Having chosen a value for either 'a' or 'n' the corresponding value of the other required to give the greatest economy at a specified point can be found from this equation. A trial procedure is best for the equation of trapezoidal channel. First assume a depth of channel; compute A and T and find h_v for a specified value of n then compute the corresponding discharge from equation

$$Q' = (2gh_v)^n \tag{3.18}$$

For a velocity-distance relation as in equation 3.15, straight water surface and a curved longitudinal bottom profile inside the trench the value of 'n' should be 0.5 [15] . If Q' is equal to the required discharge the trial depth is correct; otherwise make a new trial with V, x and n determined, the corresponding value of a is computed from equation 3.15.

The cross-section of the gallery may be rectangular or trapezoidal. Its bottom grade can be constant in each section or varies throughout the length as per the general relation of water surface level and bed profile of the trench for different values of 'a' and 'n'. The flow within the trench is subcritical. To find out the control section the practical way is to set it at the downstream end of the channel, assuming the normal water depth at the end being (0.9-0.95) h_{cr}, where h_{cr} is the critical water depth for the diverted flow. The computation begins at this control section and proceeds towards the upstream by step method or a special program.

The practice for the threshold velocity is that at the beginning of the channel it should be 1.5 - 2 m/sec depending on the slot width or the particle size of bed load coming into the channel. An air clearance of 0.2 to 0.3m should be given to ensure the free flow of water in the gallery.

3.2.2.6 Design Criteria of CBIP

The Central Board of Irrigation and Power has standardised the dimensions of trench weir based on the following guide lines [11]

(1) The trash rack slope should be of about 1 in 10 in the direction of flow.

(2) The trench itself is provided with a bed slope of not less than 1 in 25 in the flow direction of the diverted flow.

(3) The water way should be adequate to pass the design flood and the gate operation controls should be above the maximum floor level.

(4) The trash-rack area of opening is adequate to draw the entire lean season flow and desired diversion during flood even if 50% of the effective area of the trash rack is clogged.

(5) Flushing discharge of about 25 to 50 percent may be provided in diversion to cater to the flushing requirements at the intake and desilting chamber.

Some suggested dimensions of trench weir for discharge 1 to 12.5 cumecs with riverbed width of 20m and trapezoidal section of trench with 30mm spacing of bars are shown in the Table 3.3 and in Fig. 3.18

TABLE.3.3: Recommended Minimum Width of Trench

SL.NO.	DISCHARGE (CUMECS)	WIDTH (M)
1	Upto0.5	1.5
2	1.0	2.0
3	5.0	2.5
4	12.5	3.0

3.2.3 Siphon Intakes

Siphon type intakes or power conduit system are generally used in penstock or closed conduit intakes in small hydro projects. The uses of this intake have been made in several countries like USA, China, Russia and Europe. In India siphon intake integrated with vertical turbines have been installed on low head canal fall SHP scheme in some northern states (Punjab) and yet to be adopted and accepted widely. The major advantage of this intake is the cost reduction by eliminating the gates and hoist arrangement with efficient water flow regulation to the turbine. Due to its simple principle of operation and low operation maintenance cost, unlike conventional intake, it has a wide application in diversion type or reservoir type SHP schemes in inaccessible remote areas. In the following paras some of advantages, disadvantages, and principles of operation and design are briefly discussed.

3.2.3.1 Advantages and Disadvantages of Siphon Intake

Practice shows that this type of intake and power conduit system has following advantages [1], [25], [50].

1. Elimination of the intake gate and its hoist, even the inlet valve of the turbine, if single penstock layout is adopted. It saves about 20% to 30% of initial cost of the conduit system.

2. Compared with gated intakes, the plant flow can be cut-off quickly and completely, thus improving the runaway condition of the unit as well as the safety condition during maintenance and minor overhaul.
3. Siphon-type conduits are more convenient and reliable in operation, thus reducing the maintenance and operation cost.
4. In cold regions ice problems can be greatly reduced and shows better efficiency towards the reduction of silt intake as well.
5. During speed regulation by the governor, the water hammer and speed rise are similar to gated intakes. During emergency shut-down by the vacuum breaker, the water hammer is much less than gated intake
6. It is suitable for hydropower plants where sudden load rejection require immediate response.
7. As contrast to the discharge over a weir the siphon intake utilises much greater head difference between upstream and down stream water level
8. It is especially useful where crest length space at diversion site is limited.

The siphon intake has following disadvantages

1. Its application is limited to diversion and reservoir type plants with fluctuation of fore bay or reservoir water level being less than 3-4m.
2. When siphon is primed, a further rise in water level in reservoir results in only a small increase in discharge whereas discharge rate of weir increases considerably with level of water above weir.
3. Conduits are expensive to construct.
4. The abrupt priming of a siphon spillway produces a sudden rush of water downstream. This can be overcome by providing a series of siphons to prime at different water levels.

3.2.3.2 Working Principle of a Siphon Intake

The working principle of siphon intake is same as that of siphon spillway. The Fig. 3.19 shows the hydraulic mechanism of self-priming of siphon duct. When the water is at full reservoir level and it stands upto the crest of the spillway, the pressure in the siphon duct is atmospheric. Further increase of water level starts the water flowing over the crest. As the exit end is such that no air can enter from that end, hence the entrapped air in the top portion of the siphon duct above the sheet of water, gradually sucked by the flowing water. The sucking of air develops the dropping of pressure inside the siphon and ultimately gives rise a pressure less than atmospheric, and creates a necessary pull for the commencement of siphonic action inside the duct. The process of self-priming is slow .So a number of priming devices have been developed which expedite the process of priming even without the rise of water level above the crest. The siphonic action once started continues as long as the water level in the forebay or reservoir remains above the full reservoir level. When the water level in the reservoir falls to the inlet level of the deprimer or air vent incorporated in the structure, air enters into the siphon duct and breaks the siphonic action. Some of the details of priming and depriming system are discussed in the succeeding paras.

3.2.3.3 Design Principles of Siphon Intake

Although the working of siphon spillway and siphon intake is same but the design parameters of some of the components differ to satisfy the smooth operation of powerhouse. Considering a typical siphon inlet section as in Fig. 3.20, the maximum vacuum at the ceiling of the throat will be [50].

$$h_V = h_0 + \Delta h + (Z_{max} - Z_{min}) + \frac{V_0^2}{2g} + \sum h_l$$

$$= h' + (1 + \sum h)\frac{V_0^2}{2g}$$

where, h_v = The maximum vacuum at the ceiling of the throat.

h' = static vacuum.

$(1 + \sum h)\frac{V_0^2}{2g}$ = Kinetic vacuum.

V0 = The average velocity at the throat.

$\sum h_l$ = The total head loss from the inlet to the throat. The value should be taken within .17 to .25, as per model test and field observation [50] .

In practice the design vacuum is generally taken as (3-5) m and no greater than (5-7) m, depending on the altitude at the site. The field tests show that small design vacuum should be used as far as possible as [50] :

1. More fine air bubbles in the siphon flow would be released in low-pressure areas, thus resulting in an air pocket at the ceiling of the throat, which might lead to instability of the unit output, even to breaking-up of the siphon.

2. Smaller vacuum is essential for quicker breaking-up of the siphonic action.

Again to avoid cavitations the maximum permissible negative head at the throat should be

$h_v < \frac{p_a}{w} - \frac{p_v}{w} + h_l$ (Neglecting the difference of area in the inlet and throat section.)

$h_v < \left(\frac{p_a}{w} - \frac{p_v}{w}\right) + \left(\frac{v_0^2 - v_l^2}{2g}\right) + h_l$ (Taking different area of cross-section at inlet and throat section)

Where, $\frac{p_a}{w}$ = Atmospheric pressure at mean sea level, 10.33m.

$\frac{p_v}{w}$ = Vapour pressure head at throat section, 2.43m at normal temperature

V_l = Velocity at intake of the siphon conduit.

V_0 = Velocity at throat.

From the equation above, the limiting value of h_v increased if $V2 < V_3$. This can be achieved by reducing the area at the exit end of the siphon as compared to that at the throat. To keep the head loss to a minimum the change of cross-section of siphon or velocity along the passageway should be made gradually, not abruptly. So that it will vary linearly with the distance from the inlet. The inlet velocity generally adopted as 1.2-1.5 m/sec and that at the throat is at least of 2.2-2.5 m/sec for the desirable self-priming during operation. The velocity in the penstock is generally taken as 4m/sec, depending on the magnitude of the plant head and other considerations.

The penstock discharge and altitude gives a limitation to the application of the siphon intakes. In case of greater discharge (15 to 20m^3/sec), the rectangular inlet is preferable, because of its smaller possible height at the throat and that of less discharge (<15 to 20 m^3/sec) circular inlet is preferable.

The following configuration should be adopted for the siphon to function efficiently.

- The included angle α should be reduced.
- Inlet should be conical and inclined to the upstream to make the water enter into the inlet from its whole circumference.
- The bend ratio R_n/d_0 should be 2.5 in order to reduce the bend loss to a minimum.
- The divergence angle of the conical inlet should not greater than 10^0-12^0.
- The inlet should be rectangular for easy fabrication.

- The minimum water level in the forebay should be determined based on the plant operation and the forebay configuration. The maximum water level should be lower than the crest elevation by 0.1-0.15m
- The minimum submergence depth of siphon inlet should be as per the following[50]

(a) Gorden's formula.

$h_s/d = 1.7F_r$ (For symetrical approaching flow)

$h_s/d = 2.3F_r$ (For asymetrical approaching flow)

where, d = Depth of inlet.

F_r = Froud number.

h_s = Depth of submergence.

(b) Prof. J.Knause's formula

$h_s/d = 1.5F_r$ ($F_r<0.33$ and $V_1=1$ to 3 m/sec.)

3.2.3.4 Priming and Shut-Down of Siphon Intakes

Siphon penstock intakes are generally designed to be primed by vacuum pumps and emergency shut down by vacuum breakers. The self-priming is used as a supplementary measure during operation. For shorter priming time a larger capacity of vacuum pumps should be used. Priming can be made by different methods such as

1. Priming by gates or stoplogs on side spillway.
2. Container-type primer
3. Self-priming.

Priming by gates or stoplogs on side spillway:

A side spillway is generally located in the forebay, with its crest to be even with the maximum water level. If a gate or stoplogs of appropriate height are specifically installed on the spillway crest, it can temporarily raise the water level in the forebay to prime the siphon.

Container-type primer:

The Fig. 3.21 shows the container-type primer. It consists of

(i) A filling system of pipes.

(ii) An air pipe system.

(iii) A cylindrical container.

(iv) Discharge pipe.

(v) Different valves to operate the system.

First the conduit is filled up upto the level of water in the forebay, together with the container upto its uppermost water level. After that the valve 5 and 3 are closed and open respectively. The water level in the container falls down and the presser in the air space decreased due to its volume expansion. As a result the water level in the conduit rises up until it reaches to the height as designed. Then the valve 3 is closed and the unit is put into operation. During emergency shutdown, the vacuum breaker 5 or the valve 4 is opened to cut off the flow in the throat. The minimum volume of the container can be determined by the formula derived from Boyle-marriotte's law [24], [50]

$$V_{min} = \frac{P_a V_0}{P_a - \gamma_w h_v} F$$

where, V_{min} = Minimum volume of container (m^3)

P_a = Atmospheric pressure in the site (t/m^2)

V_0 = Volume of the air space in the conduit before priming (m^3)

γ_w = Specific weight of water (t/m^3)

h_v = The design vacuum of the siphon.(m)

F = Factor of safety =1.1

The container is of cylindrical-type or box type and can be arranged in the forebay dam or separately located and its strength should be varied as per collapsing resistance of the container. The top of the container should be located lower than the minimum water level by 0.5m, so as to fill up the container under the lowest water level and to fully use the volume of the container. The height of the discharge pipe should be greater than design vacuum, so as to discharge the water from the container into tailrace or atmosphere when the water level of the container is under its lowest position. As a result, the (Z_{max}-Z_d) as shown in Fig 3.21 should be greater than 7-10m, depending on the magnitude of the plant flow and the design vacuum.

The diameter of the pipe can be calculated by general hydraulic principles and time for filling up the conduit and the container, as well as for emptying it. The diameter of the vacuum breaker can be adopted as 80,100 and 150mm according to the head and discharge of the plant, based on the experience [50].

Self-priming:

The hydraulic mechanism of the self priming can be well explained from the fig. 3.22[50]. The conduit is first of all filled upto the headrace water level by opening the valve 1. Then the valve and the vacuum breaker are closed with a certain opening of the wicket gate of the turbine. As the water in the lower leg moves down the water in the upper leg rises up due to the volume expansion of the airspace until the "hump crest" begins to over flow. The air inside gradually entrain in and carried away by the spillway flow. After full evacuation of all the air the siphonic action starts. The creation of full flow depends upon the Froude number of the flow at crest) [50] and the value of $(P/w)/(V^2/2g)$. For self-priming the velocity at the throat should be greater than 2.2 - 2.5 m/sec.

3.2.4 Intake Through Hole in Ogee Type Weir Using Self Cleaning Screen

Intake arrangement in ogee type weir using Self cleaning or Coanda- effect screens are an emerging technology that offers the potential for water diversions with high capacity, low-maintenance screening of fishes and fine debris, used for irrigation and small hydropower development. These screens have the potential to reduce capital and running costs in Indian SHP schemes. A typical layout of intake using coanda screen is shown in the Fig. 3.23. The coanda screen removes all silt particles over 1mm in diameter. So the main advantage of this type of intake is the elimination of extra desilting system/arrangement from intake water in a small hydropower scheme. In the United States the screens have been in use since the early 1980's installed in more than 15 schemes of capacity upto 11.9 MW and discharging upto $7m^3$/sec flow [29]. Their performance has been proven under severe conditions of freezing and floods. The screens can resist impact damage from free limbs and boulders during floods, due to the screen's steep angle and the high degree of frame rigidity.

The coanda effect is known to most hydraulicians, although perhaps not by name. The effect was first observed in 1910 by Henri-Marie Coanda (Pronounced "Kwan-dah"), in connection with exhaust flow from an experimental jet engine [29]. The coanda effect is the tendency of a fluid jet to remain attached to a solid boundary, flow entrainment into the jet is inhibited on the surface side. For the jet to separate from the surface there must be flow entrainment into the jet on the surface side beginning at the separation point. However, the close proximity of the surface limits the supply of flow to feed such entrainment. Thus, the jet tends to remain attached to the surface. If the surface deviates sharply away from the jet, separation will occur, but if the surface curves gradually away, the flow may remain attached for long distances. Primary applications of the coanda effect have been in aeronautics; wings and engines using the effect have achieved increased lift and thrust. The phenomenon of coanda effect can also be effectively utilised for fine debris removal applications at water- intakes and diversion structures. The following pares describes the mechanism and functioning of coanda effect screen for silt free water intake.

3.2.4.1 Mechanism of Silt Exclusion

The coanda effect screen panel consists of a row of horizontal tilted, wedge wire' bars arranged 1mm apart perpendicular to the flow of water. The important features of a coanda-effect screen installation are illustrated in Fig. 3.24. The screen panel is installed in the sloping down stream face of a hollow overflow ogee shaped weir. The frame is bolted into position top and bottom to horizontal and vertical concrete faces of hollow weir. The flow passes over the crest of the weir, across and down a solid acceleration plate, and then across and through the screen panel. The flow water passing through the screen is collected in a conveyance channel below the screen surface and beneath the weir structure. The flow passing over the screen carries fish and debris off the screen. Typical screen openings are 1mm or less, making it feasible to exclude fish eggs, larvae, and very fine debris. The screens are substantially self-cleaning for most type of debris due to the high sweeping velocity down the screen face (usually exceeding 2m/s). If the inflow to the screen is not large, all the flow will pass through the screen. For best self screen operation and wherever fish passage down the screen face is needed, it is desirable to maintain some bypass flow off the screen so that no portion of the screen is dry. For this reason, it is important to accurately know the screen capacity. As an average the coanda effect screen can draw 140 liters/second of clear water per linear meter of the screen width.

The typical screen panel is a concave arc with a radius of curvature of approximately 3 to 4m although planer screen panels can also be used. The screen is fabricated to high tolerances from stainless steel. The key feature of the coanda effect screen is the tilting of screen wires in the downstream direction to produce a shearing offset into the flow above the screen [Fig.3.25]. The typical tilt angle is 5°, but angles of 3° to 6° are available from most screen manufacturers. The screen with wedge wire spaced 1mm apart has a desilting function 90% of 0.5 mm suspended silt particles and all 1mm particles are separated, eliminating the desilting tank in many systems. Screen with finer structure are available, down to a 0.2mm screen, which separates 90% of 0.1 mm particles, but their costs are higher. Aquadyne, Inc., Healdsburg, CA, now markets one specific coanda effect screen configuration under the trade name Aqua Shear static intake screen. Some aspects of this screen design are covered under US patent 4,415,462[29] .

The crest of the weir and acceleration plate can be either an ogee-shaped profile or a simple circular arc. The objective of the acceleration plate is to provide a smooth acceleration of the flow and deliver it tangent to the screen surface at the upstream edge. The total loss of head in this type of intake is about 1m from the weir top to the intake water level.

The mechanism of water extraction through coanda screen is better explained by the Fig. 3.25. The figure shows the flow of water over the flat wire screen and over a tilted wire screen, as it would occur with and without the coanda effect. The ogee crest has established the flow direction and when water flow over the flat–wire screen or tilted-wire screen without any coanda effect, the flow separates off the high point of each wire and skips to the next wire, with essentially no flow being sheared off. Gravity, pressure forces and the curvature of the screen panel will force a small amount of flow through the screen, and as the flow continues down the screen the flow field will begin to deviate toward the screen. Once this deviation matches the tilt angle of the wires, the flow will be similar to that shown at the right of the Fig. 3.25 by the combined action of orifice flow through the slots and shearing of flow through the offset created by the tilted wires.

The coanda effect causes the flow to remain attached to the screening surface of each wire, directing the flow into the offset created at the next downstream wire. A thin layer of the flow is sheared off by the next wire, which is offset into the flow due to the tilted wire construction. The incremental discharge through the screen at any wire is a function of the flow velocity and the thickness of the sheared water layer. The elevation drop from the crest to the screen produces high velocity flow over and through the screen. Since the coanda effect keeps the flow in contact with the screening surface of each wire, even near the top of the screen, it helps in producing high capacity flow over the full length of the screen.

3.2.4.2 Advantages

The following are the advantages of use of coanda effect screen

(a) Coanda effect intake can be located at a site with only temporary access hence available head can be increased in some cases.

(b) The coanda effect screen removes all particles over 1mm or below thus reducing significantly or eliminating the desilting system. Hence simplifying the civil works of an SHP scheme.

(c) The water in the collection canal has no particles above 1mm and so flow velocity can be low, and the shape is not critical.

(d) The screen is less likely to be blocked with debris

(e) Perform well in freezing condition as the flow over the screen involves a high velocity.

(f) Lesser investment and maintenance cost.

3.2.4.3 Disadvantages

The following are the disadvantages of the system when used in the SHP scheme

(a) 1 to 1.5m head is lost at the screen intake. This makes the use of self–cleaning screen less attractive at heads of 20m or below.

(b) The particle removal characteristics of the screens will not be sufficient for some turbines, especially where very high heads or quartzite grains in the flow are involved. The most likely applications for 1mm screens are in those schemes where heads range between 30m and 100m, and which use impulse machine.

(c) Coanda effect screen has lesser water extraction capacity. It is in the range of $0.14m^3/s/m$. Hence require larger diversion weir for high design discharge of intake canal. The constraint can be overcome by positioning the screen on a side channel weir. The largest scheme to date is Forks of Butt Hydro scheme USA , which uses 56m of screen with a capacity of around $8m^3/sec$.

3.2.5 Inflatable Dam

The inflatable dam or canvas dam is really a bag, properly anchored on the base slab or the crest of a weir in a common type of headworks structure. It can be inflated or deflated by water or air filling and emptying system in order to variate the water level in the upstream. The bag is made of polymide or nylon canvas coated with polychloroprene rubber or neoprene as a protective layer. It is invented by U.S engineer N.M. Imbertson in 1956 and first installed in 1959 On the Los Angelos river in Califernia, USA. Shortly afterwards it has been rapidly developed around the world because of its economy and fast installation in headworks of small hydel plants in canal and existing system .The canvas dam less than 5m height has been standardized in different countries and used successfully.

3.2.5.1 Site selection and Layout

The site should be selected on a straight reach of a uniform river flow and stable riverbed. For silt-laden rivers, when the canvas dam is collapsed or deflated for removal of the deposited silt and debris, the silt, especially the coarse sediment would give abrasion to the bag. This problem should be properly considered during the design of the canvas dam. The schematic diagram of this type of dam with anchor system is shown in Fig. 3.26 for reference.

3.2.5.2 Types of Canvas dam

The following are different types of canvas dam usually adopted worldwide.

- **(a)** Breast-wall type canvas dam
- **(b)** Pillow type canvas dam
- **(c)** Multi-pillow type canvas dam

Breast-wall type canvas dam

In this type the dam-bag is anchored in the base slab only, with its both ends being half spherical and being stretched through the breast wall [Fig. 3.27]. The configuration of the inflated bag should be identical to the underlying curve of the breast wall, giving some pressure to the curved surface. Thus the folding gaps and wrinkles near both ends are eliminated.

Pillow type canvas dam

In this case, both ends of the bag are evenly closed by rubberized canvas and directly in contact with the vertical abutment or pier, i.e. the length of the bag is equal to the net span of the bay [Fig.3.28]. The end of inflated bag is tightly contacted with the vertical wall,thus eliminating the leakage through both ends of the bag and the wrinkle of the deflated bag.

Multi-pillow type canvas dam

In general, if the length of crest is long, there would be many intermediate piers on the crest. But in case of inflated weir these piers can be eliminated and pillows be butt-contacted one by one.

The anchorage of the bag is one of important features in the design of canvas dam. Any accident of the anchorage shall result in failures of the dam. The anchorage pattern [Fig.3.26] can be classified into

1. Single-line anchorage, and
2. Double-line anchorage

The circumferential length of single-anchored bag is longer and less stable than that of double-anchored one. Therefore the double-line anchorage is generally used.

3.2.5.3 Vibration in and Swing of canvas Dam

The canvas dam is generally filled either by air or water. The water filled double-anchored dam is more stable. Generally vibrations and swing in dam are occurred during spilling or collapsing of the dam bag. For better vibration control during spilling of water the relative spilling depth (h_e/H_1) should be less than 0.3[24]. During the process of collapsing, the water in the bag flows out from the outlet and successive vibrations and swings occur, as shown in Fig.3.29 .At the end, the "bag tail" will be formed because its exit flow through the outlet is obstructed by the collapsed bag, thus resulting in serious vibration and clapping on the base slab. To mitigate this problem, parallel tubes are installed on the inside of the bag to divert the water directly to the outlet. For silt-laden rivers, if the thickness of base slab is small large amount of silt would be deposited in the upstream after a certain period of operation [Fig3.30]. When the bag is collapsed for flood release, the upstream silting will be overflowing over the collapsed bag, which would scratch the bag. Hence a sponge layer is generally installed inside the bag to overcome this problem [Fig.3.30].

3.2.5.4 Advantages and Disadvantages

The following are the advantages of using this type of dam [24]

(1) It is actually a substitute for the steel gate and reduces the cost of headworks.

(2) The installation of dam bag is simple and quick.

(3) It creates a less obstruction to the passing flood as compared to other type.

(4) It has a good resistance to earthquake.

The disadvantages of canvas dam are as follows

(1) The service life of canvas dam is ~~only~~ around 20 years.

(2) It needs more minor overhauls during operation especially in silt-laden river.

(3) The bag is susceptible to scratch by silt and floating debris.

(4) It is difficult to repair or overhaul under water.

(5) It is sensitive to typhoons.

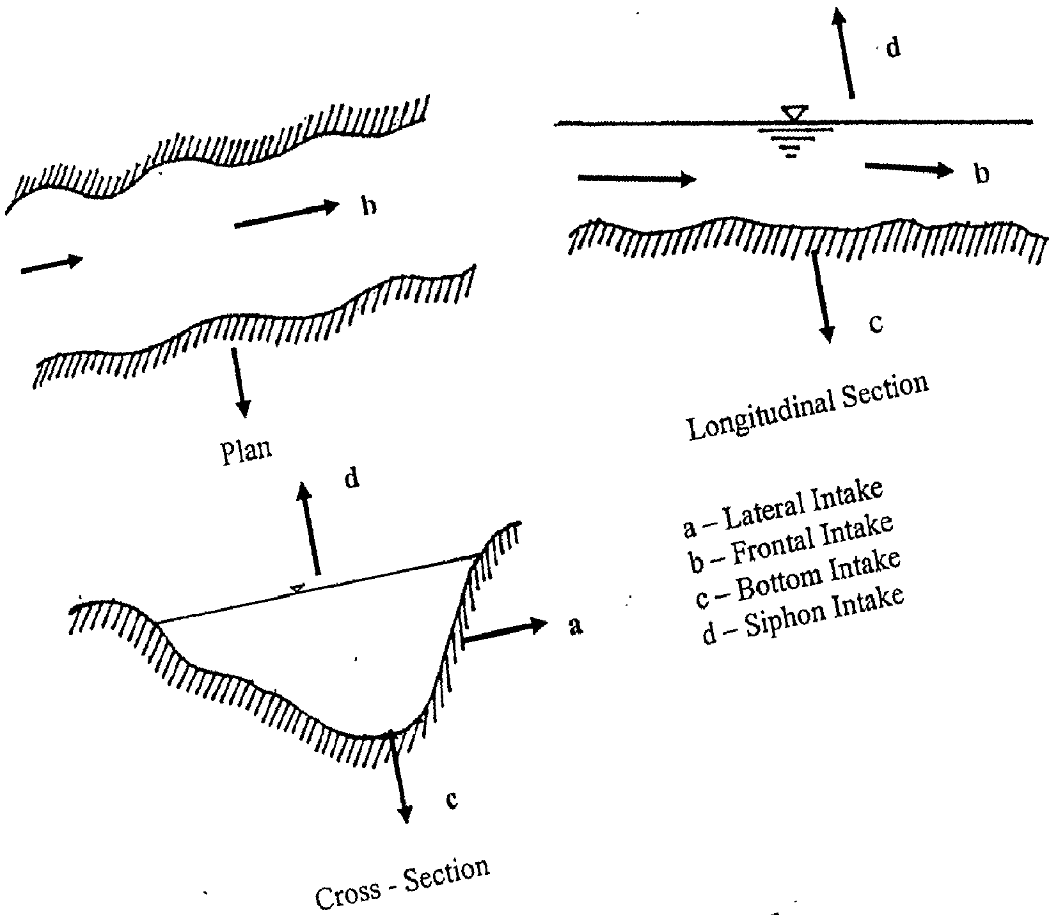
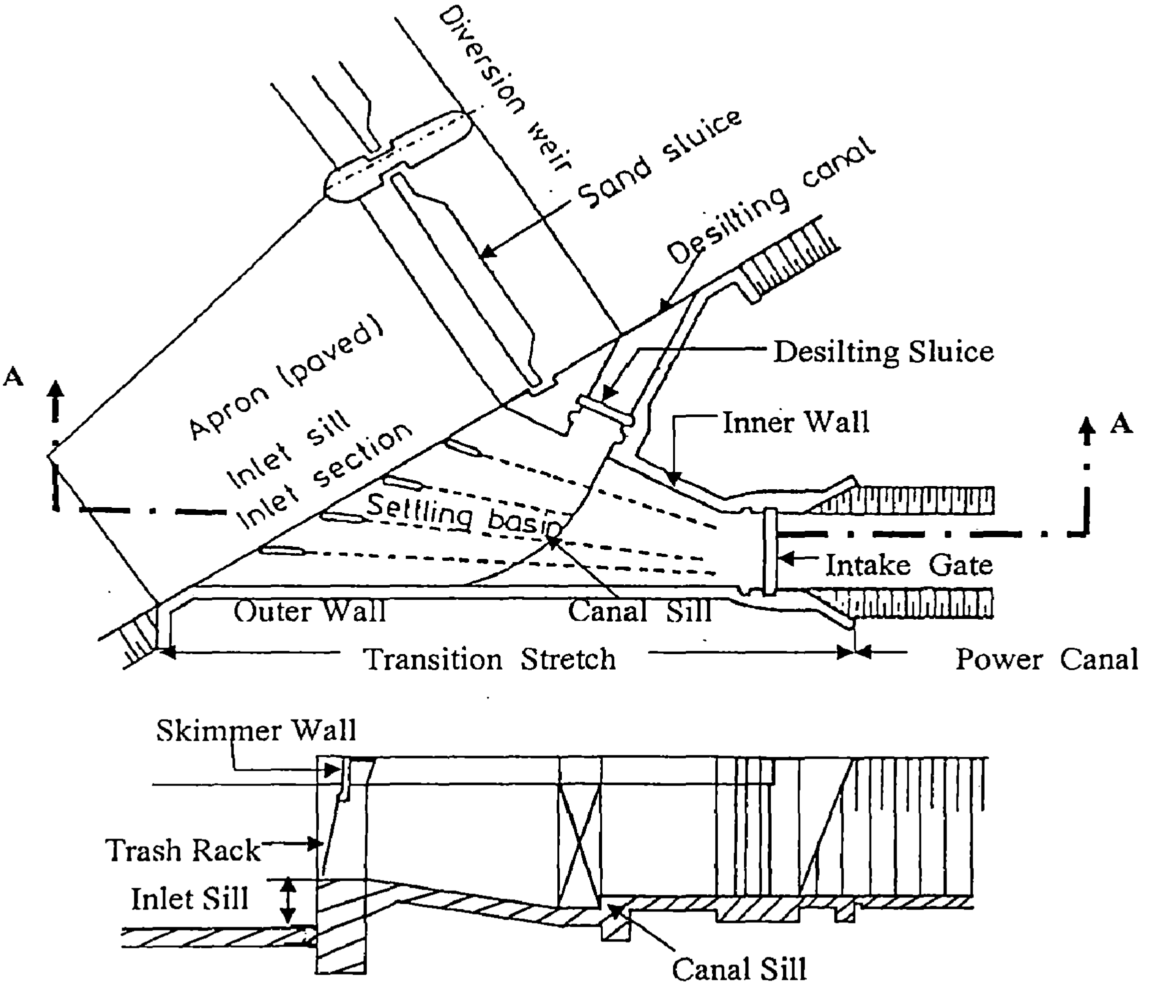


Fig.3.1: TYPES OF INTAKE



Section A- A

Fig.3.2: CANAL DIVERSION TYPE HEADWORKS

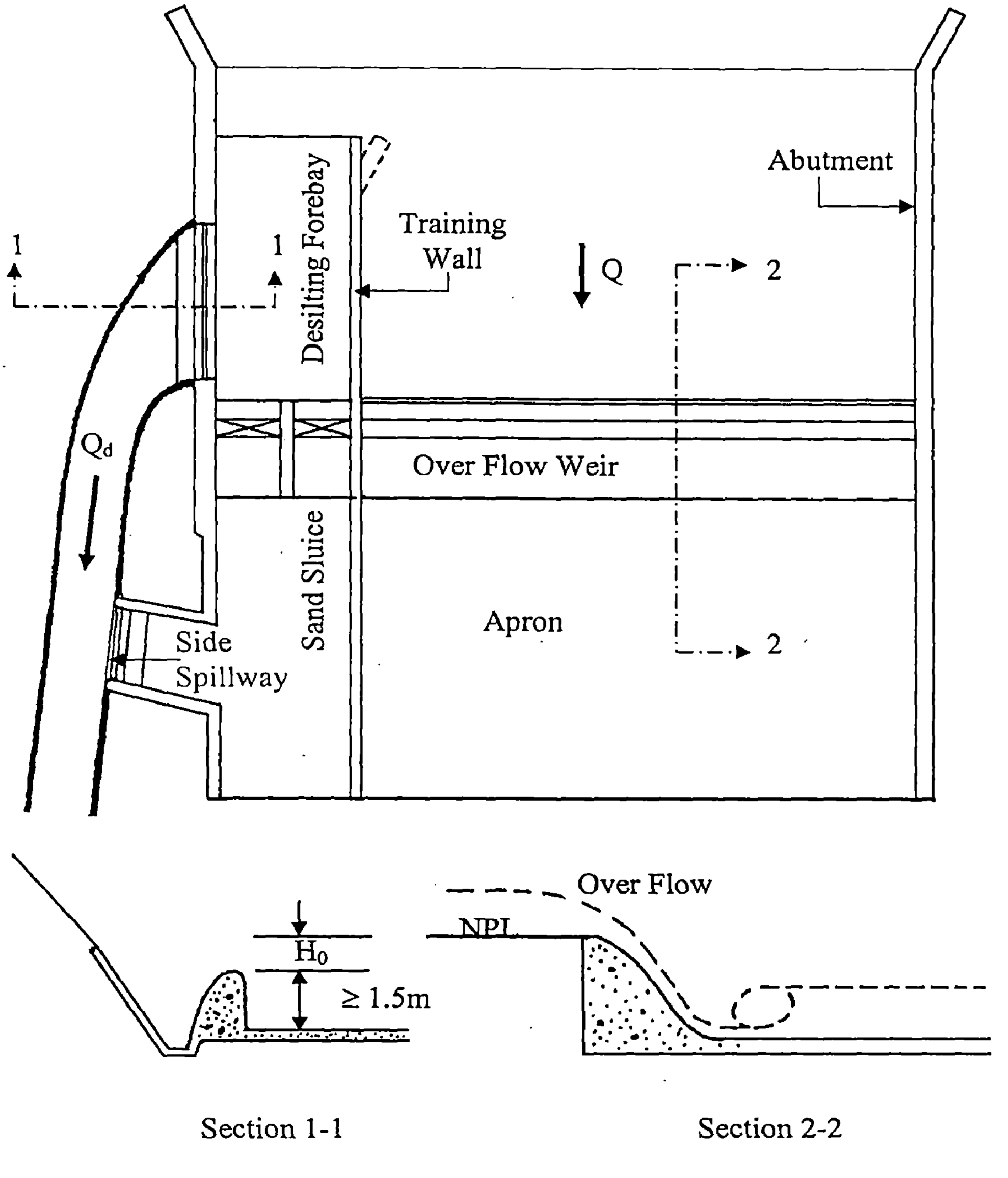
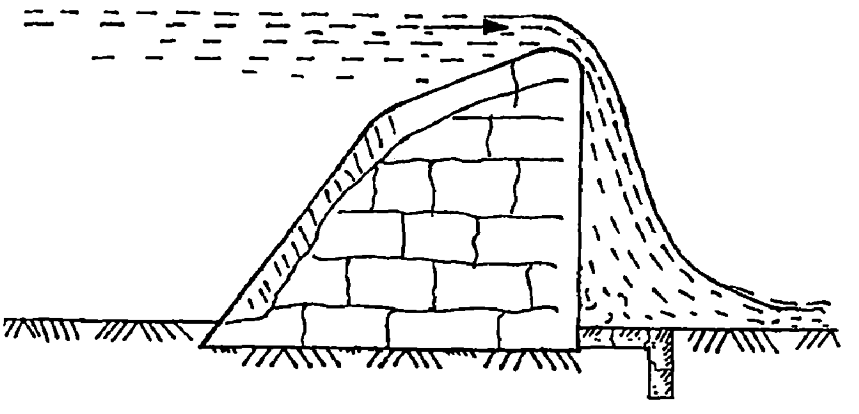
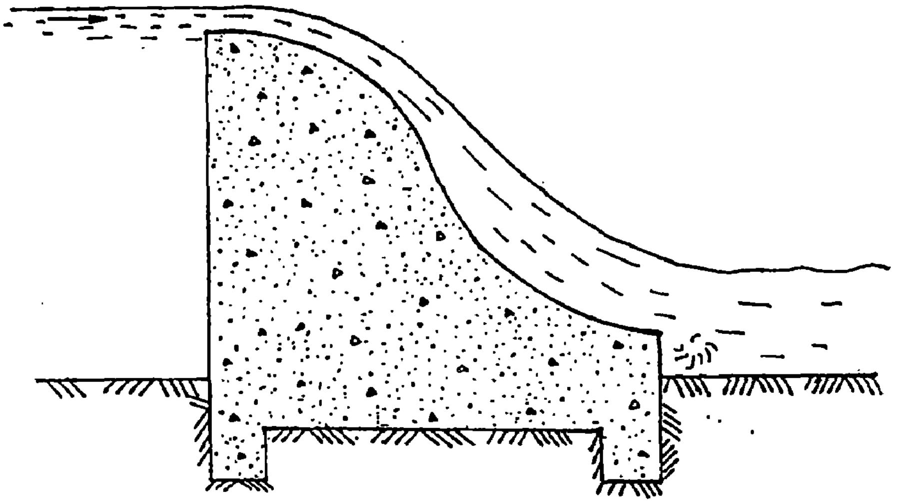


Fig.3.3: TYPICAL LAYOUT OF HEADWORKS

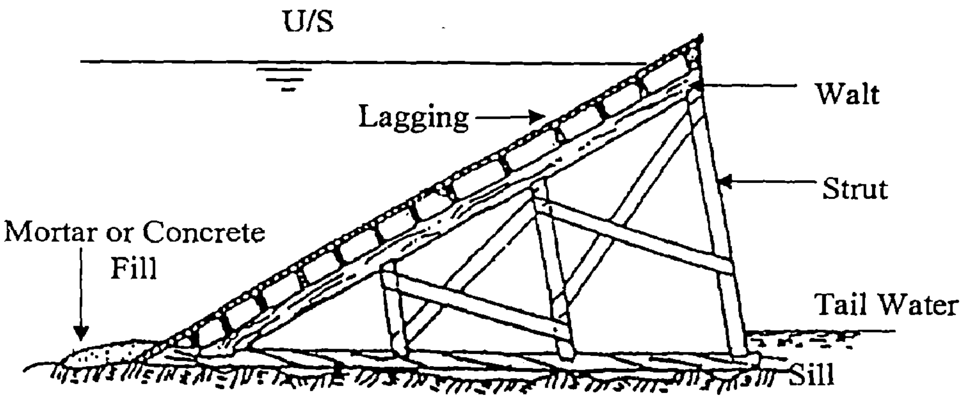


Masonry Dam



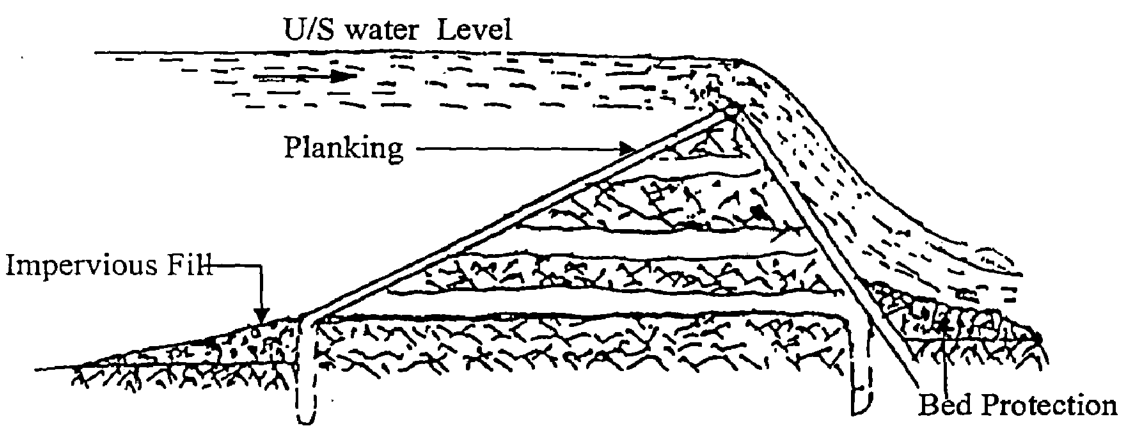
Concrete Dam

Fig.3.4: SMALL MASONARY AND CONCRETE DAM



(a)

Alternative Type of A – Frame Timber Dam



(b)

Fig.3.5: CRIB DAM

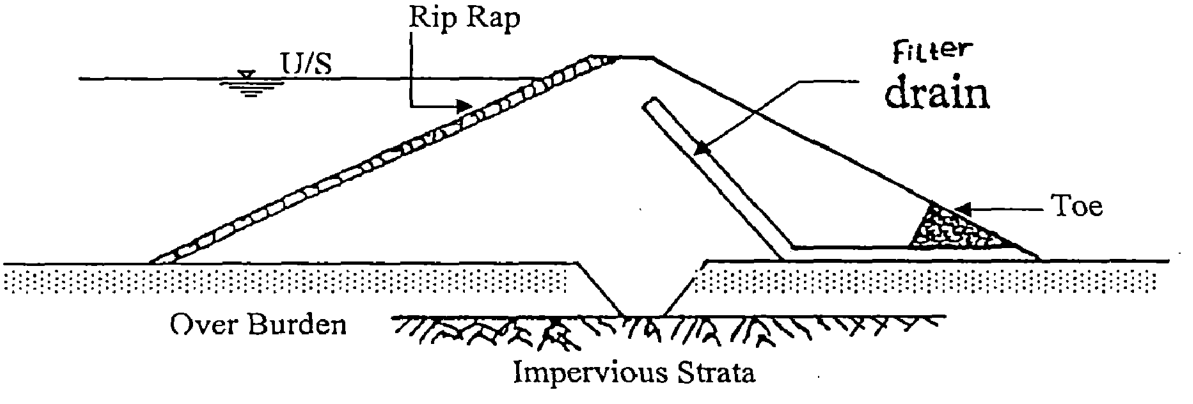


Fig.3.6: HOMOGENEOUS EARTH FILL DAM

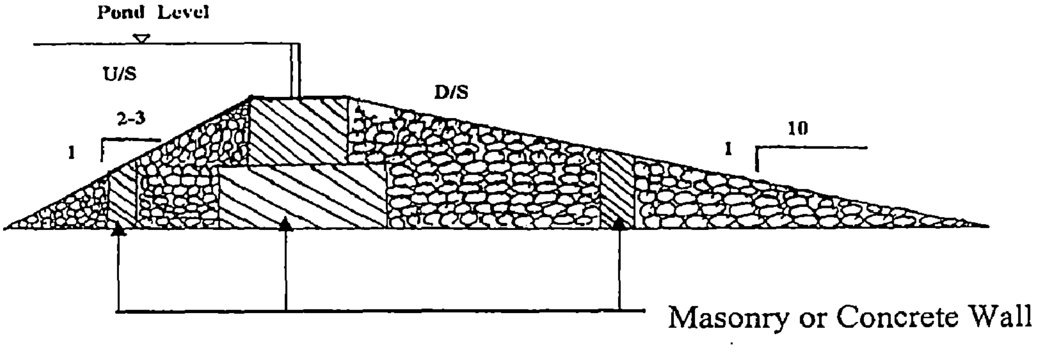


Fig.3.7: ROCK FILL WEIR

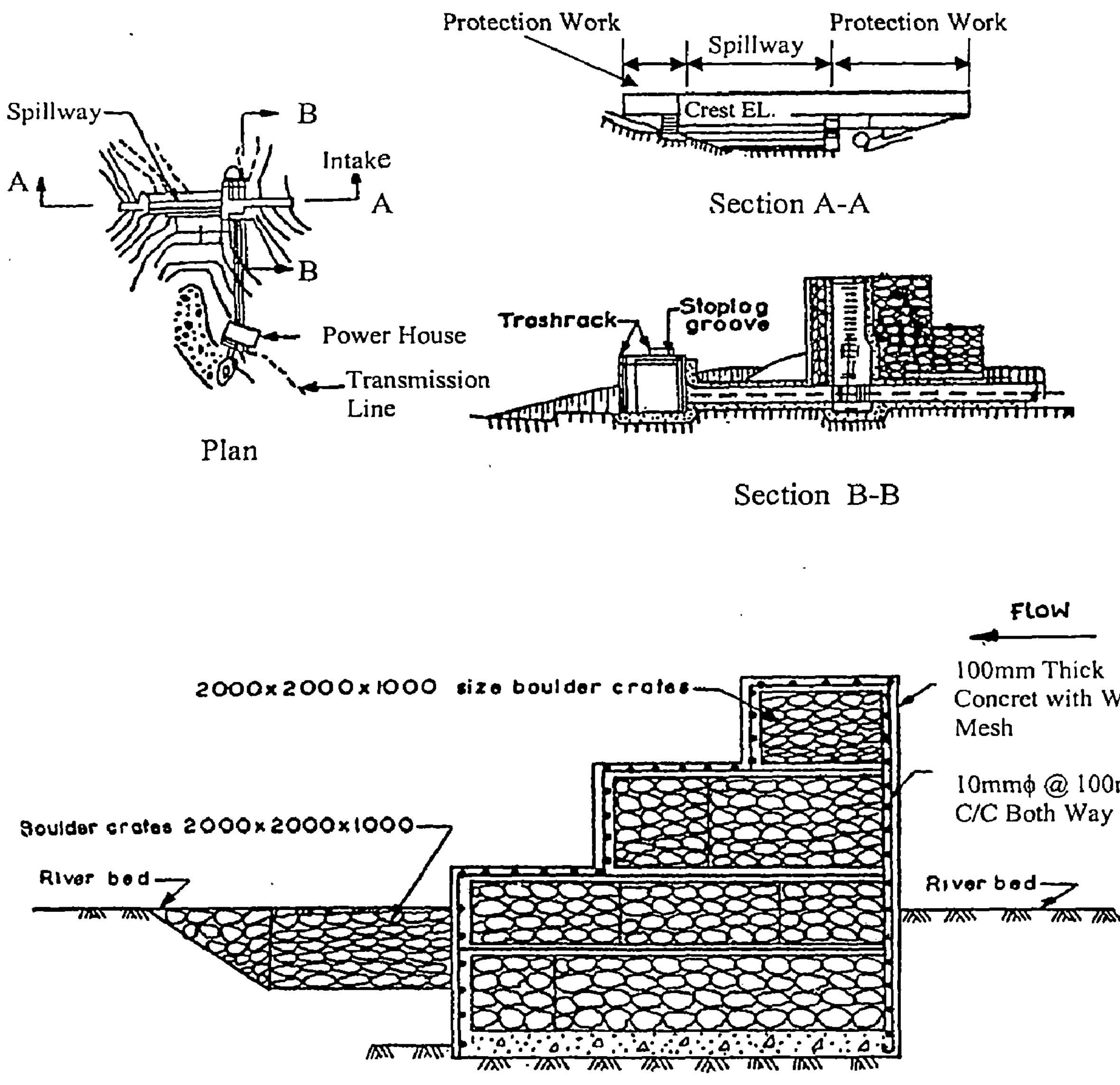
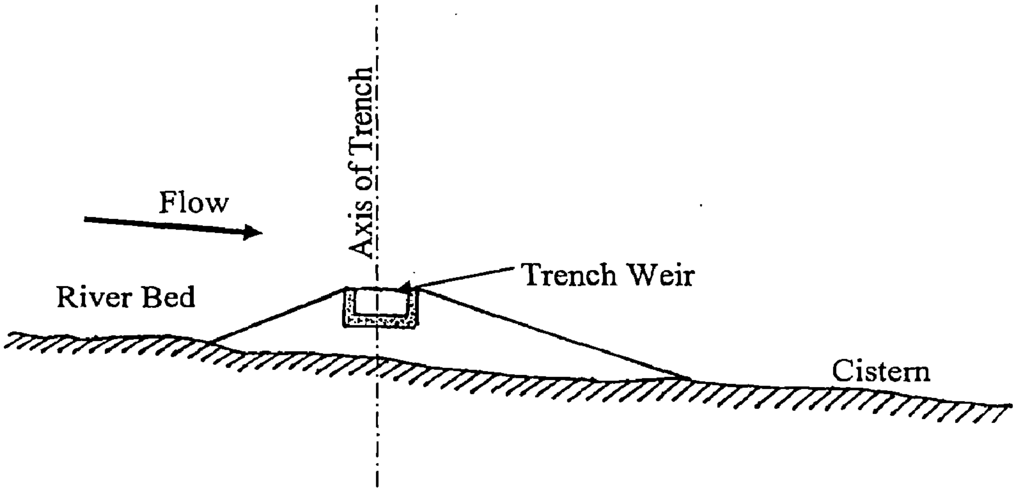
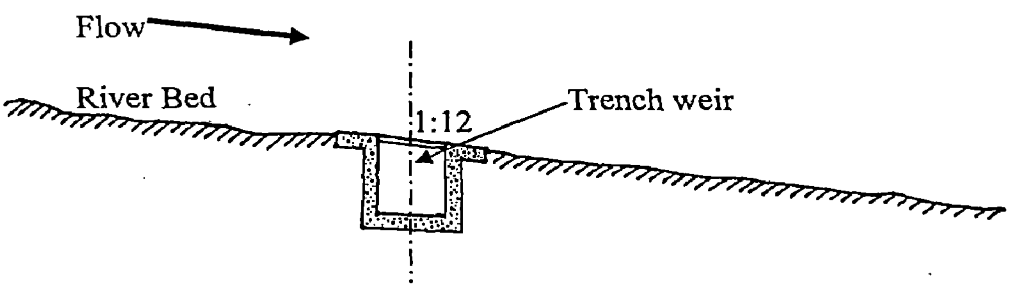


Fig.3.8: BOULDER BAR WEIR IN SMALL HYDEL SEHEME



Trench Weir Above River Bed



Trench weir in River Bed

Fig.3.9: TYPES OF BOTTOM INTAKE TRENCH WEIR

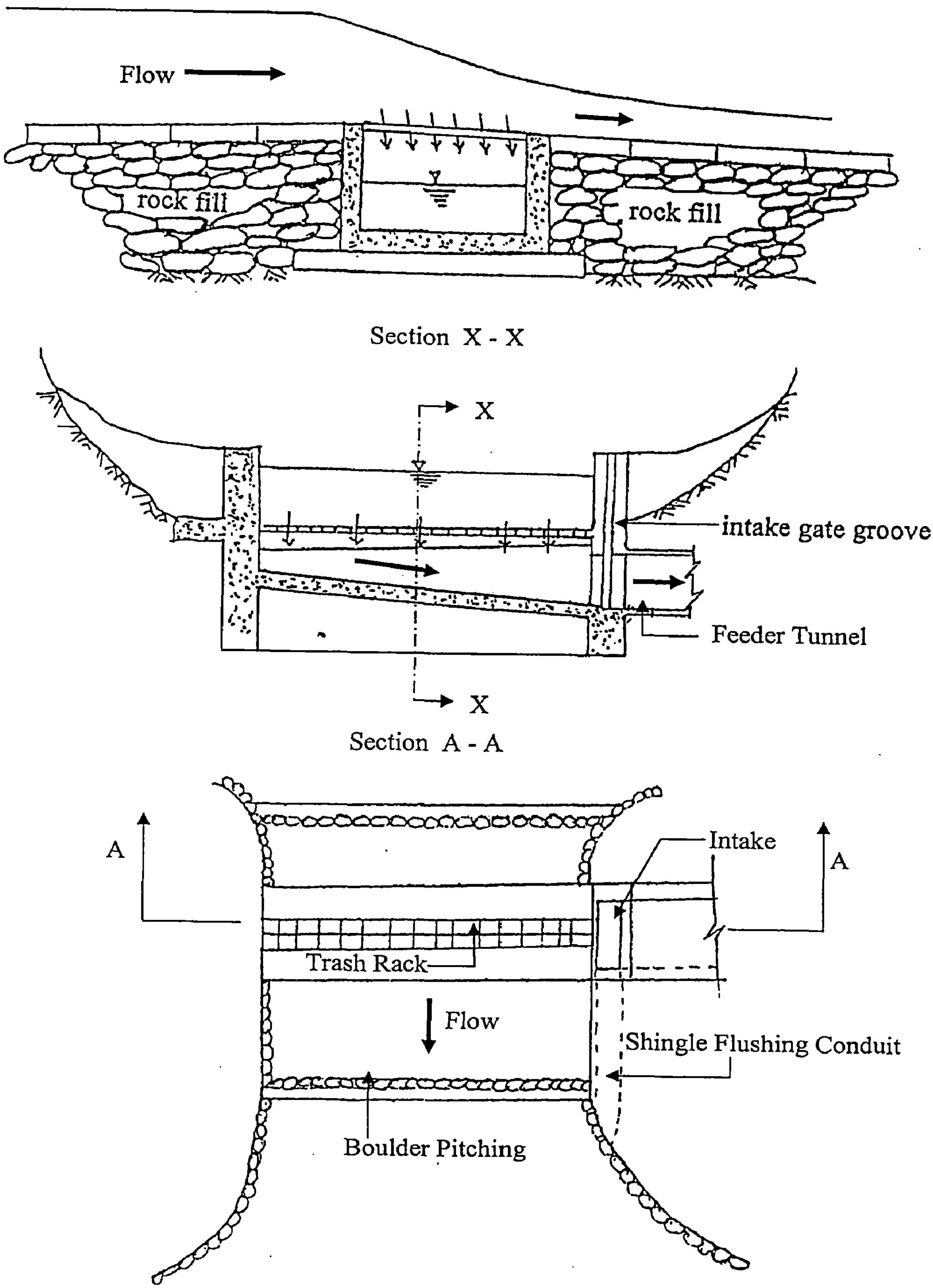


Fig.3.10: TYPICAL TRENCH WEIR

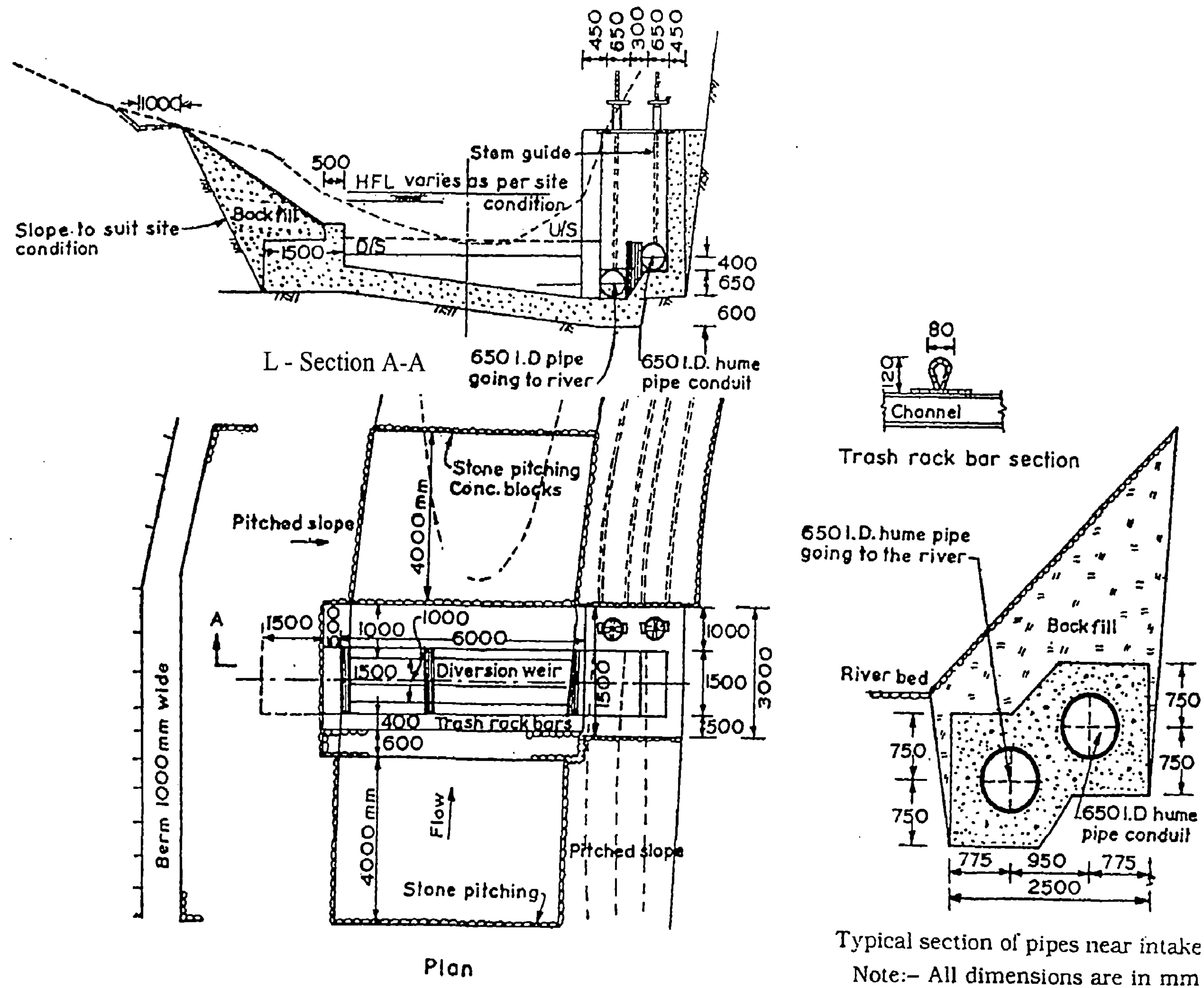


Fig.3.11: TYPICAL LAYOUT OF TRENCH WEIR WITH INTAKE WELL

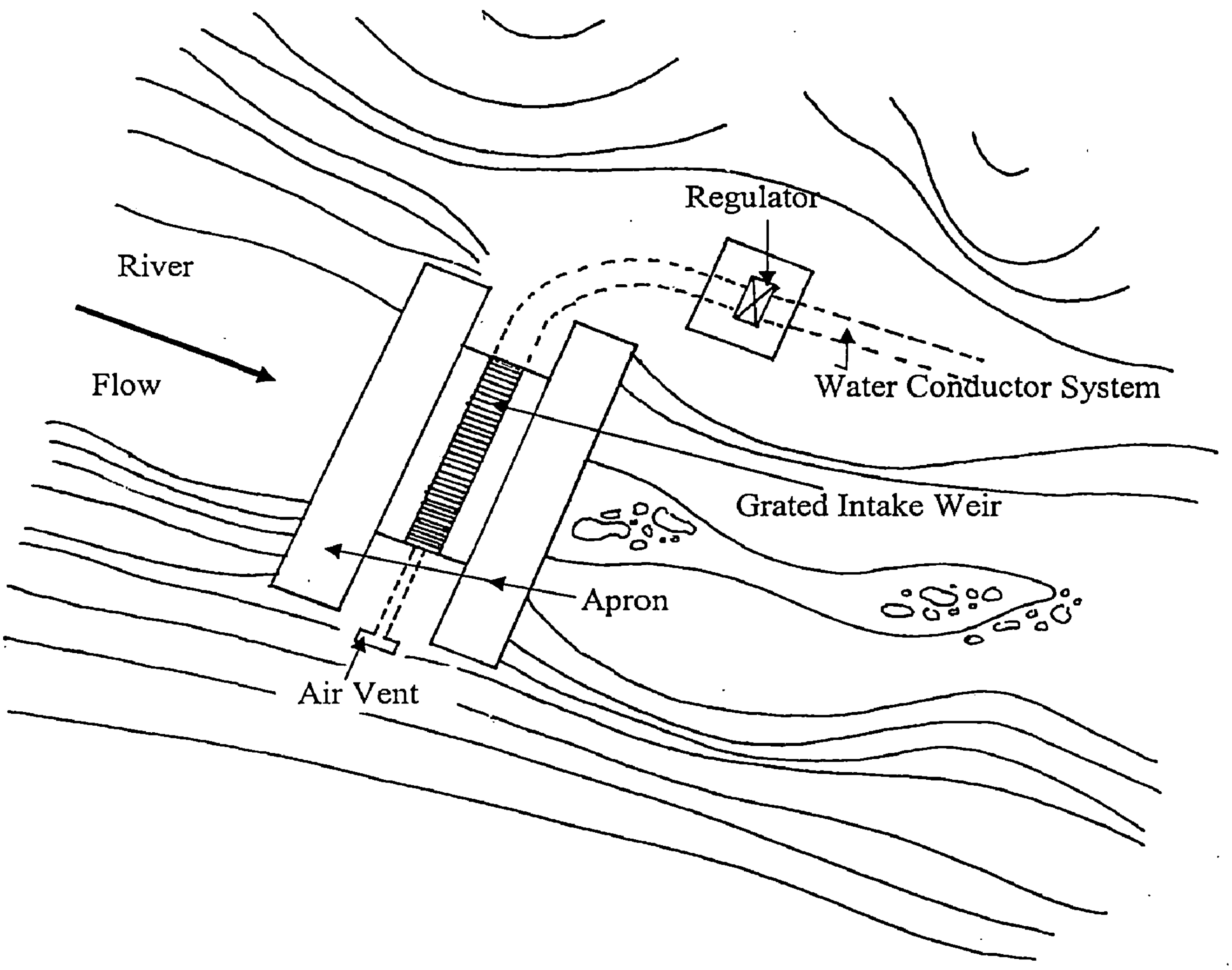


Fig.3.12: A MINI HYDRO HEADWORKS IN SHINGSHAN COUNTRY,HUBEI PROVINCE,CHINA[51]

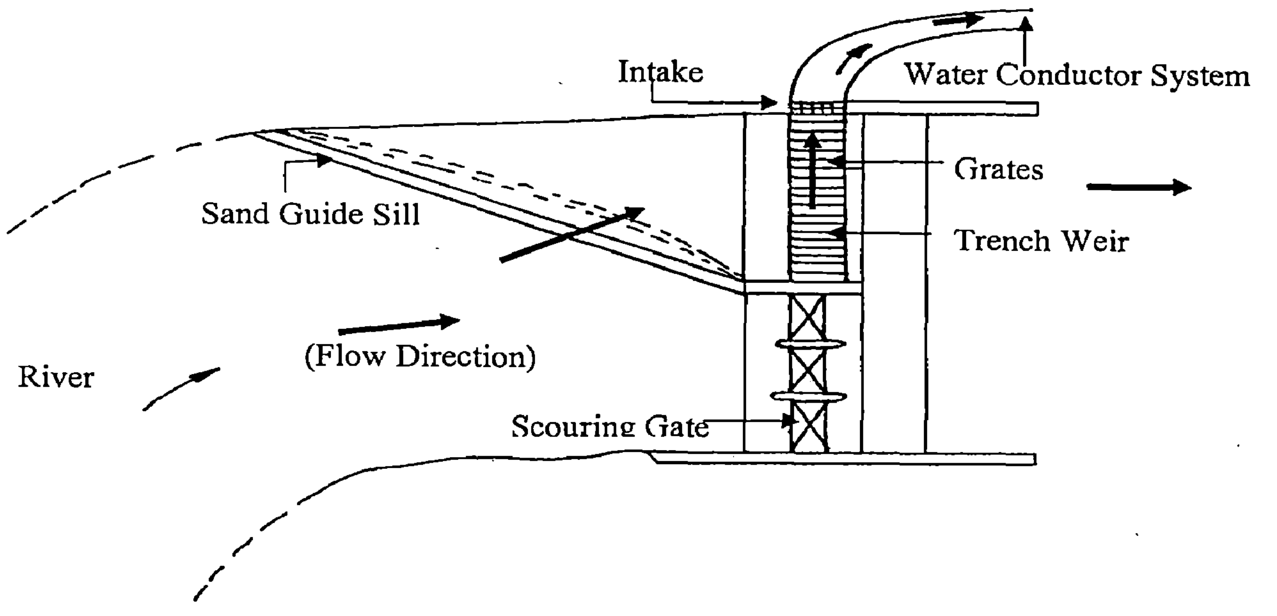


Fig. 3.13: LAYOUT WITH A SAND GUIDE SILL AND SCOURING GATE

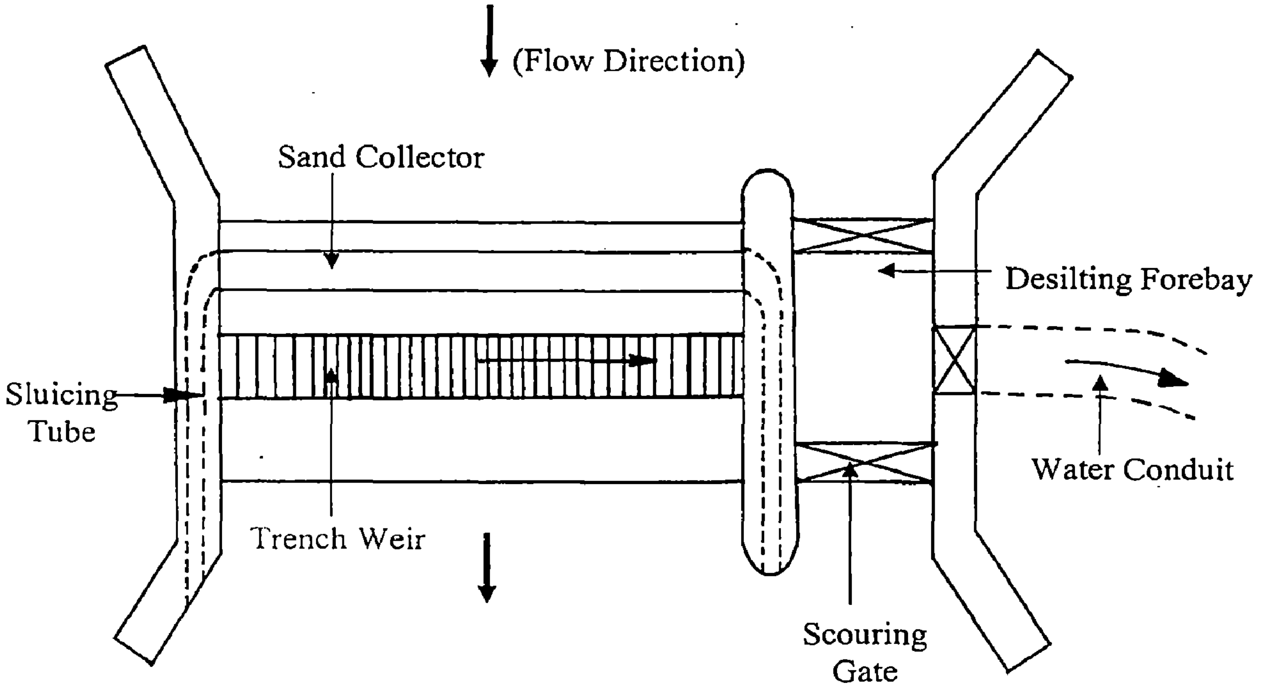
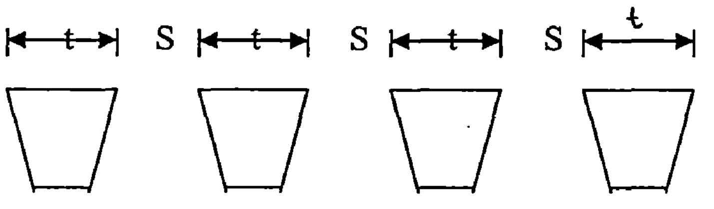


Fig. 3.14: TRENCH WEIR WITH FRONT SAND COLLECTOR



$$t = 12\text{mm to } 25\text{mm}$$
$$S = 12\text{mm to } 20\text{mm}$$
$$\epsilon = \frac{S}{S+t} = 0.4 \text{ to } 0.5$$

Fig. 3.15: DIMENSIONS OF TRAPEZOIDAL GRATES

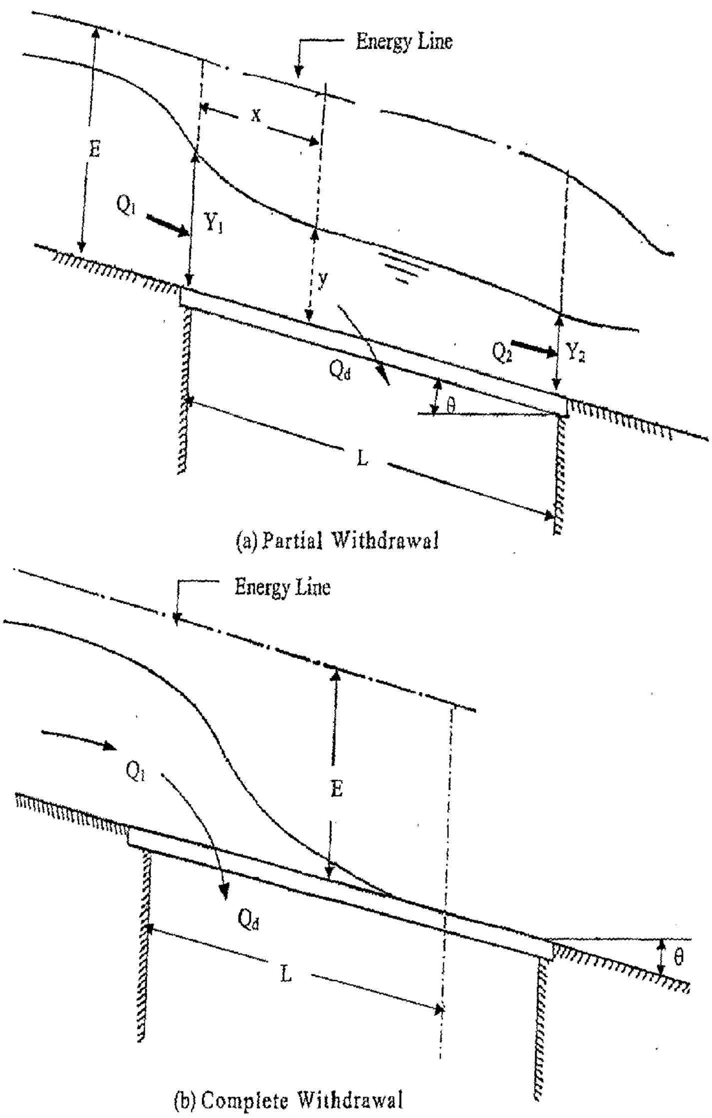


Fig.3.16: FLOW OVER BOTTOM RACKS

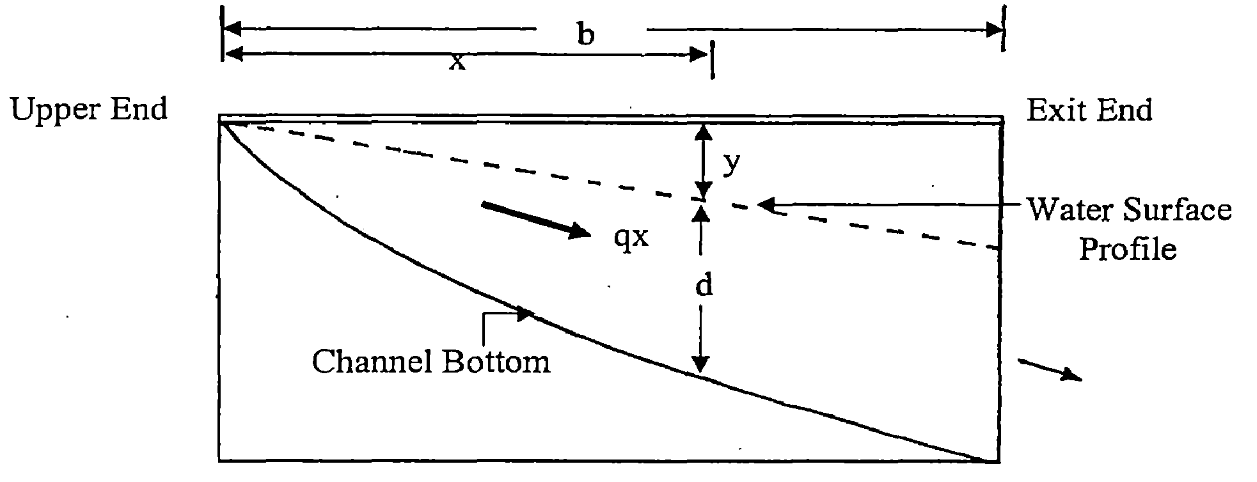
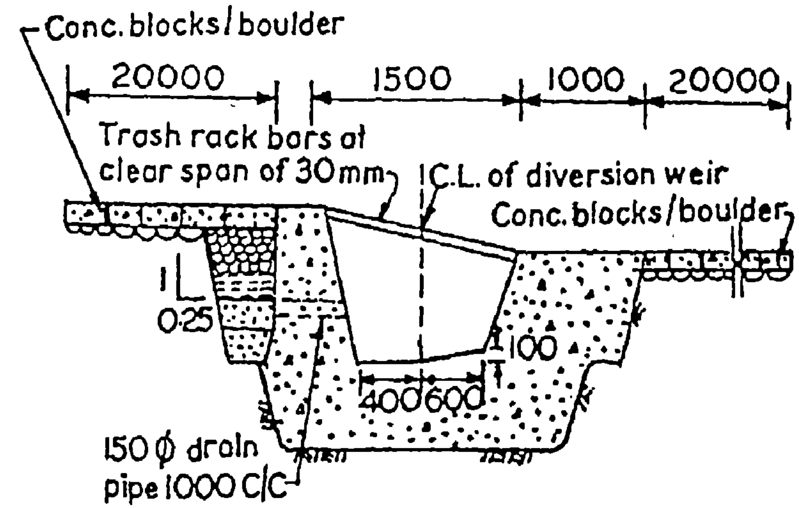
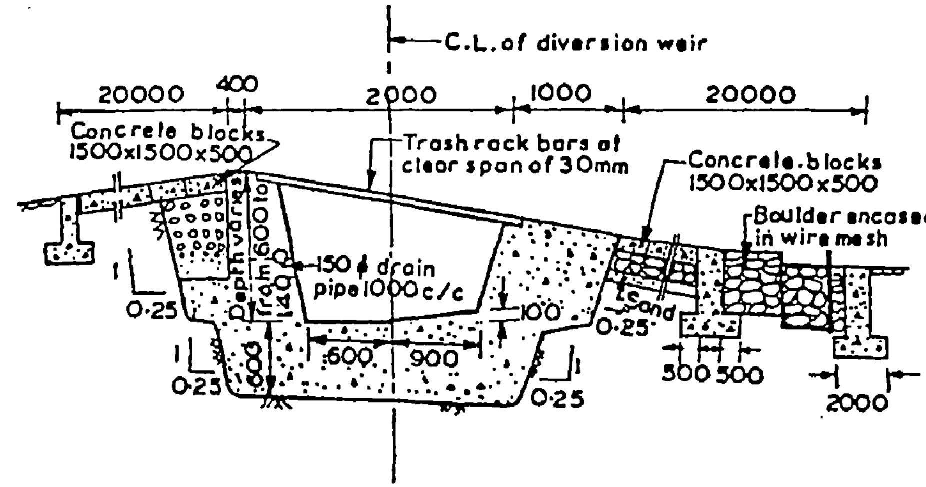


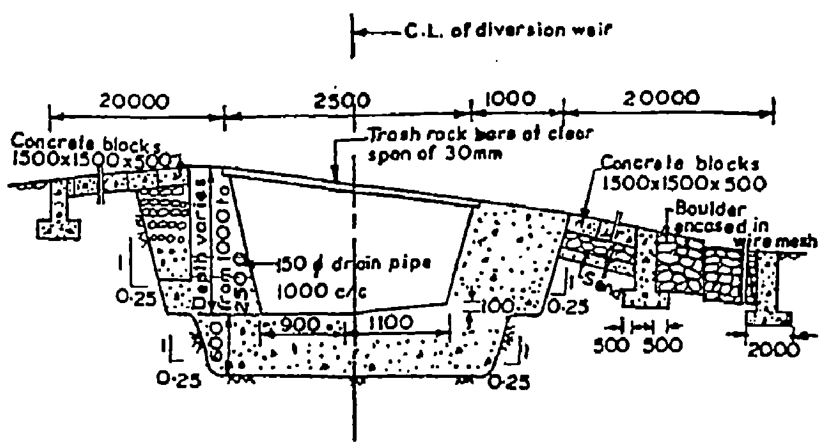
Fig. 3.17: Water surface and bed profile inside the trench weir



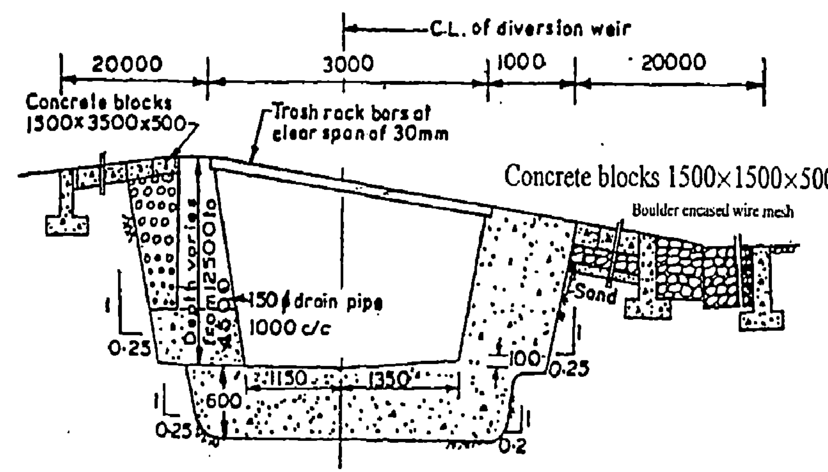
Typical section for 0.5 cumec discharge



Typical section for 1 cumec discharge



Typical section for 5 cumecs discharge



Typical section for 12.5 cumec discharge

Fig.3.18: TYPICAL SECTIONS OF TRENCH WEIR FOR 0.5 TO 12.5 CUMEC

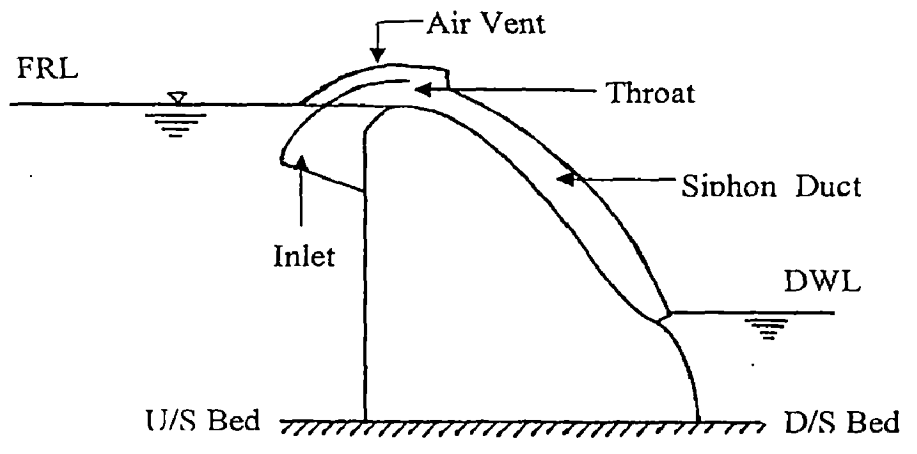


Fig. 3.19: HYDRAULIC MECHANISM OF SELF PRIMING SIPHON DUCT

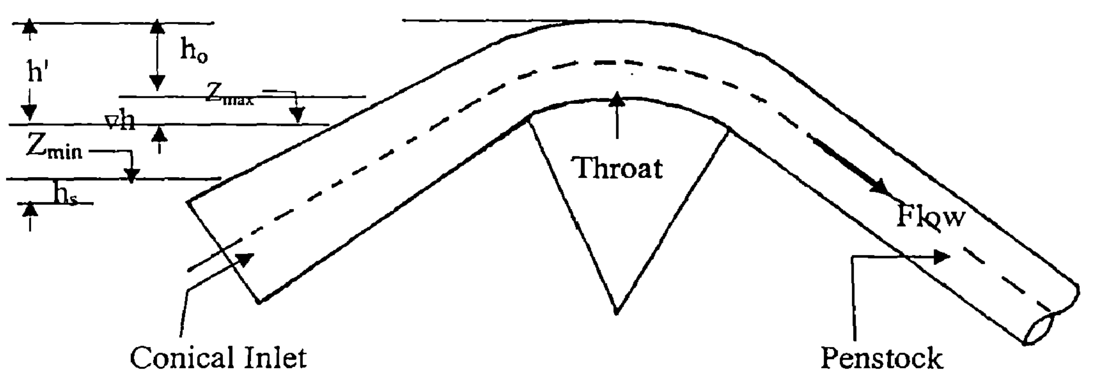


Fig.3.20: SIPHON INLET SECTION

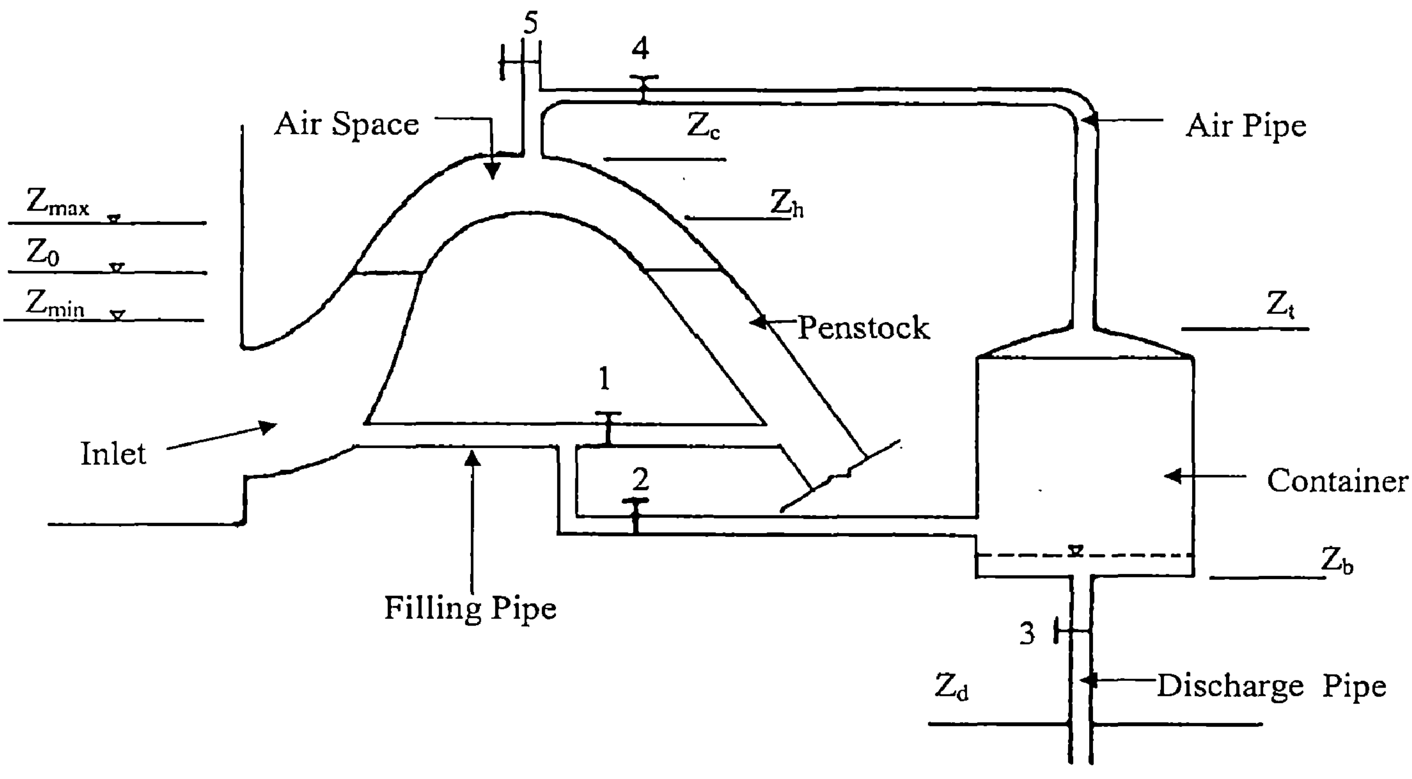


Fig. 3.21: SKETCH OF CONTAINER-TYPE PRIMER

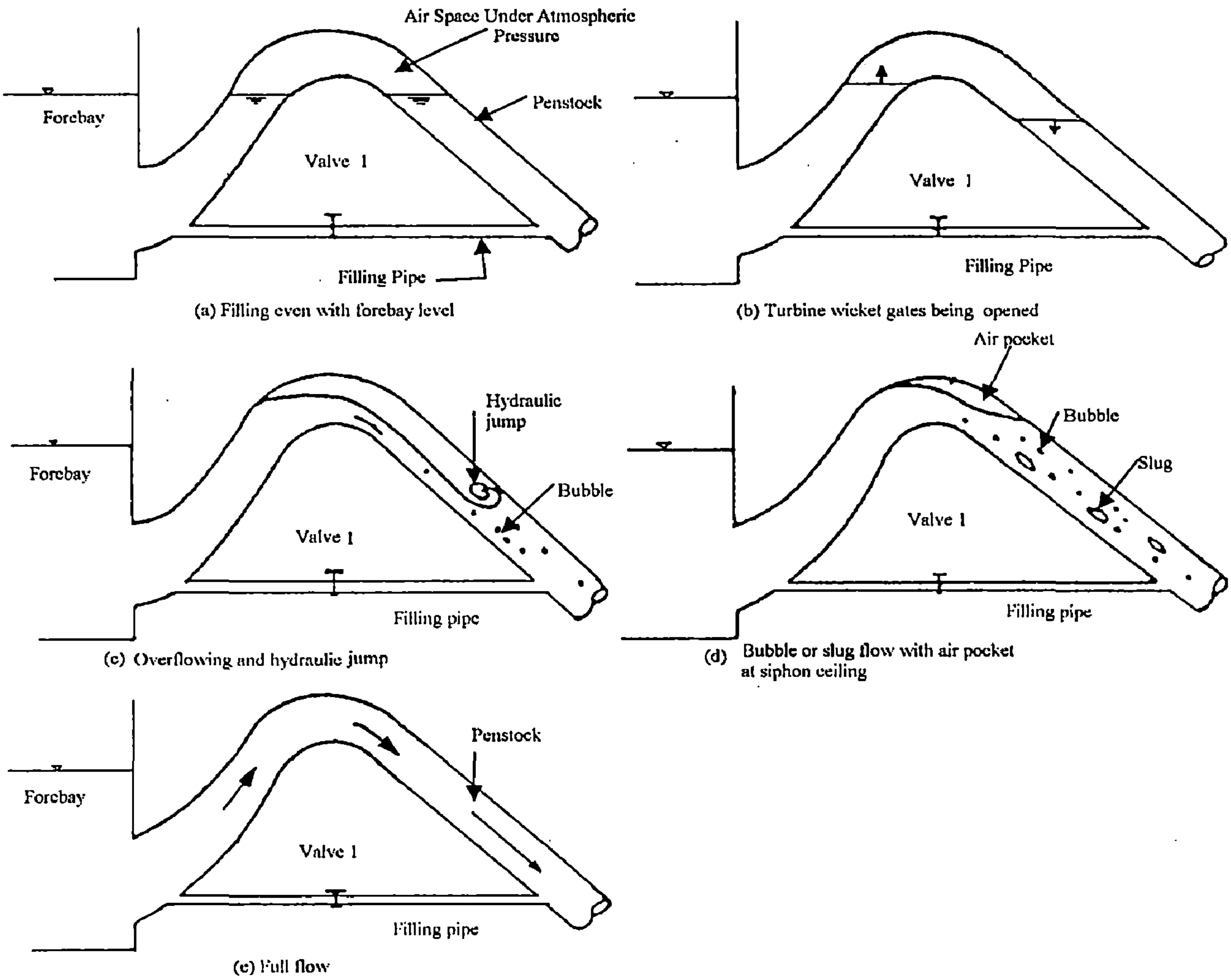


Fig.3.22: SCHEMATIC OF SELF-PRIMING

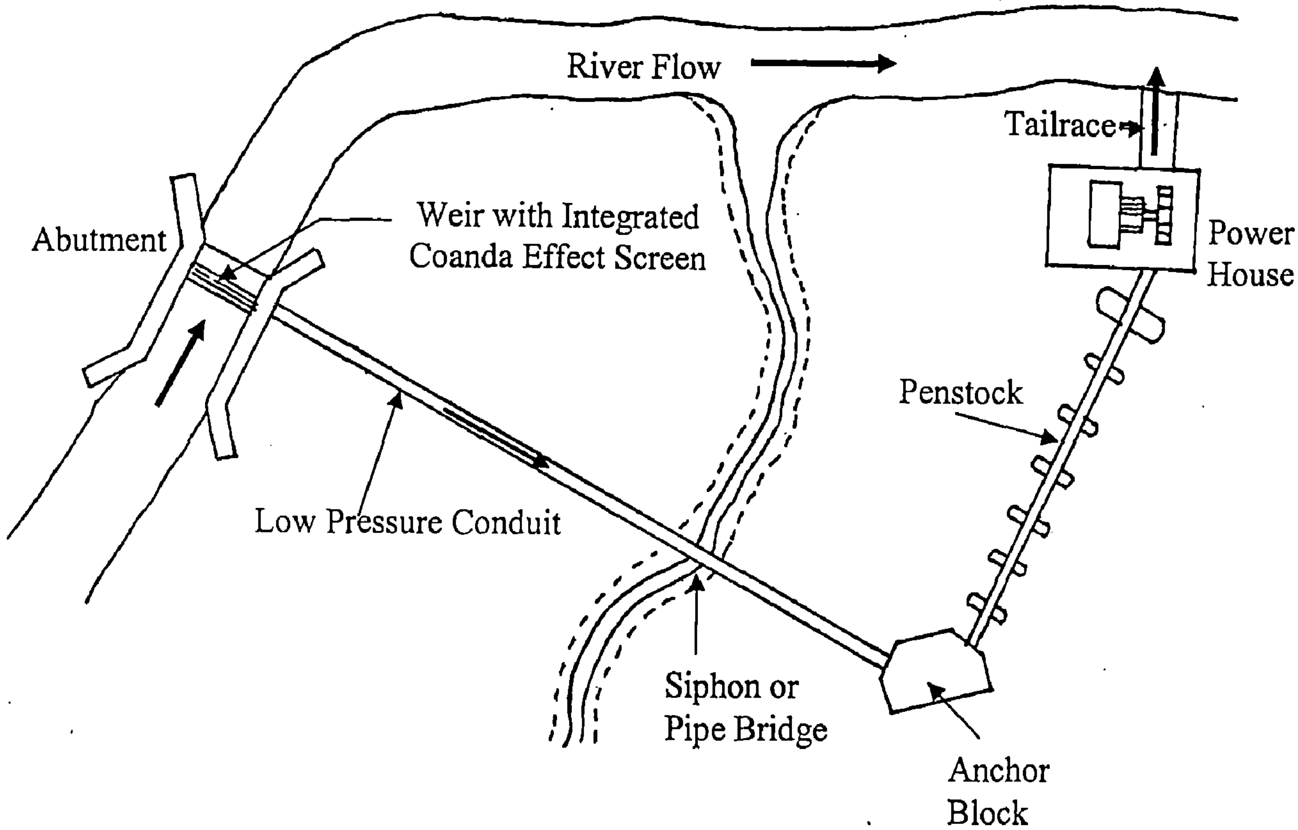


Fig. 3.23: SHP SYSTEM WITH COANDA EFFECT SCREEN INTAKE

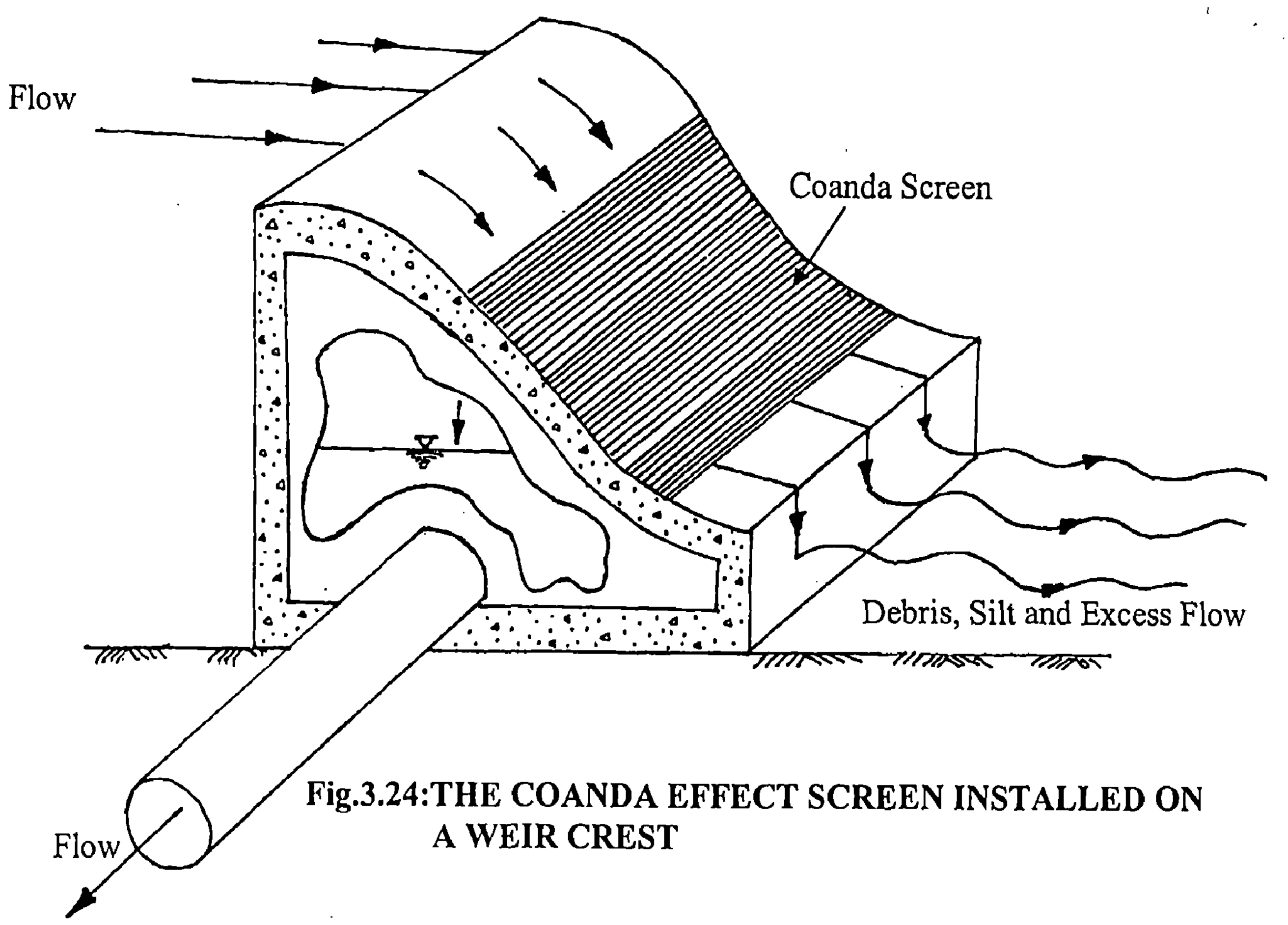
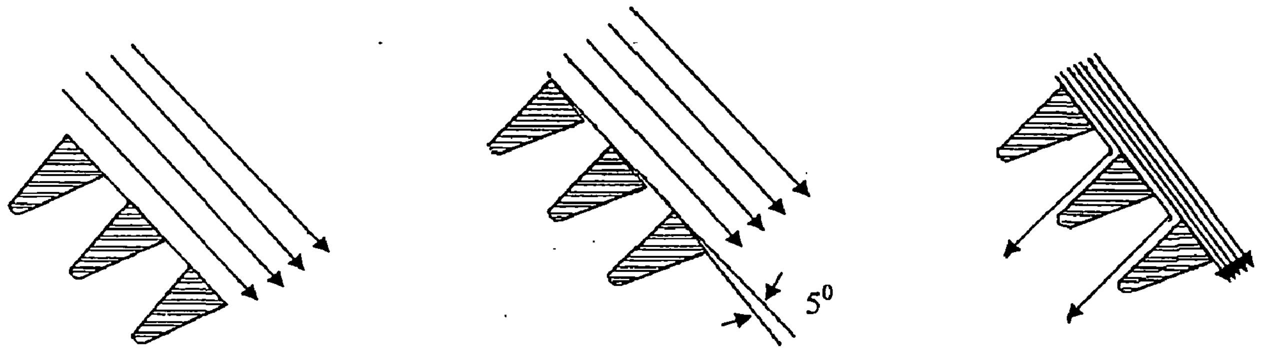


Fig.3.24:THE COANDA EFFECT SCREEN INSTALLED ON A WEIR CREST

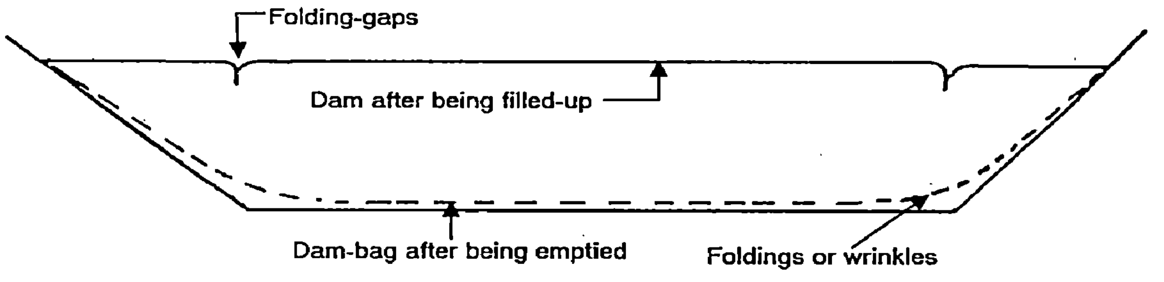


Flat - Wire Screen

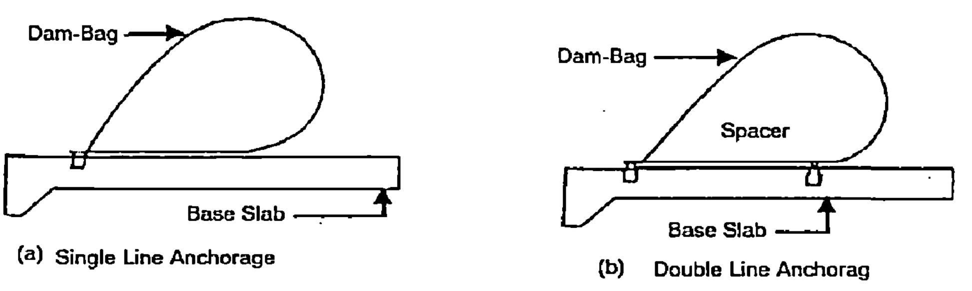
Tilted-Wire Screen
Without Coanda Effect

Tilted – Wire Screen
With Coanda Effect

Fig.3.25: FLOW OVER FLAT-WIRE AND TILTED-WIRE INCLINED STATIC SCREENS, WITH AND WITHOUT THE COANDA EFFECT



Conventional Canvas-Dam with Folding gaps



Anchorage Pattern

Fig. 3.26: INFLATED DAM WITH ANCHORAGE SYSTEM

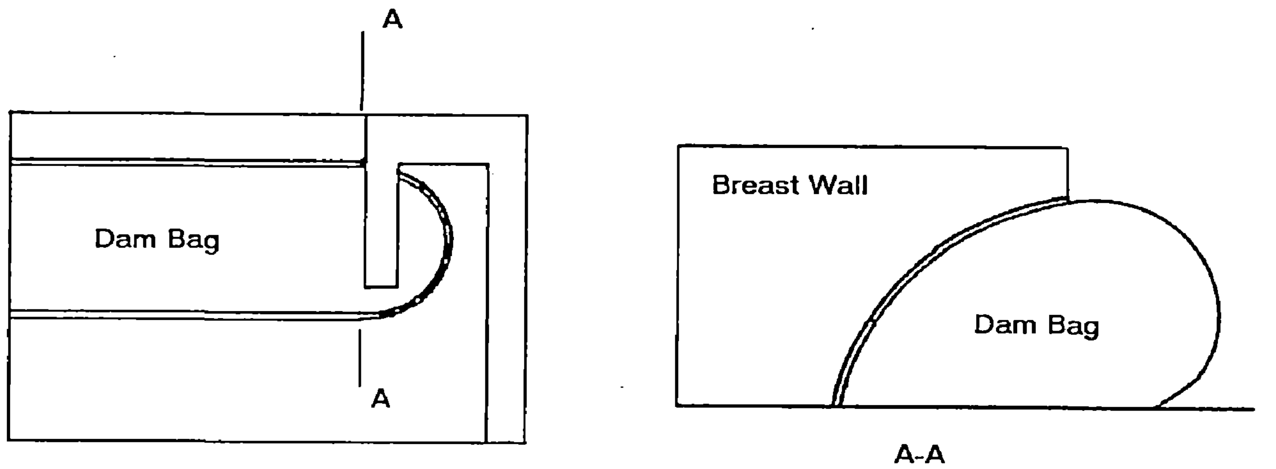


Fig.3.27: SHOWING LOCATION OF BREAST WALL

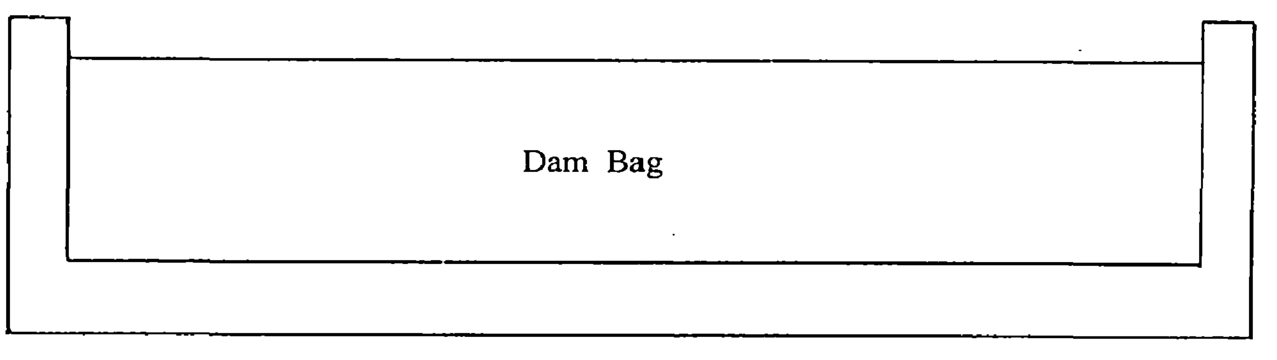
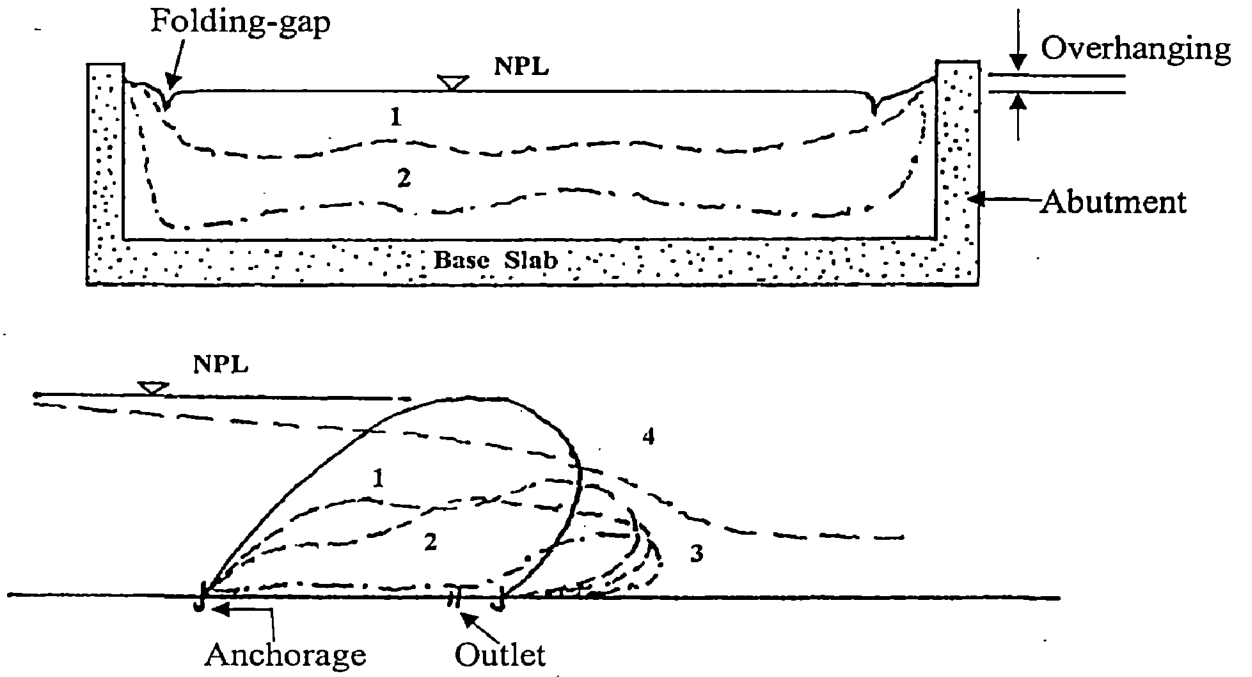
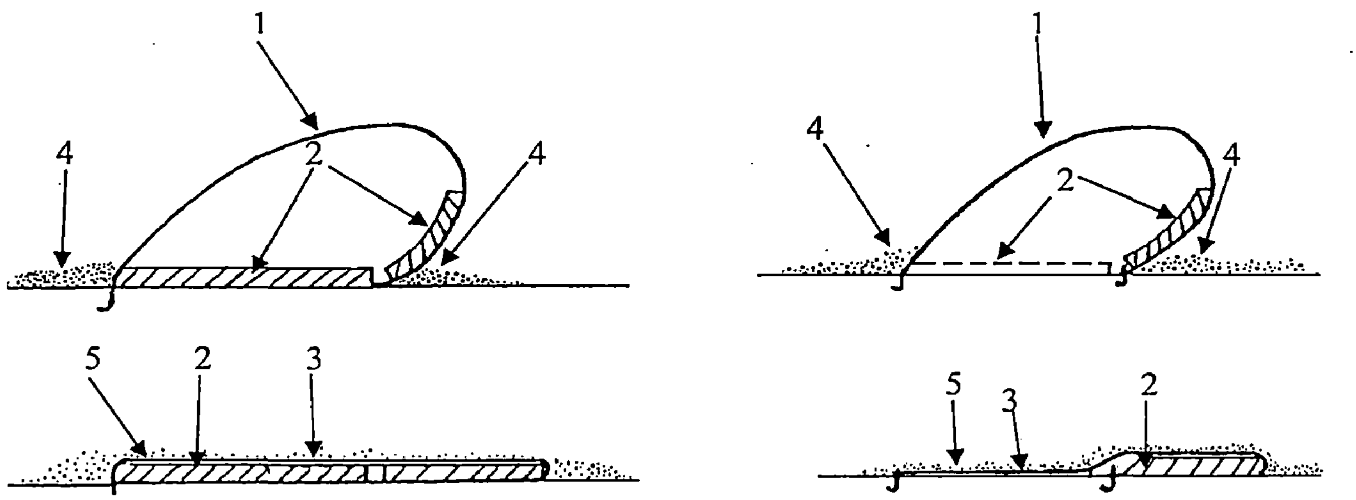


Fig.3.28: SHOWING A PILLOW TYPE DAM



1 and 2 - Successive collapsing and swinging;
3 - "Bag tail" with serious swinging;
4 - Water surface.

Fig..3.29: PROCESS OF COLLAPSING BAG



(a) Single Anchorage (b) Double Anchorage

1 – Inflated Bag; 2 – Sponge Layout; 3 – Collapsed Bag;

4 – Silt-deposit; 5 – Bed Load

Fig.3.30: SPONGE LAYERS INSIDE THE BAG

CHAPTER 4

DESILTING ARRANGEMENTS IN SMALL HYDROPOWER SCHEME

4.1 GENERAL

Streams in hilly areas, particularly in the Himalayas, carry a large amount of sediment both in suspension as well as bed load. The silt, sand and boulder exclusion devices employed at diversion structures for small hydropower plant intake are not sufficient to exclude all the suspended sediment that is transported in the upper layers of river/stream which is diverted into the intake channel. So further desilting devices are always necessary in the intake channel for the safety of operation and reduction in the maintenance of the turbine blades, other components of the plant and water conductor system. Small hydel plants may not justify costly desilting arrangements. So simpler devices are adopted for the purpose. In this chapter some of the measures, which are commonly adopted for desilting the intake water are discussed.

4.2 SEDIMENT CONTROL

Problem of sediment control involves two aspects. First is to prevent the entry of sediments into the primary water conductor system by suitable preventive measures. The second method or approach is to eliminate the sediment, once it enters the canal or tunnel and before it reaches the turbine by certain curative measures [9], [34][23].

4.2.1 Preventive Measures:

The preventive measures for silt free water in the stream can be made as follows.

1. **Measures in the catchments:**
   (a) Sediment control in the catchments.
   (b) Afforrestation in the higher reaches and on steep slopes.
   (c) Treatment of soil erosion by meadows.
   (d) Control and elimination of gullies.
   (e) Provision of check Dams at different strategic points.

2. **Measures at intake for large streams:**

   These involve the following measures.

   (a) Locating intakes in curved reaches.

   (b) Under sluices.

   (c) Pocket formed with divide wall.

   (d) Barrage regulation.

   (e) Excluder tunnels.

   (f) Degravelling tunnels below head regulator.

   (g) Deflector vanes in pocket

   (h) Bottom deflectors.

   (i) Floating deflectors.

   (j) High crest level and off take angle.

3. **Measures for small streams:**

   (a) Trench weir type diversion head- works / intakes.

   (b) Flushing system.

4.2.2 Curative Measures

As stated above a part of suspended sediment would enter into the power channel/tunnel even after several preventive measures are taken at the head works to prevent sediment entry into the canal. These sediments not only reduce the conveyance capacity of the channel but also damage turbine runners, valves and other components. As a result maintenance costs as well as loss in power generation increases after a few years of powerhouse operation.

The curative measures provided in the water conductor system include the following desilting arrangements which are techno-economically suitable to small hydel schemes.

(a) Settling basin/chamber.

(b) Vertex tube sediment extractors.

(c) Vertex settling basin.

(d) Use of coanda effective screen in the intake. *

* Use of Coanda effect screen at intake is already discussed in chapter 3

4.3 SETTLING BASIN OR CHAMBER

It is an arrangement in which the velocity of the flowing water is reduced by expansion of the channel cross-section and as a result the suspended sediment particles settle which are removed by flushing continuous or intermittent. It is one of the most effective device for removing sediment particles from the intake water. The reduction of velocity reduces bed shear stress and turbulence hence stops bed material from moving and causes most part of the suspended material to deposit. Once the minimum size of sediment to be excluded has been decided the design of settling basin would include determination of length of the basin and choice of the method of removal of the deposited sediment.

Desilting chamber adds to the cost of mini/micro hydro power stations but it is necessary where the water contains large quantities of course silt to minimize erosion damages to the turbine runner, etc. A guideline, indicating the relation between the head and size of silt particles to be removed from the water to achieve a power draft free of abrasion is given in the Table 4.1 for reference.

TABLE 4.1: Head And Size Of Silt Particle To Be Removed

SL. NO.	HEAD	SIZE OF SILT PARTICLES TO BE REMOVED
1	Small head < 15m	> 0.5mm
2	Medium head	0.2 to 0.5mm
3	High head > 300m	0.1 to 0.2mm

4.3.1 Principle of design and operation of Settling Basin

Principle of settling basin[32] is based on the assumption that the fall velocity of the suspended sediment is not effected by the turbulence of the flowing water. Let us consider 'U' the horizontal mean velocity of flow at any vertical section of a channel flow, 'ω' be the fall velocity of the sediment to be removed. So the time required for the particle on the water surface to settle to the bottom is

$$t = \frac{D}{\omega};$$

where, 'D' is the depth of water in the basin. The length required by the sediment particle to cover a horizontal distance 'L' with a velocity of flow 'U' will be

$$L = \frac{UD}{\omega} \tag{4.1}$$

Because of the turbulence in water flow, the fall velocity of the particles reduces and the length of the required basin is correspondingly increased. Since the quantitative information concerning the effect of turbulence on fall velocity is inadequate, an arbitrary increase (20%) in length worked out by equation 4.1 is recommended. However the silt particles at lower depth and the coarser materials will be deposited on the bottom in a shorter distance within the desilting basin. As the length is directly proportional to the horizontal velocity of the silt particles to be removed, the economy will be obtained by decreasing the horizontal particle velocity 'U'. For this purpose the cross–sectional area of the basin should be increased and that can be achieved by increasing the width or depth or both in the basin. The mean horizontal velocity in the settling basin that is considered desirable depends on the smallest size of the sediment to be removed and the economic length of the basin. The depth of flow in the basin is generally kept within 3m to 4m with velocity not higher than 0.4 to 0.6 m/sec; the smaller velocity should be used if finer material is to be removed.

4.3.2 Various Approaches Of Desilting Basin Design:

(a) Mosonyi's Method

Mosonyi[32] has adopted the fundamental approach for determining the length of the basin by finding the settling time of the particle through the depth after accounting for the effect of turbulence. The three basic relations used for determination of the required basin length are,

$$Q = bhU \tag{4.2}$$

$$t = h/\omega \tag{4.3}$$

$$L = Ut \tag{4.4}$$

where, Q = Discharge passing through the basin (cumec).

b = Width of the basin (m).

h = Depth of flow (m).

U = Flow through velocity (m/sec).

t = Settling time of the particlc(sec).

L = Length of the basin required(m).

ω = Fall velocity of a particle in stagnant water(m/sec).

Eliminating t from last two equations, following relations will be established such as

$$Q = bhU \text{ ; and}$$

$$L\omega = hU \tag{4.5}$$

Out of the unknowns the values of Q, U, ω are generally known or specified by the size of particles to be removed and not to be picked up again if once settled. For deciding the parameter U, Mosonyi [32] recommends adoption of the critical flow through velocity which will not entertain the particles once settled at the bottom of the basin. He has recommended use of critical velocity relation given by Camp.

$$U = a\sqrt{d} \text{ cm/sec.} \tag{4.6}$$

where, d is the diameter of the particle in mm with the following values of constants to be adopted for different particle sizes.

a = 36 for d >1mm.

a = 44 for 0.1mm< d <.1 mm.

a = 51 for d<0.1mm

However, if the velocity of flow in the desilting chamber, as computed from above is very low, hydraulic short-circuiting may occur. This phenomenon has been observed by Davis and Mosonyi and so the flow through velocity as already mentioned above shall be adopted.

The value of settling velocity in stagnant water can be determined from L.Sudry's curve for different sediment concentrations [Fig. 4.1]. The chart provides information as to the settling of coarse quartzite particles, but cannot be precisely used for determining the settling characteristics of very fine fractions (fine sand and smaller particles). For fine sediment it is advisable to carry out settling tests or to use other standard curves (Hunter Rouse). To take the turbulence into consideration the corrected settling velocity can be obtained as

$$\omega_c = \omega - \omega' \tag{4.7}$$

where, ω_c = corrected / effective settling velocity.

ω = Settling velocity of particle in still water.

$\omega' = \alpha U$; by L. Levin [32] (m/Sec) = reduction in settling velocity

$$\alpha = \frac{0.132}{\sqrt{h}}$$

h = the depth of water in m.

Therefore, making substitutions for the fall velocity

$$L = \frac{hU}{\omega_c} = \frac{hU}{\omega - \omega'} = \frac{h^{3/2}U}{h^{1/2}\omega - 0.132U}$$

The negative denominator indicates the fact that no settling can take place under the assumed condition and other parameters will require revision.

In addition to the above three values one has to assume one of the main dimensions of the basin by taking the fact that long and/or wide basin can in general be constructed at lower costs than deep one, the minimum practical depth recommended is between 4.0m to 8.0m. The other two remaining dimensions of the basin can be computed by above equations.

Again from equation (4.3) and (4.4) we have the condition that the water mass conveyed during settling time should be equal to the capacity of the settling basin. i.e. $Q_t = hbL = V\ (m^3)$, where Q_t is the total discharge within a specified time. The correct determination of the setting velocity constitutes one of the most important problems to be solved in designing settling basins. The experimental approach to this problem with the given type of sediment is deemed most expedient. The computation methods may be adopted to work out preliminary design to be tested on hydraulic models before finalisation.

(b) Velikanov's Design Function Approach

M.A.Valiknov [32] investigated the complicacy of settling velocity and the settling length for turbulent flow and derived the following equation purely based on the theory of probabilities. The equation is as follows:

$$L=\lambda^2U^2(\sqrt{h}-0.2)^2/(7.51\ \omega^2) \qquad (4.8)$$

where, L = required settling length.

U = velocity of flow in basin

h = depth of water.

ω = Fall velocity of silt particles.

λ depends on the removal ratio and defined by a function $\lambda = f(w)$ and can be determined from the curve given in the Fig. 4.2. Here w denotes the ratio of settled sediment to the total load entering with the flow mathematically,

$$w = 100\text{-}(1-C_0/C) \qquad (4.9)$$

Where C_0 is the permissible concentration of sediment in water at exit of basin and C is the concentration of sediment in incoming water into the basin.

The following considerations should be taken into account in applying Veliknov's formula to obtain satisfactory results.

(i) In the positive range of coefficient of 'λ', pertaining to w values of 90% to 98% removal of limit particle size should preferably be applied.

(ii) It should be noted that w is related only to the fraction to be settled out and therefore can't be used for the total sediment load unless the limiting particle size is the smallest particle in the load or also the sediment is composed of uniform size.

(c) Camp T.R and Ensign Dobbins's Approach

An analysis of flow in settling basin has been presented by T. R. Camp & Ensign Dobbins's [3], [17]. Dobbin derived an analytical solution for design of settling basin by assuming the following facts of the fluid flow.

(i) The suspended particles in the flowing water settle without any diffusion along the longitudinal direction of flow.

(ii) The velocity of the flow is constant throughout the cross-section of water flow.

(iii) Diffusion coefficient is constant through out the cross-section of water flow.

Camp expressed Dobbins's analytical solution in graphical form by using the parameters q_{se}/q_{si}, ω/U_*, $\omega L/UD$

where, q_{se} = Amount of suspended sediment of a given size leaving the basin.

q_{si} = Amount of suspended sediment entering the basin.

U = Mean velocity of flow.

U_* = Shear velocity.

D = Depth of basin.

L = Length of basin.

Using Manning's equation the parameter ω/U_* can be written as

$$\frac{\omega}{U_*} = \frac{\omega D^{1/6}}{nU\sqrt{g}} \tag{4.10}$$

where n is the Manning's roughosity coefficient. Fig. 4.3 and Fig. 4.4 shows the graphical relationship of the $\frac{\omega D^{1/6}}{nU\sqrt{g}}$ and $\frac{\omega L}{UD}$ parameters with removal efficiency of the basin.

(d) Cean et-al. Approach:

Cean et-al [3] [17] established a mathematical model for the settling of suspended particles in turbulent flows. They obtained a differential equation for suspension concentration in a settling basin from the law of conservation of matter and assuming concentration distributions to be similar along the direction of mean flow. The assumed distribution is

$$C \quad (x,y) \quad = \quad C_a \quad \exp \quad \left[\frac{\omega}{\epsilon_y}(y-a)\right]\left[\exp\left\{\frac{U}{2\epsilon}-\frac{U}{2\epsilon}\sqrt{1+\frac{4\omega^2}{U^2}\frac{\epsilon_x}{\epsilon_y}}\right\}\right]$$

(4.11)

where, C = concentration of sediment at a point.

C_a = C (0,a) i.e. sediment concentration at a distance 'a' above the bed.

ϵ = Sediment transfer coefficient

ϵ_y = Cross-sectional mean of vertical diffusion coefficient.

Here x-axis is chosen in the direction of flow and y-axis is vertically upwards. The solution of the above equation is used to determine the similarity criteria for settling basins. The distribution of concentration is found to depend on three non-dimensional variables $\frac{\omega D}{\epsilon_y}$, $\frac{U_*}{\epsilon_x}$ and $\frac{\omega^2}{U^2}\frac{\epsilon_x}{\epsilon_y}$. It was found that the models of settling basins must have similarity for both the flow velocity and settling velocity of suspended particles.

Sumer [18] transfers Cean et-al solution into a series of graphs as shown in Fig. 4.5 and can be used in predicting the removal ratio of fine sediments in the settling basin.

(e) Richardson et-al, Approach:

Richardson et-al [10] have given the methodology for design of desilting basin. He derived the equation of continuity of flow coupled with the resistance equation developed for a plane bed with little or no sediment transport as

$$\frac{U}{U_*} = 5.9\log_{10}\left(\frac{D}{d_{85}} + 5.44\right) \tag{4.12}$$

where $U_* = \sqrt{\frac{\tau_c}{\rho}}$ = Critical shear stress velocity.

τ_c = Critical shear stress.

ρ = Density of suspended particles.

U = Non-scouring flow through velocity inside the desilting basin.

D = depth of flow.

Equation (4.12) is solved for U, assuming D. The correction for the effect of concentration is applied to the settling velocity by making use of curves proposed by Camp, Mcnown and Lin as shown in Fig. 4.6. After obtaining the value of ω , D and U the length of the desilting basin can be evaluated as, $L = \frac{UD}{\omega}$

(f) U.S.B.R Approach:

The United State Bureau of Reclamation [3] has developed a basic relation of removal efficiency for the design of desilting basin i.e.

$$\eta = 1 - \frac{q_{se}}{q_{si}} = 1 - \exp\left(-\frac{\omega L}{UD}\right) \tag{4.13}$$

where, η = Basin efficiency.

q_{se} = Amount of sediment leaving the basin per unit time.

q_{si} = Amount of sediment entering the basin per unit time.

ω = Fall velocity of sediment.

L = Length of basin.

D = Depth of water in basin.

This equation is a particular form of a functional relationship as given by Camp and Dobbins.

(g) **Sumer's Approach:**

M.S.Sumer [40] analysed the settling of sediment particles in an open channel assuming a logarithmic velocity distribution and developed the following relation

$$\frac{\epsilon_y}{UD} = \frac{6y}{D}\left(1 - \frac{y}{D}\right) \tag{4.14}$$

where, ϵ_y = Diffusion coefficient.

y = The elevation above the bed.

U and D are same as defined earlier.

The suspended particles in the flow of water inside the channel are under the influence of turbulence and hence they will trace a random path. As the particles in suspension would be a statistical quantities, the properties of the settling length of particle can be predicted by the approach developed by Syre [3].

Syre pointed out that since the heaviest deposition should occur close to the source and the amount of particles deposited should decrease with distance, the distribution of deposited particles should follow an exponential function with increasing t,

i.e

$$f(t) = \frac{\lambda}{\mu}\exp\left(-\frac{\lambda}{\mu}x\right) \tag{4.15}$$

$$\mu = \frac{U\rho}{\epsilon_y}$$

where, t = the mean retention time

x = Distance in flow direction.

λ = Mean rate at which particles settle.

μ = Non-dimensional mean flow velocity.

The design of settling length corresponding to certain efficiency 'η' has been worked out

as

$$L = \frac{-6\left(\frac{U}{U_*}\right)D}{K\lambda} - \log(1-\eta) \tag{4.16}$$

$$U_* = \sqrt{gDS}$$

Where k is the Von-Kaman's constant taken as 0.4. Here first of all certain value of efficiency that desired is assumed then the length of basin can be find out from the above equation by assuming the value of 'D' and calculating U, U_*, λ and k. The value of λ can be obtained by referring the Fig. 4.7 of λ Vs β as given by Syre's numerical solution. Where β is a non-dimensional settling velocity parameter whose value is equal to $\frac{\omega}{KU_*}$.

(h) Grade et- al. Approach

Laboratory experiments have shown that the observed values of removal efficiency considerably vary from those given by Camp, USBR and Sumer. They analysed the data and expressed efficiency of the desilting basin, η (in %) by the following relation,

$$\eta = \eta_0\left(1 - e^{-\frac{CL}{D}}\right) \tag{4.17}$$

Where, η_0 and C are functions of $\frac{\omega}{U_*}$, U_* is the shear velocity in the desilting basin and ω the fall velocity of sediment particle in clear water. The values of η_0 and C for different values of ω/U_* are given in Table 4.2.

TABLE 4.2: Values Of Different Parameters In Grade et- al. Approach

ω/U_*	0.70	0.90	1.20	1.60	2.0	>2.2
C	0.02	0.03	0.06	0.14	0.215	0.24
η_0	34	40	50	70	97	100

(i) Ranga Raju et. Al, Approach

Ranga Raju. et al [39], found that the following equation yields a better result than the equation proposed by Garde et al. and recommended it for use, when $\frac{\omega}{U_*} < 2.5$

$$\eta = 11.7\left(\frac{\omega}{U}\right)^{0.81}\left(\frac{LB}{B_c D_c}\right)^{0.23}\left(\frac{D^{1/6}}{n\sqrt{g}}\right)^{0.98} \tag{4.18}$$

where,
- ω = Fall velocity of the particle.
- U_* = Shear velocity of flow.
- U = Mean velocity of flow.
- D_C = Depth of flow in the approach channel
- Bc = Width of approach channel.

B = Width of basin.

D = Depth of basin.

Ranga Raju et. Al, found that for coarse sands the equation proposed by him and by Garde give identical results. For finer sediments where $U/U_* < 0.4$ his method gives satisfactory results.

A typical general layout of desilting chamber is shown in Fig. 4.8. As per CBIP [11] recommendation various dimensions of desilting chamber for discharge upto 12.5 Cumecs are given in Table 4.3.

TABLE 4.3: Dimensions of Desilting Chamber for Different Discharges

SL.NO.	DISCHARGE (CUMECS)	LENGTH (M)	WIDTH (M)	DEPTH (M)
1.	UPTO 1	10	3.5	2.5
2.	1 TO 5	30	3.5	3.0
3.	5 TO 7.5	40	3.5	3.5
4.	7.5 TO 12.5	50	4.5	4.0

4.3.3 Removal Of Sediments From Sedimentation Basin

There are two major ways to approach removal of sediments from sedimentation or settling basins in the form of sediment flushing conduit. Deposited sediment may be removed from the basin while the basin is operational or while the basin is taken out of operation. Mechanical means or different kind of flushing systems may be used to remove settled particles.

4.3.3.1 Removal While The Basin Is Out Of Operation

In this approach of sediment removal, the water flow through the basin is cut-off throughout the removal period. It will therefore cause loss in power generation. Deposited sediments may be removed manually or by mechanical equipments after the basin is dewatered. Sediments may also be removed by lowering down the water level inside the basin and generating a swift flowing free surface gravity flow throughout the basin.

4.3.3.2 Removal While The Basin Is Operational

Here the water level and water flow must be maintained in the basin throughout the flushing period in order to facilitate continuous power generation. Flushing is independent on power generation and may therefore be carried out as often as required. Removal of sediments while the basin is operational may be achieved by use of continuous flushing or intermittent flushing or by use of some kind of suction devices. Hence it is necessary to generate a current close to the settled particles by flushing without mixing them with the main water flow in the basin.

Out of the two systems the continuous flushing type operation is most often used as it allows uninterrupted power generation.

4.3.4 Recommended Design Guidelines For Flushing System.

Flushing of sedimentation basins at small hydropower projects in river with large sediment load should meet the following requirements.

(i) Removal of sediments shall normally be carried out while settling basins are operational.

(ii) The system and its operation shall be straight forward and do not require any physical intervention inside the basin, bearing in mind that it is not possible to see any thing inside silty water.

(iii) The system shall be operated without use of any machinery, which requires power input.

(iv) Flushing water consumption shall be as low as possible. The system shall be flexible with respect to flushing capacity. High sediment loads may be handled by increasing flushing frequency and thus ensure continuous water supply to the power plant also during periods with extreme sediment loads.

As per the above design guidelines the following arrangements can be adopted for continuous operation [32], [44].

(a) Use of several basins.

(b) A continuous flushing of settled sediment by flushing canals.

(c) Serpent sediment sluicing system.

4.3.4.1 Use Of Several Basins

When a condition of sediment removal by stopping the water flow through the basin and at the same time continuous power generation is required then these can be achieved by placing more than one parallel desilting basins, operating intermittently [Fig. 4.9].In this arrangement some of basins can be flushed mechanically or using flushing canals while others are in operation.

4.3.4.2 Automatic Flushing Settling Basin

In this type of system the flushing of settled sediments is continuous while the basin is operational. An inflow of 20 to 25% excess of water demand should be admitted into the basin for continuous flushing of the sediment at the channel bottom or flushing sluice.

The principal structural features of settling basins are shown in Fig.4.10. Care should be taken to convey flow into the basin through a transition section of suitable length and proper design, following the inlet sluice. A dufour type settling basin can also be used [Fig.4.11], which prevents the settled sand to be picked up again by the whirls of the turbulent flow. The floating sand trap grate [Fig.4.11] developed by 'Biichi' [32] is quite beneficial to achieve the above objective.

4.3.4.3 Serpent Sediment Sluicing System

This is basically an intermittent type flushing arrangement without any interruption to the functioning of the powerhouse. This new approach requires less quantity of water for flushing operation than a continuous type flushing system. It is invented by research Engineer Haakon stole at Sintef, Norwegian Hydro Technical Laboratory in 1988 [44]. The system works by gravity forces, and it does not require any machinery or energy input. The system is operated by use of one gate and one valve only. The sedimentation basin might be pressurised as shown in Fig. 4.12, or it might have a free surface flow as shown in Fig. 4.13. The bottom of the basin might have one or more flushing channel. The flushing channel is connected to the sedimentation basin by a longitudinal slot. A float unit popularly called the serpent covers the slot as shown in Fig. 4.13. The flushing channel is connected to a flushing pipe in the down stream end. A hydrostatic pressure difference between the sedimentation basin and the outlet of the flushing pipe provides the necessary energy potential for flushing.

The float unit is furnished with an operation valve. By operating the valve, the float unit might be de-watered and thus gradually buoyant or filled with water, which causes it to gradually sink down and cover the slot in the bottom of the basin. Serpent sediment sluicing system might be operated in either "opening mode" or “closing mode" and thus facilitate flushing of sedimentation basin.

(i) Opening Mode

Normally the floating unit is filled with water and thus blocking the slot between the sedimentation basin and the flushing channel. Sediments will be deposited in the sedimentation basin on top of the float unit as shown in Fig. 4.13. The flushing gate in the flushing pipe is opened and causes a pressure difference between the basin and the flushing channel (i.e. a pressure difference over the float unit). By use of the operation valve the float unit is de-watered at a controlled speed and it becomes gradually buoyant. The slot between the sedimentation basin and the flushing channel is opened in the upstream end. Water and sediments is sucked into the flushing channel in the suction area around the opening and it causes erosion and slides in the deposited sediments. The suction area will slowly shift downstream and remove deposited sediments throughout the entire basin as illustrated in Fig. 4.14.

(ii) Closing Mode

If the sediments deposit in the bottom of the basin while the float unit is buoyant [Fig. 4.15], the sediments might be removed by operating in “closing mode”. Here the flushing gate is opened, and deposits located closed to the outlet of the flushing channel is sucked into the flushing pipe. Sediment located further inside the basin, however, will not be removed because the erosive force in this area is too low. Water then gradually filled into the float unit. The downstream end of the float unit first sinks down and closes the slot gradually. The suction area will correspondingly be moved further upstream in the basin and it causes erosion and slides in the deposits as indicated in Fig. 4.15. When deposits along the entire length of the sedimentation basin is removed, the float unit will close the slot between the basin and the flushing channel and thus prevent spilling of flushing water through the flushing channel after the sediment removal process is completed.

The difference in water level between sedimentation basin and the flushing outlet level is the available energy head for flushing. This head may be split into the suction head and transport head. Suction head depend on particle size and other features of the sediments. Model studies have shown that a mix of sand and fine gravel requires a suction head of about 0.5m. The remaining head is utilised for transport purposes. The hydraulic transport capacity of flushing pipe should be properly designed, which has a direct effect on functioning of serpent sediment sluicing system. During flood both tail water and water level at the intake will rise hence the driving force of the sediment flushing system is maintained and operate efficiently.

The speed at which the serpent opens the slot controls the concentration of sediments in the flushing water. The flushing capacity is not dependent on the thickness of the sediment layer. It only effects the duration of flushing period. A thicker layer requires a slower speed of the serpent through the basin in order to maintain constant concentration of sediments in the flushing water. The upstream end of the flushing channel will always be located above any possible sediment level and assure that the serpent may start to remove sediments.

This sediment sluicing system is applied in different small hydro projects like Beiarely intake at Storglomfjord in Norway, Jhimruk and Kali Gandaki 'A' hydropower project in Nepal .The system has been subjected to international patent investigations and it is protected internationally. The patent right is hold by SINTEF NHL (Norwegian Hydro Technical Laboratory) and the inventor Haakon Stole. Norwegian state power board holds extensive license rights.

4.4 VORTEX-TUBE SEDIMENT EXTRACTOR

The vortex tube is a device for extracting sediment from canals that consists of a tube laid horizontally across the canal bed with an open slit along its top edge. The flow near the canal bed is collected in the vortex tube and is extracted from the canal to a near by escape channel. The idea of a vortex tube for a sediment extractor was first introduced by Crump (Blench 1952)[4]. Early research into the device largely consisted of laboratory scale model studies that helped to explain the behaviour and action of the tube which enabled the development of some design criteria [Parshall (1975); Blench (1952); Ahmed (1958); Robinson (1962)]Mahmood (1975) presented an analytical model for the flow through vortex tubes, which he calibrated using Robinson's data; without any design recommendations. H.R Wallingford, Wallingford, England, is the first institute that has undertaken the research work to produce guidelines for the hydraulic design of vortex tubes (Sanmuganathan 1976; Sanmuganathan and Lawrence 1980; Lawrence and Sanmuganathan 1981; Singh 1983; Atkinson 1987; Atkinson 1990; Russel 1991). Atkinson, E (1994) has presented theory for predicting the trapping efficiency of a vortex tube sediment extractor, compared the predictions of the theory with field data, and discussed its use in the design process.

4.4.1 Proper Location And Functioning Of Vortex-Tube

Vortex tubes are usually located in the head reach of a power canal, where additional water can be diverted to operate the extractor, and where head is available so that the extracted water and sediment can be conveyed back to the river. It is an inexpensive method as compared to other sediment control devices. The majority of bed load can be removed from the canal at the expense of about 10% to 20% of the canal discharge. The open-top vertex tube is placed across the bed of a canal at an angle of 45° to 90° to the axis of flow. The downstream end discharge of the tube that is extracted from canal flow is regulated with the aid of a valve. The edge, or lip, of the tube is kept in level with the bed slope of the canal. Flow of water over the opening of the tube generates vortex motion in the tube along its full length. This spiral flow has sufficient velocity to transport the sediment along the tube into the escape channel. The most efficient width of slot is dependent on particle size of bed-load or suspended load near to bed besides other physical and hydraulic characteristics and need to be fixed by a model study after the design criteria given by Edmund Atkinson (1994). Atkinson design criteria of vertex tube sediment extractor is based on trapping- efficiency theory. Here sediment concentrations, upstream of the vortex tube are assumed to be in equilibrium by the effect of the turbulence and the force of gravity. At the head of the canal more turbulence is generated and produces a uniform sediment distribution than an equilibrium sediment concentration profile. So the vertex tube should not be located too close to the canal head, which in turn reduces the trapping efficiency of the device.

4.4.2 Theory Of Sediment Extraction

Sediment extraction from canal flow by vortex tube device is based on the trapping-efficiency theory. Trapping efficiency is defined as the ratio of sediment load extracted by the device to that of sediment load carried in the canal flow. In this theory a sediment flux profile in the flow of canal water is predicted in the channel upstream of a vortex tube. Sediment flux profile is defined as $U_z C_z \rho_w$.

where, U_z = velocity of flow,

C_z = sediment concentration at height z,

ρ_w = density of water.

A typical sediment flux profile is illustrated in the Fig. 4.16. The sediment concentrations upstream from the vortex tube are assumed to be in equilibrium as a result of gravity and turbulence force acted upon the sediment, which maintains sediments in suspension. Since the sediment concentrations depend upon the sediment size, the calculation of trapping efficiency is repeated for each size fraction of the material being transported in the canal.

The following algorithmic velocity profile is assumed in the channel i.e.

$$U_z = \overline{U} + \frac{U_*}{k}[\ln\left(\frac{Z}{h}\right) + 1] \qquad (4.19)$$

where, $\overline{U}$ = mean velocity.

U_* = Shear velocity.

k = Von Karman's constant

h = Depth of flow.

The sediment concentration upstream from the vortex tube is obtained as per the following equation.

$$C_{zj} + \epsilon_s \frac{dC_{zj}}{dZ} = 0 \qquad (4.20)$$

Solution to the above equation with boundary condition $C_{zj} = C_{bj}$ at referece height Z= bj

is
$$\frac{C_{zj}}{C_{bj}} = \exp\left[-15\frac{\omega_j}{U_* h}(Z - b_j)\right] \tag{4.21}$$

The reference height for each size fraction b_j is taken as the greater of twice the grain size and height from the bed at which $U_z = U_*'$ and $C_{zj} = C_{bj}$

Where suffix j = individual sediment size fraction.

ω_j = settling velocity.

$\in_s$ = sediment- diffusion coefficient constant for all sediment sizes.

$= \frac{hU_*}{15}$, [After Lane and Kalinske (1942)]

U_*' = Shear velocity related to bed material gain roughness.

The steps involved calculating the trapping efficiency is as follows.

1. The size distribution of the material in transport is calculated from the bed material size grading. The method is based on the treatment of graded sediments described by Bettess(1982). Here the sediment is split into size fractions, and a transport rate trj for each fraction j is calculated by assuming that the whole of the bed consists of material of that size. The transport rate for each fraction is then multiplied by the proportion of the bed material. Summation of the individual transport rates yields the total transport rate for all fraction ttr. The proportions within the transported material P_{tj} for each fractions are therefore derived as

   $$P_{tj} = \frac{trj}{ttr}$$

   Then the setting velocity ω_j for each size fraction are calculated using Gibbs etal (1971) relationship.

2. The next step involves the calculation of U_* and U_*' by using method of Engelund (1966). Then b_j is determined for each size fraction. Where b_j is taken as the greater of twice the grain size or the height at which $U = U_*$ and within the bed layer $U_z = U_*'$ and $C_{zj} = C_{bj}$.

3. Determine the height of dividing streamline a_i from the following equation.

$$R_i = \frac{a_i}{h} + \frac{U_* a_i}{\overline{U}kh} \ln\left(\frac{a_i}{h}\right) \tag{4.22}$$

where, R_i = extraction ratio (initially assumed an over all extraction ratio) U^*, $\overline{U}$, k, h are defined earlier. The equation above can be solved iteratively.

4. Next step is the calculation of trapping efficiency TE_{ji} for each size fraction and tube section using the equation

$$TE_{ji} = \frac{b_j \frac{U_*'}{U_*} + \int_{b_j}^{a_i} \left[\frac{\overline{U}}{U_*} + \frac{1}{k}\left\{ \ln\left(\frac{z}{h}\right) + 1 \right\} \right] \exp\left[-15 \frac{\omega_j}{U_* h} (Z - b_j) \right] dZ}{b_j \frac{U_*'}{U_*} + \int_{b}^{h_j} \left[\frac{\overline{U}}{U_*} + \frac{1}{k}\left\{ \ln\left(\frac{z}{h}\right) + 1 \right\} \right] \exp\left[-15 \frac{\omega_j}{U_* h} (Z - b_j) \right] dZ} \tag{4.23}$$

Then obtain 'TE' the trapping efficiency for all size fractions from equation

$$TE = \frac{1}{N} \sum_{j=1}^{N} TE_j \tag{4.24}$$

$$\text{Where, } TE_j = \frac{1}{M} \sum_{i=1}^{M} TE_{ji} \tag{4.25}$$

M = Number of tube section into which the tube is divided (usually10)

N = Number of sediment size fractions

4.4.3 Design Of Vortex Tube Sediment Extractor

The first step in the design process would be an assessment of the potential trapping efficiency. The method as described in the previous para, suggested by Atkinson [4] should be used to predict trapping efficiency for a range of extraction ratio (uniform extraction should be assumed). If the required trapping efficiency can be achieved with an acceptable extracted discharge Q_{ex}, then the procedure set out in the following should be used to determine tube dimensions from Q_{ex}.

Before designing the different parameter of a vortex tube the following definition should be kept in mind [Fig.4.17].

V_n = Normal component of the flow velocity near bed level entering the tube.

V_t = Tangential component of the flow velocity near bed level entering the tube.

V_b = Mean velocity in the near-bed region of the flow upstream of the vortex tube

θ = Angle between the vortex-tube axis and the channel flow direction.

h = Depth of flow in the canal.

g = Gravitational acceleration.

H = Static pressure head in the vertex tube; assumed constant across the tube section.

x = The distance along the tube axis from its closed end.

t = Silt width of the vertex tube.

U = Average axial velocity in the tube.

A = Tube cross-sectional area.

l = length of the tube.

d = Tube diameter.

a = Height of the dividing streamline between the flow entering the tube and the flow passing over it.

Initially a design with one tube should be tried with the following dimensions.

$\frac{t}{d} \approx 0.2$; $\theta = 90^0$; $\frac{l}{d} \approx 8$; Set $Q_T = Q_{ex}$ and 'l' is the canal bed width.

Step 1. From the equation below determine the value of $\bar{a}$, the height of dividing streamline, where the discharge below the dividing streamline is equal to the extracted discharge for the conditions when the abstraction is uniform across the channel.

$$\overline{U}\bar{a} + \frac{U_*}{k}\bar{a}\ln\left(\frac{\bar{a}}{h}\right) = \frac{Q_T}{l\sin\theta} - \frac{bU_*}{k} \tag{4.26}$$

The equation above is solved by an iterative method.

Where, $\overline{U}$ = Mean velocity in the canal.

U_* = Shear velocity.

B = Height at the top of the bed layer.

Q_T = Discharge extracted.

k =Von Karman's Constant.

Step 2. Evaluate C_d, the silt coefficient, V_b, mean velocity in near bed region of channel upstream of vortex tube and k, the von karman's constant as per the following equation.

$$C_d = 0.47\sqrt{1 - e^{-\frac{\bar{a}}{0.75t}}} + 0.0016\frac{\bar{a}}{t} - 1 \qquad (4.27)$$

$$V_b = \frac{Q_T}{l\bar{a}} \qquad (4.28)$$

$$k = \frac{C_d t}{0.98}\left[\frac{\sqrt{1-\left(\frac{t}{d}\right)^2}}{a} + \frac{1}{d}\right] \qquad (4.29)$$

Step 3. Calculate $\beta = \sqrt{\frac{2}{1+k^2}\left(\frac{C_d tl}{A}\right)}$ (4.30)

Step 4. Compute the static head loss ΔH by using the equation

$$\Delta H = H_T\text{-}H_o\text{-}\frac{(V_b \sin\theta)^2}{2g} \qquad (4.31)$$

and $$Q_T = A\left[\tanh\beta\sqrt{g(H_T - H_0)} + \frac{V_b \text{Cos}\theta}{2}\left(1 - \frac{1}{\cosh\beta}\right)\right] \qquad (4.32)$$

As the initial assumption for $\theta = 90^\circ$,

$$\Delta H = \left[\frac{Q_T}{A\tanh\beta}\right]^2 \frac{1}{g} - \frac{V_b}{2g}$$

Step 5. Calculate the abstraction rate q_o at the closed end of vortex tube from the following formula and substituting the value of Q_T from equation 4.32 .

$$q_x = \frac{A\beta}{l\cosh\beta}\left[\sqrt{g(H_T - H_0)}\cosh\left(\frac{\beta x}{l}\right) + \frac{V_b \cos\theta}{2}\sinh\left(\beta - \frac{\beta x}{l}\right)\right] \qquad (4.33)$$

With $\theta = 90^\circ$, $q_o = \frac{Q_T\beta}{l\sinh\beta}$

Step 6. Repeat step 1 and 2 for conditions at the closed end of the tube (i.e. q_o replaces $\frac{Q_T}{l}$ and a_o replaces $\bar{a}$

Step 7. Calculate the tangential velocity at the closed end of the tube 'V_{to}' from the equation $Vt = \frac{0.98 q_x k_x}{t C_d x}$, using the values obtained in step 6.

Step 8. Calculate R_{oc}, by the equation

$$R_{oc} = \frac{2\cosh\beta\sqrt{g(H_T - H_0)}}{2\sqrt{g(H_T - H_0)} + V_b \cos\theta \sinh\beta} \tag{4.34}$$

Where, $R_{oc} = \frac{\text{water extraction rate at open end}}{\text{extraction rate at closed end}}$

As $\theta = 90_o$, $R_{oc} = \cosh\beta$

After obtaining the dimensions the predictions should be used in the following ways.

1. If ΔH is unacceptably large, then design with a large tube diameter should be attempted; $\frac{t}{d}$ could also be increased slightly.

2. If Roc > 2, try either a large tube or a design with more tubes; t/d could also be reduced slightly.

3. If predicted Vto does not exceed the settling velocity for the D_{90} canal bed material size, then the tube may partially blocked. This can be overcome by these measures

   - If Roc is higher than 1.5, the measures set out in '2' should be attempted.
   - If Roc is low, smaller tube diameters should be used; if head loss is also a constraint, a slightly higher $\frac{t}{d}$ ratio is required.
   - If neither of the preceding measures improve V_{to} sufficiently, the channel velocities at the vortex tube must be increased by sitting the vertex tube in a short flumed reach.

Once the vortex-tube dimensions have been set, then the trapping efficiency is recalculated as per previous procedure by taking the extraction ratio,

$$R_i = q_0 \cosh \frac{\left(\frac{\beta x}{l}\right)}{\overline{U}h}$$

Where the value of x applies to the midpoint of the tube section i and q_0 as derived in step (5). If the recalculated trapping efficiency is not significantly different from the one computed for uniform extraction then the tube dimensions are corrected, otherwise require redimensioning.

4.5 VORTEX SETTLING BASIN

The vortex-settling basin is a fluidic desilting device that uses the vortices of the water flow to extract the bed and suspended load in the power channel or irrigation canal. It is an alternative to the conventional sediment ejector because of its high efficiency, lower escape discharge and low construction cost .It is preferable to settling basin where subjected to a space constraint. Vortex settling basin is usually provided downstream of head regulator, where it is essential to remove sediment coarser than 0.22mm or so and head to operate silt ejector is not available. It is especially suitable as a pre-settling basin in hydropower intakes to remove the coarser particles so as to save the main settling basin from un-necessary loading. The essential feature of the vortex-settling basin is its low flushing discharges about 5% to 10% [31] and high sediment removal ratio, using high velocity of the canal flow.

4.5.1. Principle Of Vortex Desilting Chamber

The vortex-desilting device consists of a cylindrical basin, which is placed tangential to the inlet canal with an orifice installed at the center. The water from the canal enters into the basin tangentially and produces a secondary flow, which moves the fluid-layer near the basin floor towards the orifice of the flushing pipe. These flows produce a combination of free and forced vortex, inside the basin. The sediment particles, heavier than the fluid begins to settle in the tank due to gravity and also rotate with the circulatory flow of water till entrapped by the vortex and removed through the orifice towards a waste channel. The air core developing at the center of the orifice due to all those fluid motion reduces the flushing discharge and makes the system more efficient.

For knowing the phenomenon of sediment settling in a vertex basin the mechanics of fluid flow and the particles flow inside it have to be known [42], [37]. To minimise the size of the circular basin, only the lower third of the depth of the inlet canal flow transporting large sediment loads has been lead into the basin. The upper two-thirds depth is bypassed into the downstream canal by providing a horizontal diaphragm in the inlet canal at a height, $h_1 = h/3$, where h is the depth of flow in the inlet canal. A deflector has also been provided that extending from the entrance of the inlet canal and covers half the circumference of the basin including the spill weir. A typical plan and cross section of vortex settling basin is given in Fig. 4.18. The following notations are used to define the physical parameter of the basin.

B = Inlet canal bed width.

h = depth of flow in the canal.

S = bed slope of the canal.

Qc = inflow discharge, full-supply discharge.

Qcc = discharge entering the basin.

Qo = Flushing discharge.

Qs = Overflow discharge = (Qcc-Qo).

d = Basin diameter.

H = Basin height i.e height of (basin) sidewall.

d_o = Flushing pipe diameter.

Sc = Radial slope of the basin floor.

h_1 = Height of diaphragm from bed of inlet canal.

h_2 = Basin depth at its periphery from inlet canal bed.

L_1 = Length of over flow weir.

h_o = depth of flow over orifice.

h_p = Water depth at basin periphery.

ω = Settling velocity of particle in quiescent water.

Z = Side slope of inlet canal (Z horizontal to one vertical)

Before outlining the design parameter of the vortex-settling basin it is important to know the fluid flow and particle flow behaviour inside the basin. These things are discussed in the following paras.

4.5.1.1. Fluid Flow

When the flow of water enters tangentially into a circular basin with an axial outlet pipe at its center, it is set to circulatory motion with respect to the vertical axis through the center of the basin. According to Cecen & Akmandor and Mashauri [37] the circulatory flow inside the basin consists of two types of flows [Fig. 4.19(a)].

- A forced vortex in the outer region i.e zone 1, $r_c \leq r \leq R$
- A free vortex in the inner region i.e zone II, $0 < r \leq r_c$

where, r = The radial distance from the center of the basin or orifice.

R = The radius of circular basin.

r_c = The distance from the center of the basin to the intersection of the forced and free vortices.

ro = Radius of flushing pipe.

According to Anwar and Yalien [37], [17] the circulatory motion produces a Rankine combined vortex comprising

- A forced vertex core in the inner region, r = 0 to ro, where, in a relatively small central portion fluid rotates as a highly viscous solid body. Mathematically $Vt = \varpi r$

- A nonviscous free vortex in the region, r = ro to R, i.e, extending radially out ward from the forced vortex core. Here the flow is irrotational and

$$Vt = \frac{\tau}{2\pi r} = \frac{\tau/2\pi}{r} = \frac{C}{r}$$

V_t = Tangential velocity.

τ = Constant circulation.

ϖ = Angular velocity.

C = Swirl or free vortex constant.

The flow characteristic is shown in the Fig. 4.19(b). The free and forced vortices intersect at a distance r_c defined by $r_c = \sqrt{\frac{C}{\varpi}}$

4.5.1.2 Particle Flow Behaviour

The particle flow behavior in the vortex flow inside the basin can be obtained by using Navier-Stokes equation [37], based on the assumption that the sediment concentration is low. So the settling velocity can be used for the flow analysis. The particle trajectories, can be calculated for zone II in Fig. 4.19(b) by assuming that

- There is no slip between particle and fluid
- Radial velocity 'V_r' is very small.
- Particle radial velocity, $V_{rs} = \dfrac{dr}{dt}$
- Particle velocity, Vu is negligible.

4.5.2 Design Parameters

The following are the different design parameters of vortex settling basin, which are briefly, discussed [10], [17], [37], [9], [31].

Basin Diameter: Various correlations for basin diameter for vortex settling basin are in the literature. These are as below.

- Sullivan (1972) proposed the basin diameter to be 6B
- Salakov (1975) gave a relation for exclusion of sediment size or diameter(Ds) between 0.5 to 1mm and the settling velocity is taken as 0.117 m/s at 20°c

$$d = \left(\frac{2Qcc}{\omega}\right)^{\frac{1}{2}}$$

- Cean et.al in 1975 putforward the relation 5.274A*, where

$$A^* = \left(\frac{k_1 k_2 U_*}{V_{tp}}\right)^{\frac{1}{4}} \left(\frac{Qcc}{\omega}\right)^{\frac{1}{2}}$$

$$k_1 = \left(\frac{\omega}{U_*}\right) = \text{Particle mobility number.}$$

$$k_2 = \frac{\omega d}{V_{tp} h_p} = \text{Coefficient of settling.}$$

U_* = Shear velocity of flow in vortex chamber..

$$V_{tp} = \frac{Q_{cc}}{A_i} = \text{Tangential velocity at basin periphery.}$$

A_i = Flow area in the inlet canal.

- Paul et al. (1991) proposed the relation as

  d = 5B.

Flushing Discharge: Salakov (1972) found that the depth h_2 at basin periphery if more, it would only marginally improve sediment removal efficiency but it is not always consistent. Chrysostomm (1983) shows that a ratio of $\frac{h_2}{h_1} = 0.6$ is an optimum basin depth because any further reduction in $\left(\frac{h_2}{h_1}\right)$ simply decreases the residence time. So the peripheral basin depth, h_2 should be 0.6 h_1 or 0.2h, which yields the better efficiency.

As per Mohammad and et. al.[31] a valve should be provided at outlet of flushing pipe so depths of flow and slope of vortex chamber shall not be affecting the value of flushing discharge Qo and the most appropriate value of $\frac{Qo}{Qc}$ is 0.1

Basin floor slope: The sloping floor helps in flushing the coarse material more easily but increase the flushing discharge. Ceccen (1977) proposed the slope of the basin floor, Sc as 1:10 for better trapping efficiency and with least flushing discharge.

Depth Of Flow: Two depths of flow are consider in the design of vortex settling basin.

1) h_p, the depth at basin periphery which provides a basis to set the height of side wall.
2) h_0, the depth of flow over orifice which influences the flushing discharge Q_0

Mashauri (1986) suggested that the height of sidewall 'H_0' should be greater than 0.26d with a free board of 0.3m. Mohammad suggested an alternate type of vortex basin with inlet and outlet level of channel bed is at different level [Fig. 4.20] and the ratio of $\frac{Z_h}{h_p}$ should be equal to 0.1, where Z_h is the elevation difference between inlet and outlet channel beds at their junctions with the vortex chamber.

Size Of Orifice: Diameter of the flushing pipe is computed by trial and error method from the equation given by Paul et.al (1987) with mean discrepancy ratio, DR = 1.009, standard deviation 0.058. The relation is as follows

$$Q0 = Cd\pi do^2 \sqrt{\frac{gh_0}{8}}$$

where,
$$C_d = \frac{0.22R^{0.075}N_T^{0.054}F^{0.965}}{\left(\frac{h_o}{d_o}\right)^{0.375}}$$

$R = \frac{Q_o}{\nu d_o}$ = the Reynolds number.

ν = Kinematics viscosity of fluid.

$N_T = \pi d_o^2 V_{to}/Q_o$= Circulation number

V_{to}= tangential velocity at $r = r_o$

$F = \frac{4Q_0}{\pi d_0^2 \sqrt{gd_0}}$ = Froud number

For better trapping efficiency Cecen and Bayazit (1975); Mashauri(1986) has suggested the parameter below for better trapping efficiency.

$$22.5 \le d/d_o \le 40 .$$

As per Mohammad [31] the orifice diameter for regulated flushing pipe should be equal to the least of 0.10d and 0.2m.

Trapping Efficiency: The trapping efficiency of the vortex-desilting basin (p), diameter of flushing pipe, (d_o) and the flushing discharge (Q_o) depends upon the grade of sediment transported by the inlet canal. The Table.4.4 shows the relation among them suggested by T. C. Paul (1988).

TABLE 4.4: Sediment Size and Trapping Efficiency in Vortex Desilting Basin

Sediment size Ds	d/d_o	Q_o/Q_c	Formula for trapping efficiency p%
Less than 0.5 mm	35 to 40	3 to 5	$98+0.92\log(\omega W)$
0.5mm to 1.0mm	30 to 35	5 to 8	$98+0.92\log(\omega/W)$
Greater than 1.0mm	25 to 30	8 to 12	$97.8\left(\frac{V_s}{V_{T0}}\right)^{0.0045}\left(\frac{Q_0}{Q_c}\right)^{0.01}$

where, ω = settling velocity of sediment particles.

$W = \left(\dfrac{4Q_s}{\pi d^2}\right)$ = Vertical upward velocity at the center of the basin.

$Q_s = (Q_{cc} - Q_o)$ = Overflow discharge

$$Vto = \left(\frac{2gh_0}{3.45}\right)^{0.5}$$

Mohammad et.al [30] however found a dimension-less relationship of efficiency from the experimental data for different types of geometric configurations [Fig. 4.20] of the vortex chamber with $\pm 40\%$ error as follows.

$$P = Ko\left[\frac{Q_0}{Q_c}\right]^{0.25}\left[\frac{Z_h}{h_p}\right]^{0.35}\left[\frac{\omega\phi}{\nu}\right]^{0.15}\left[\frac{Q_w^2}{gR^3h_p^2}\right]^{0.11}$$

where, K_0 =10.24 and 6.17 for geometric configuration I and II respectively as suggested by Mohammed.

Z_h = Elevation difference between inlet and outlet channel beds at their junctions with the vertex chamber.

ϕ = Sediment size

$Q_w = Qo + K_0(Qc - Qo)$

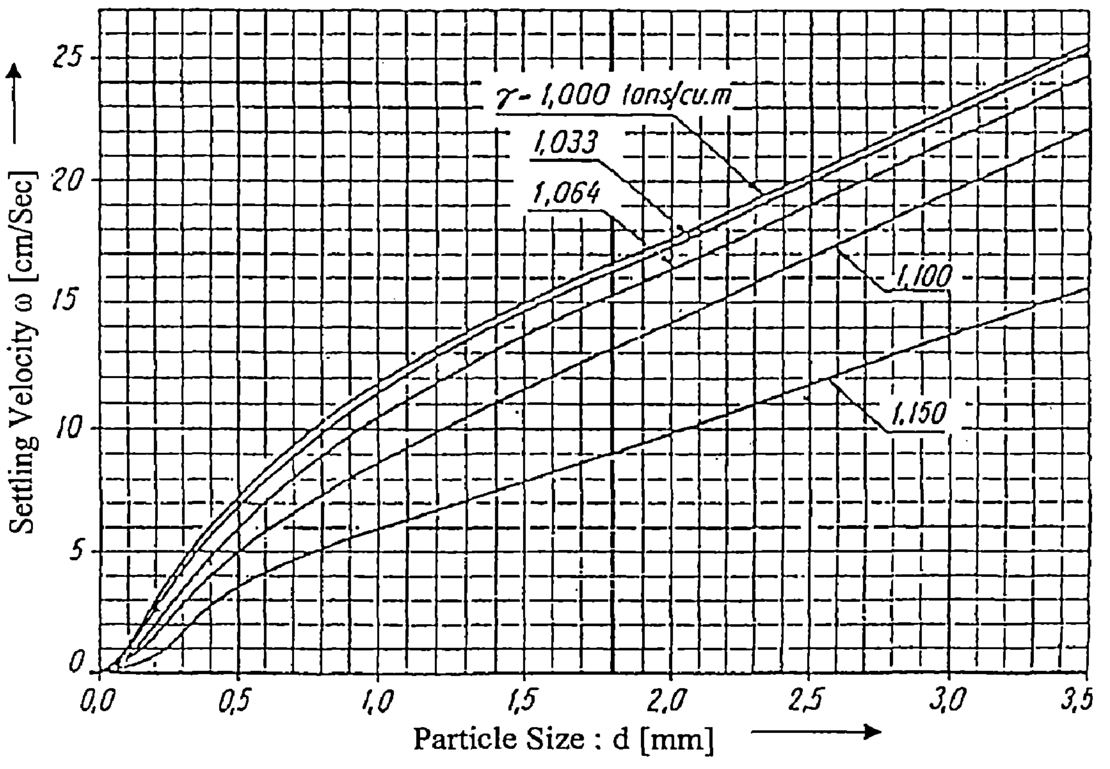


Fig.4.1: SETTLING VELOCITY IN STAGNANT WATER (SUDRY'S CURVE)

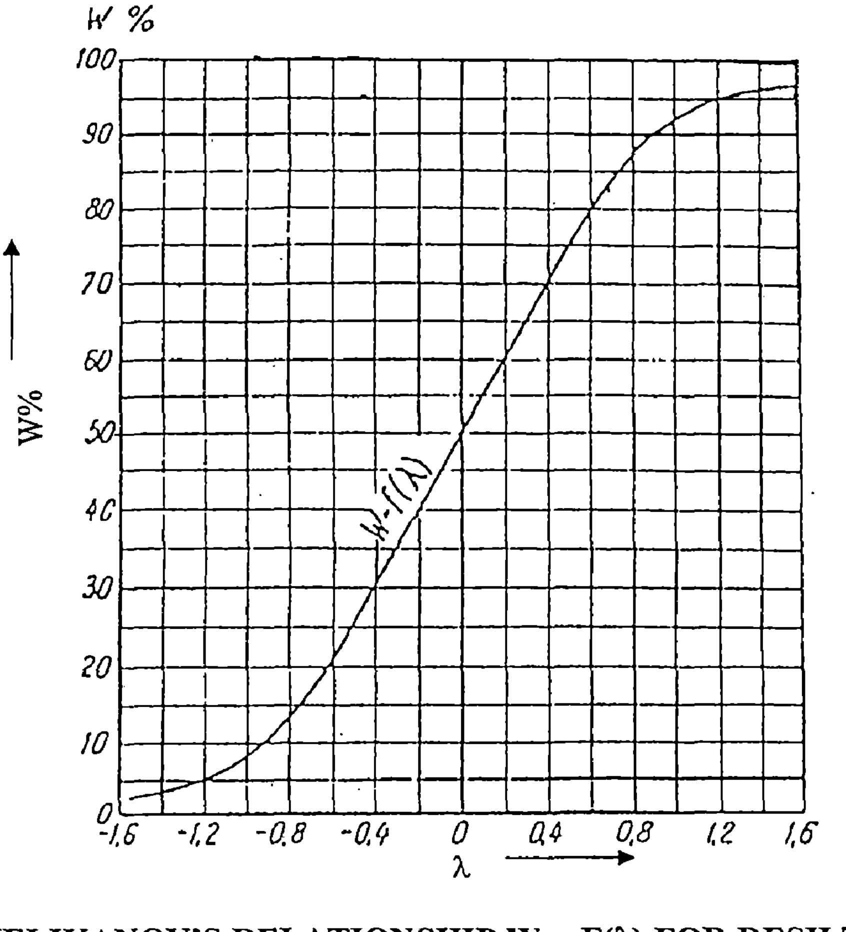


Fig.4.2: VELIKANOV'S RELATIONSHIP W = F(λ) FOR DESILTING SETTLING BASIN

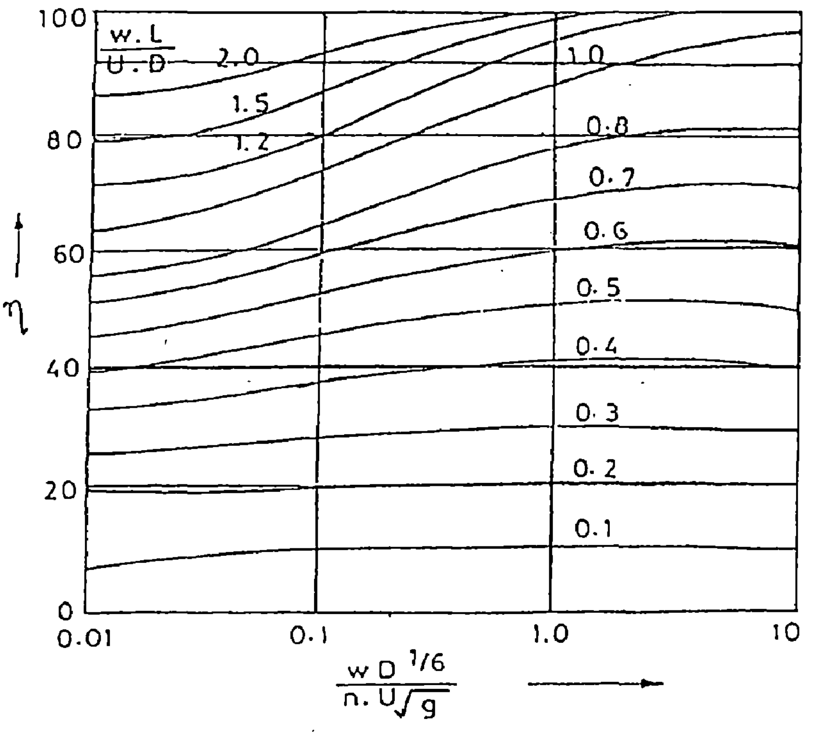


Fig.4.3: CAMP AND DOBBIN'S RELATION FOR EFFICIENCY OF DESILTING BASIN

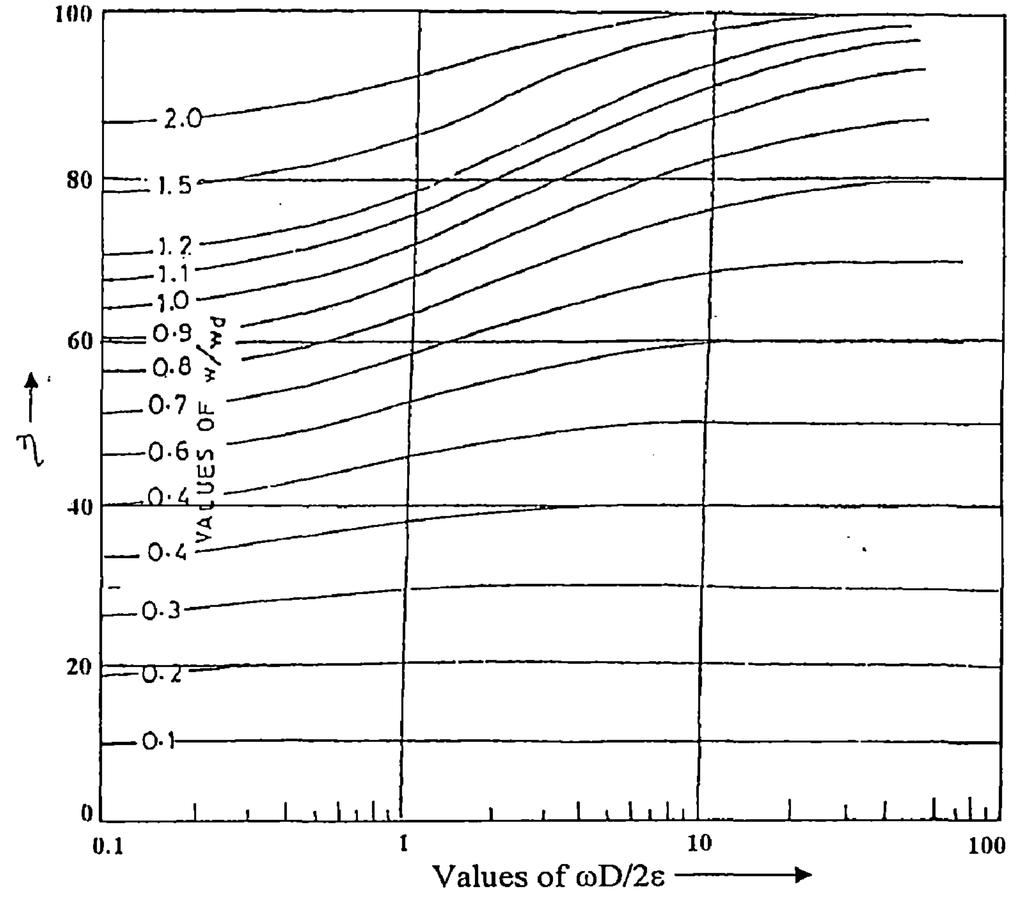


Fig.4.4: EFFICIENCY IN TERMS OF MIXING COEFFICIENT

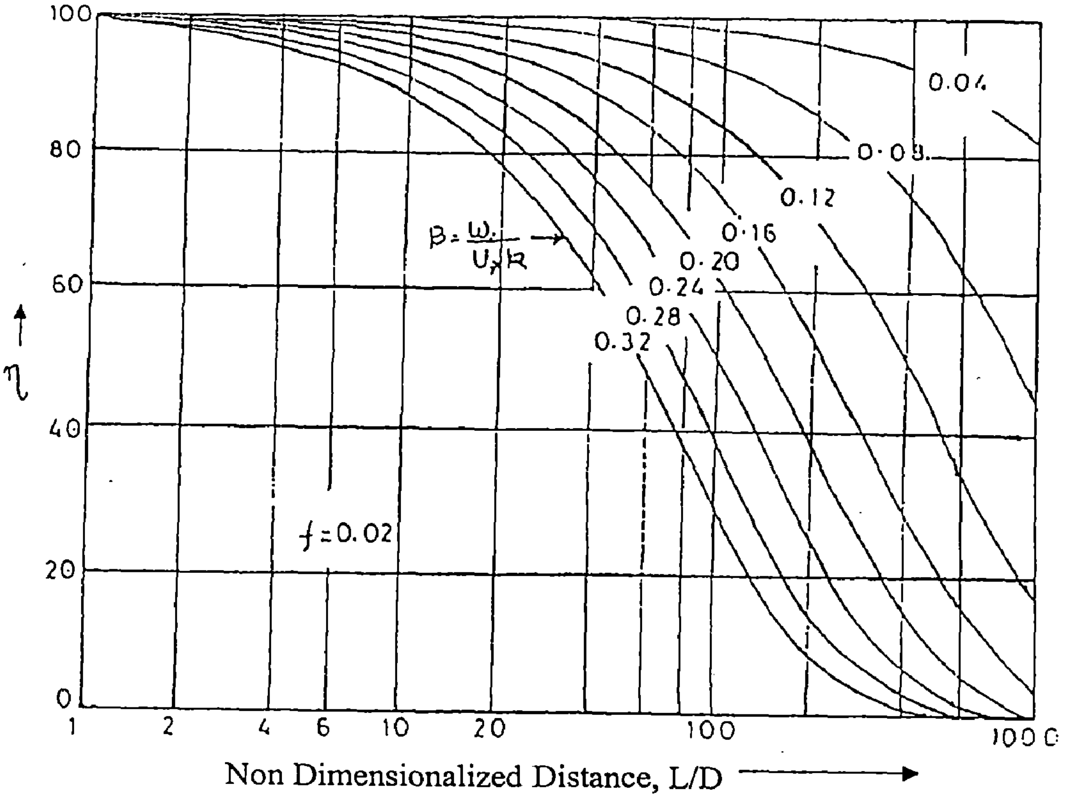


Fig.4.5: SUMER'S EFFICIENCY CURVES FOR DESIGN OF SETTLING BASIN

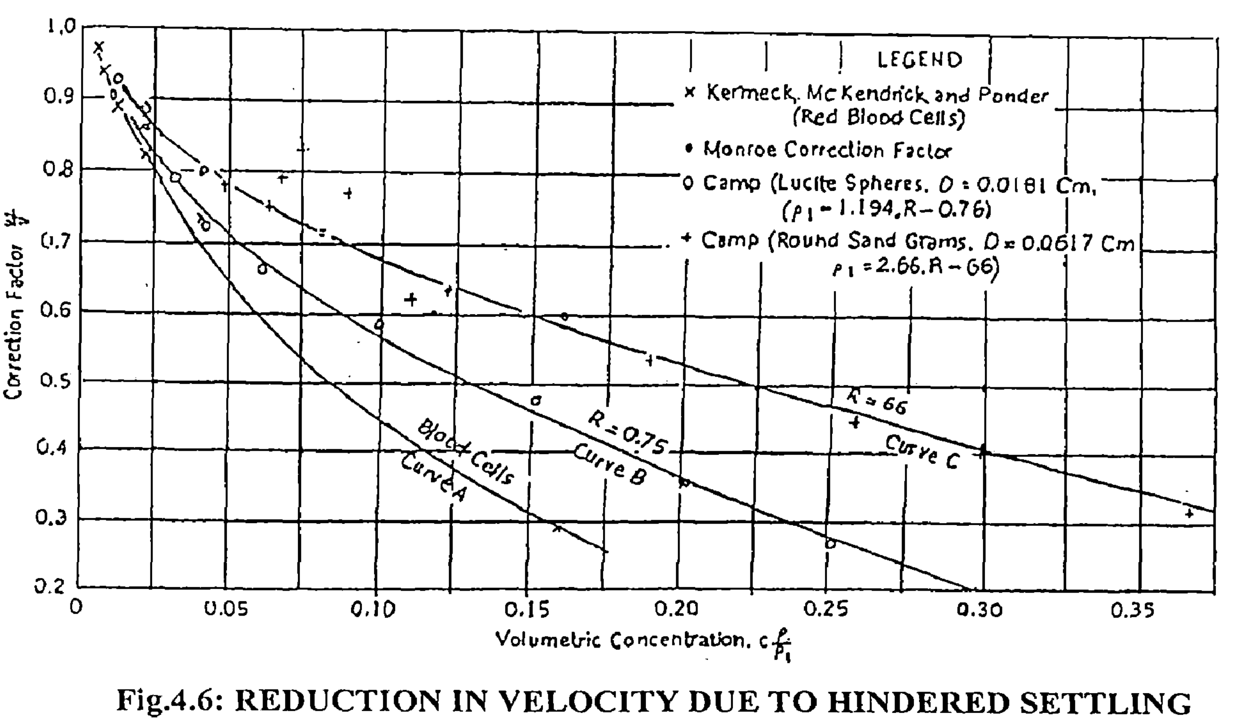


Fig.4.6: REDUCTION IN VELOCITY DUE TO HINDERED SETTLING

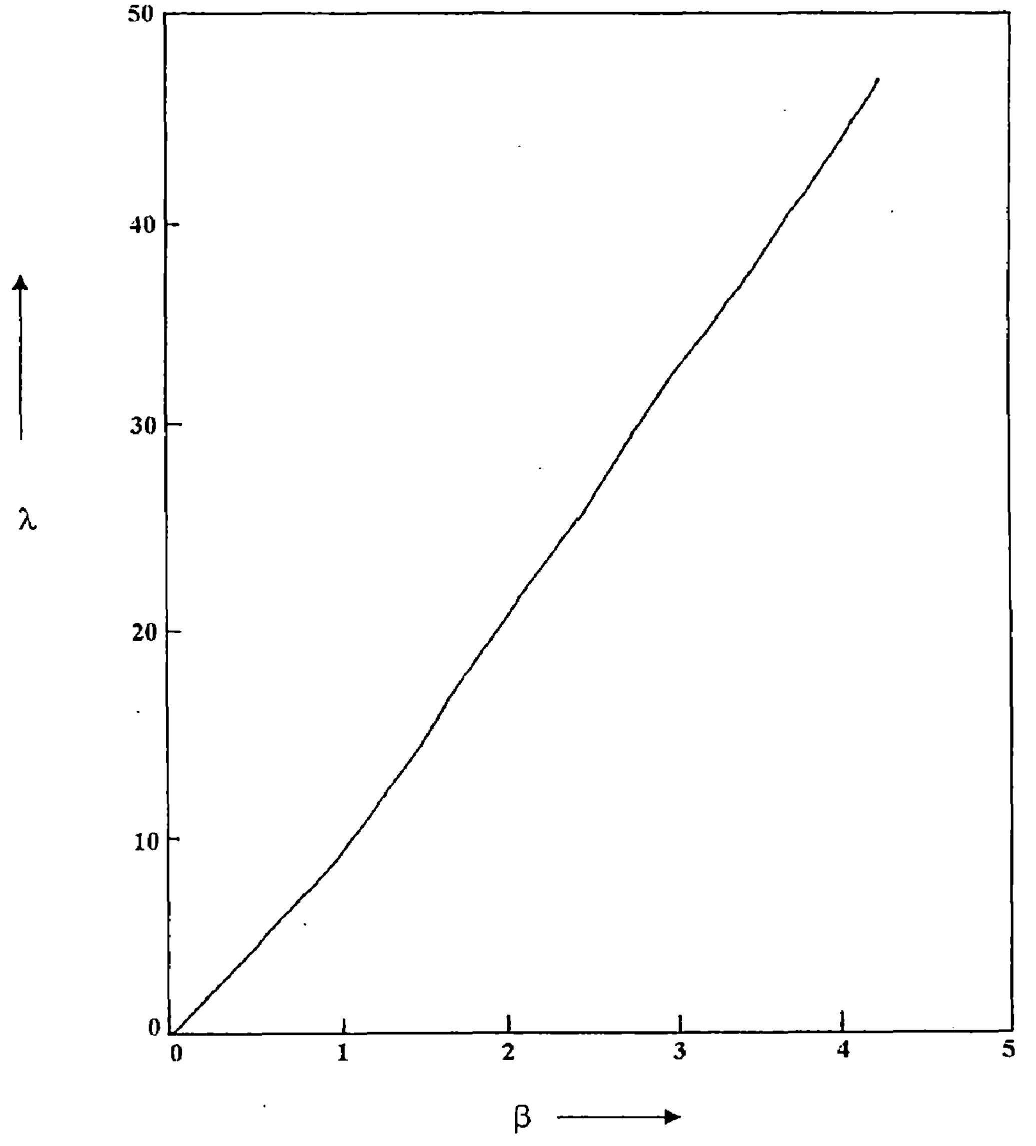


Fig.4.7: RELATION BETWEEN λ AND B FOR PARTICLE SETTLING

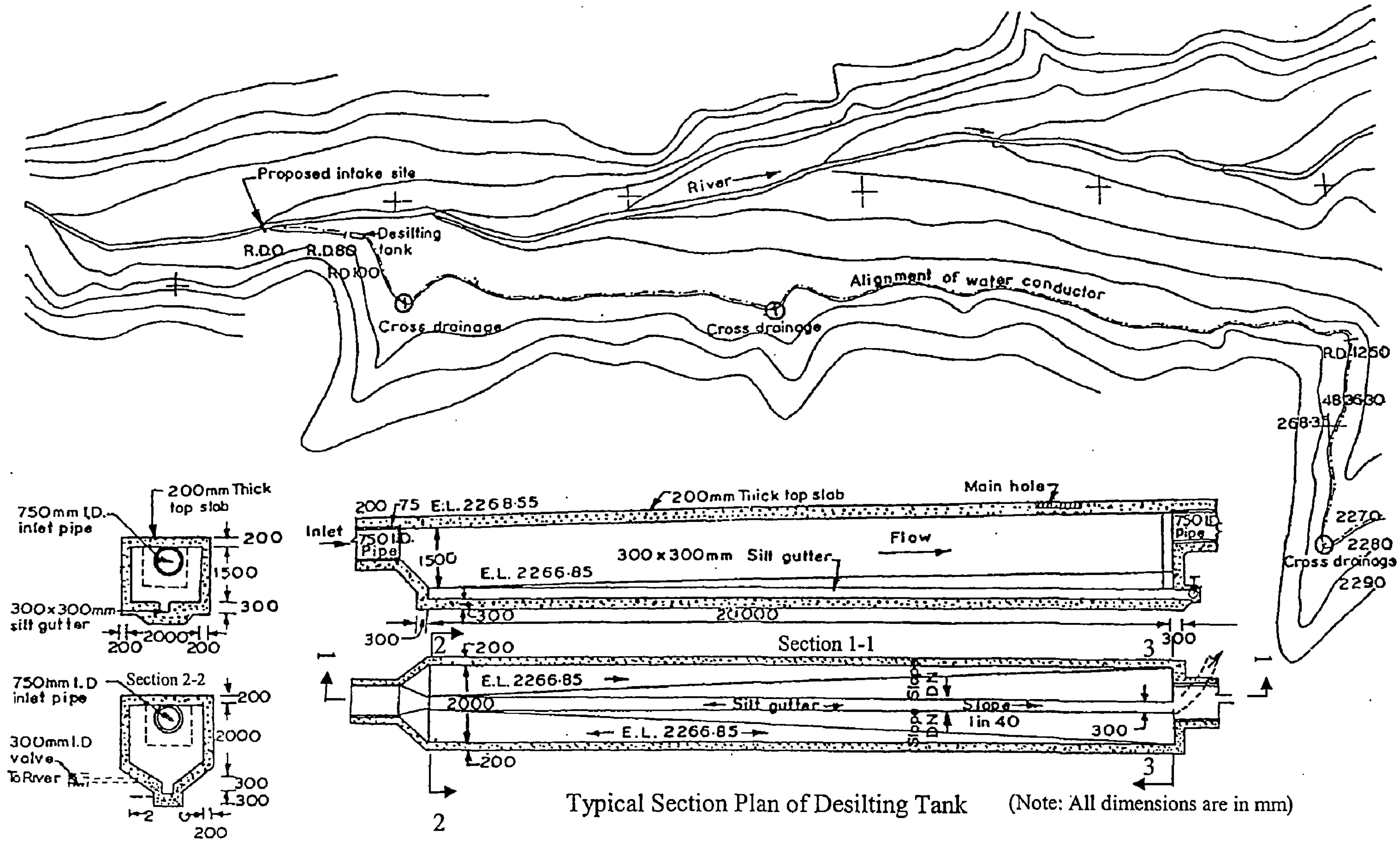


Fig.4.8: TYPICAL LAYOUT OF DESILTING TANK

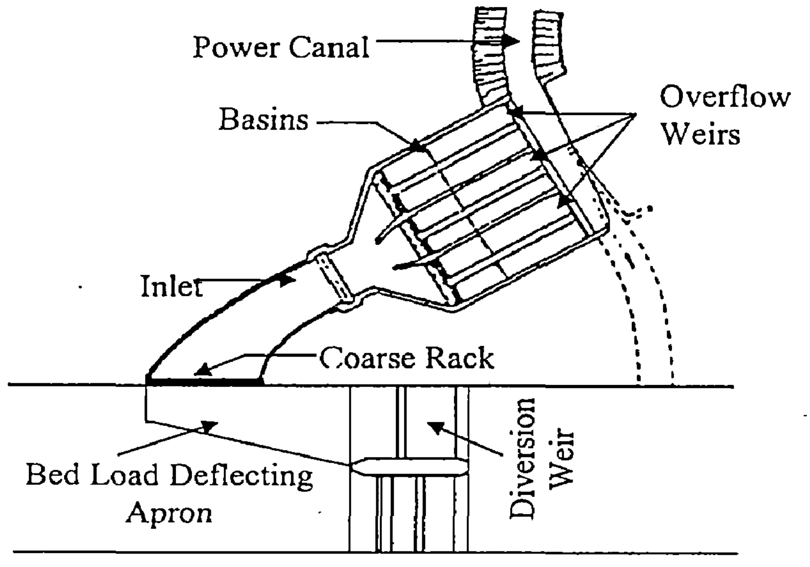


Fig.4.9: MULTICHAMBER SETTLING BASIN FOR CONTINUOUS OPERATION

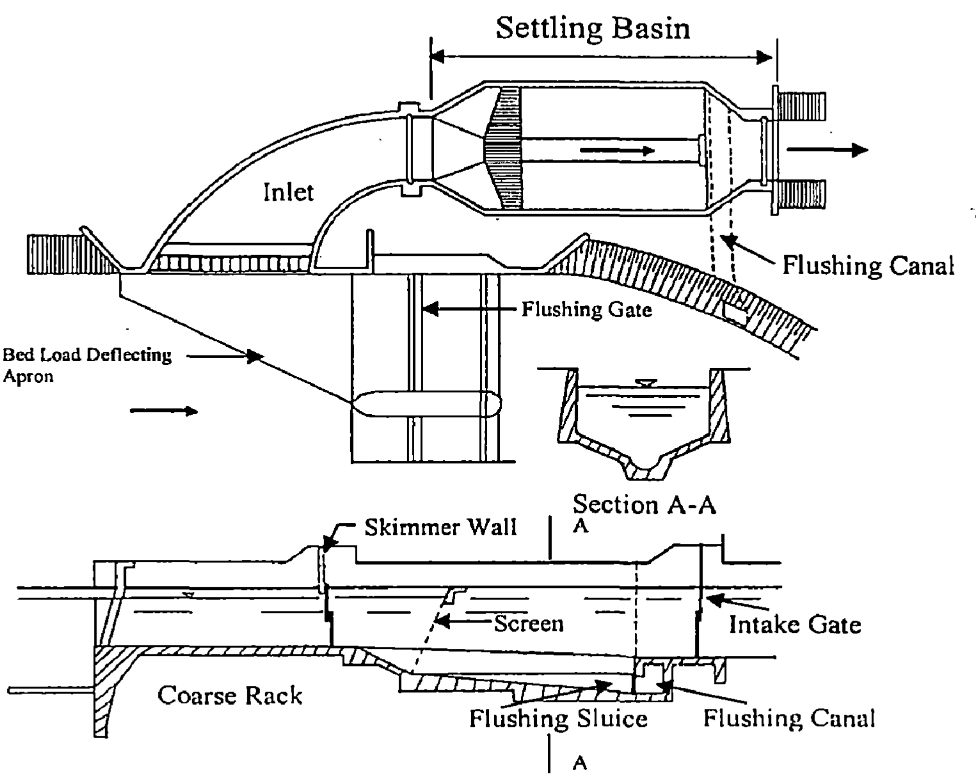


Fig.4.10: GENERAL ARRANGEMENT OF CONTINUOUS TYPE DESILTING BASIN

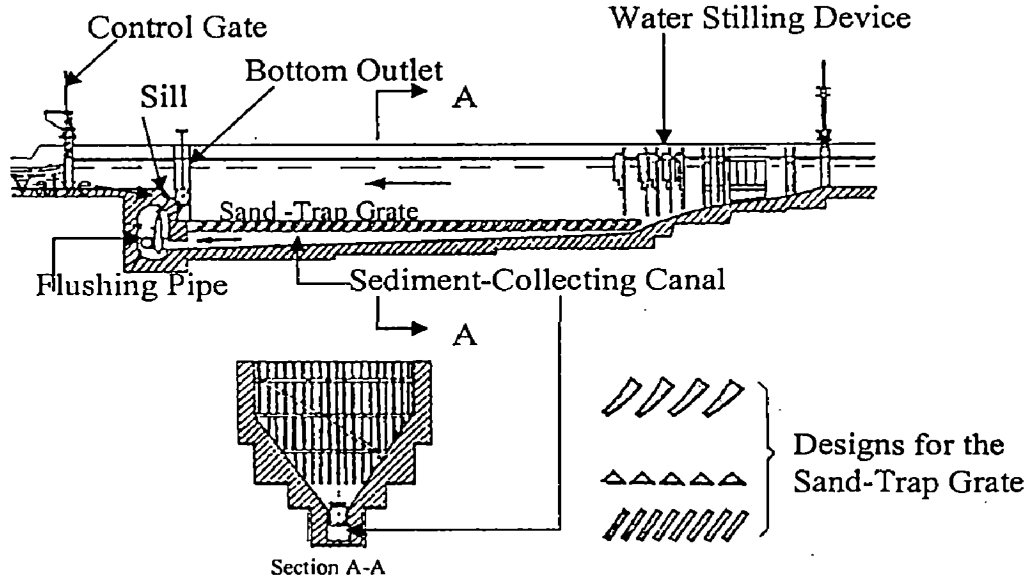


Fig.4.11: DUFOUR TYPE SETTLING BASIN

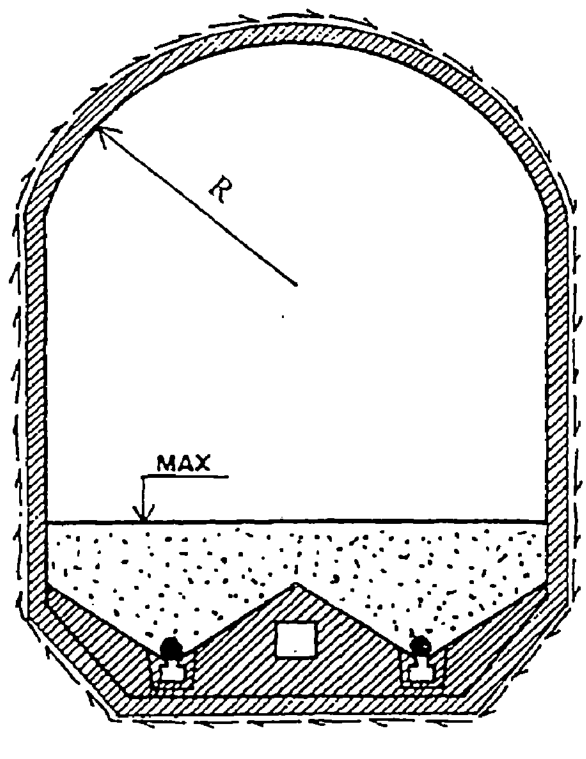


Fig.4.12: PRESSURISED SEDIMENTATION BASIN WITH SERPENT ARRANGEMENT

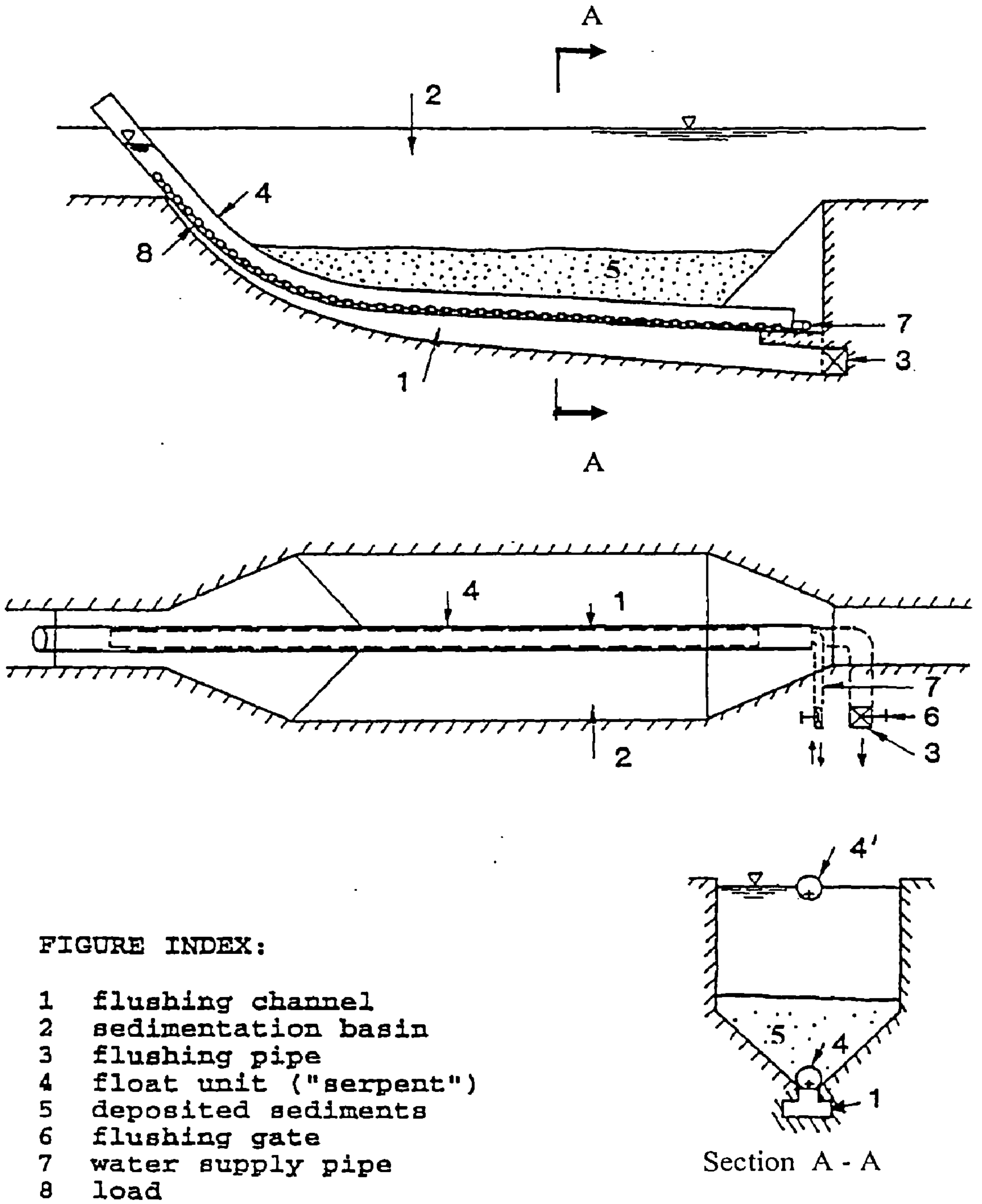
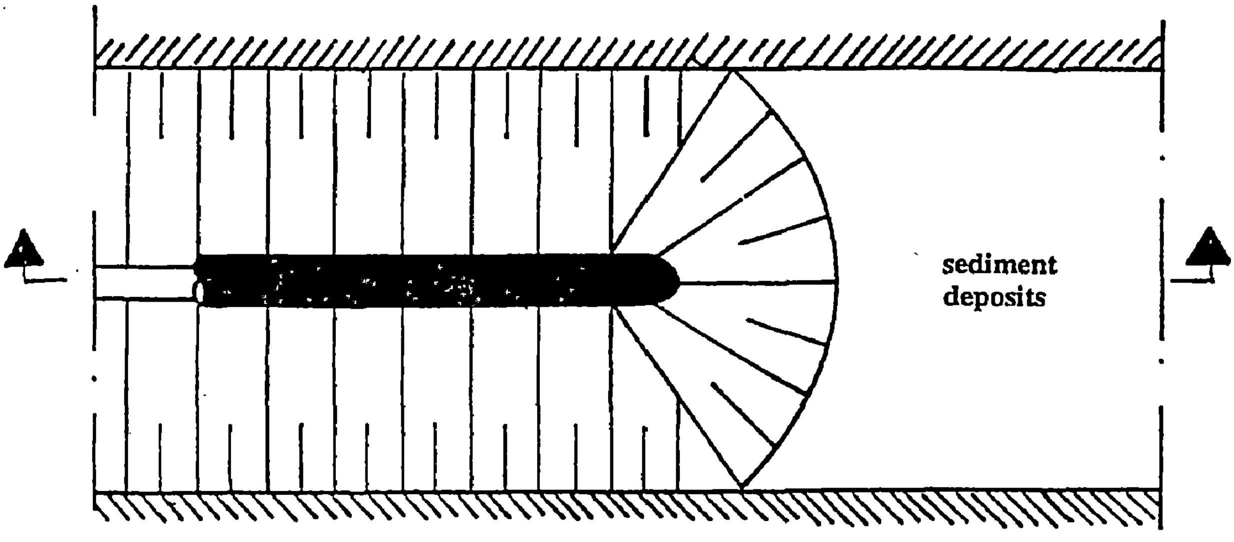
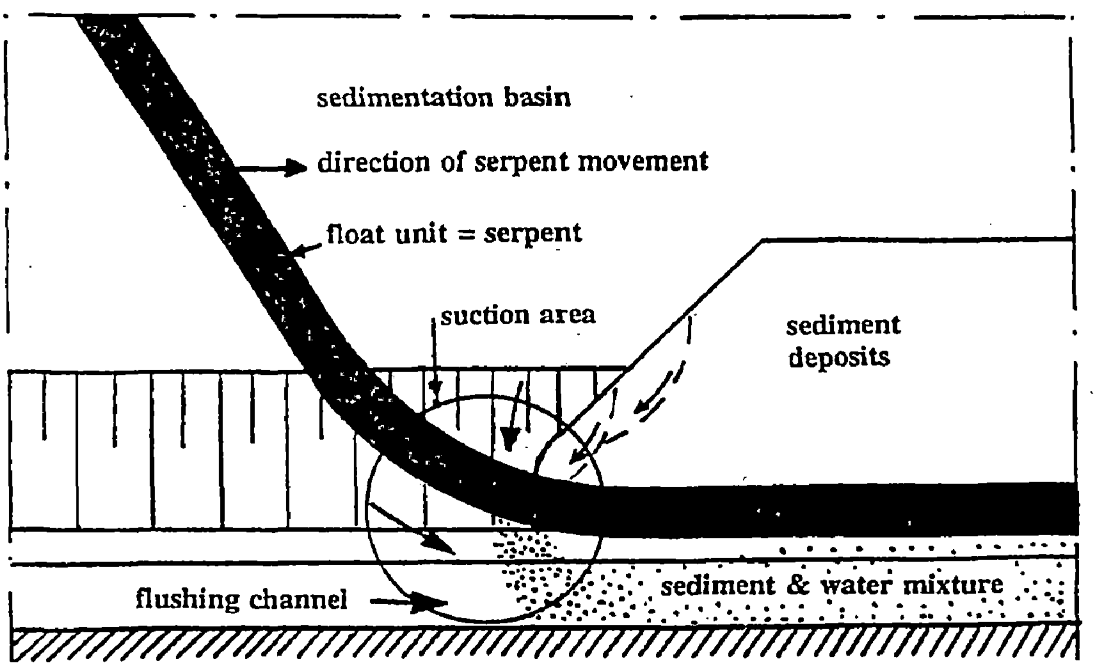


Fig.4.13: SERPENT SEDIMENT SLUICING SYSTEM SEDIMENTATION BASIN

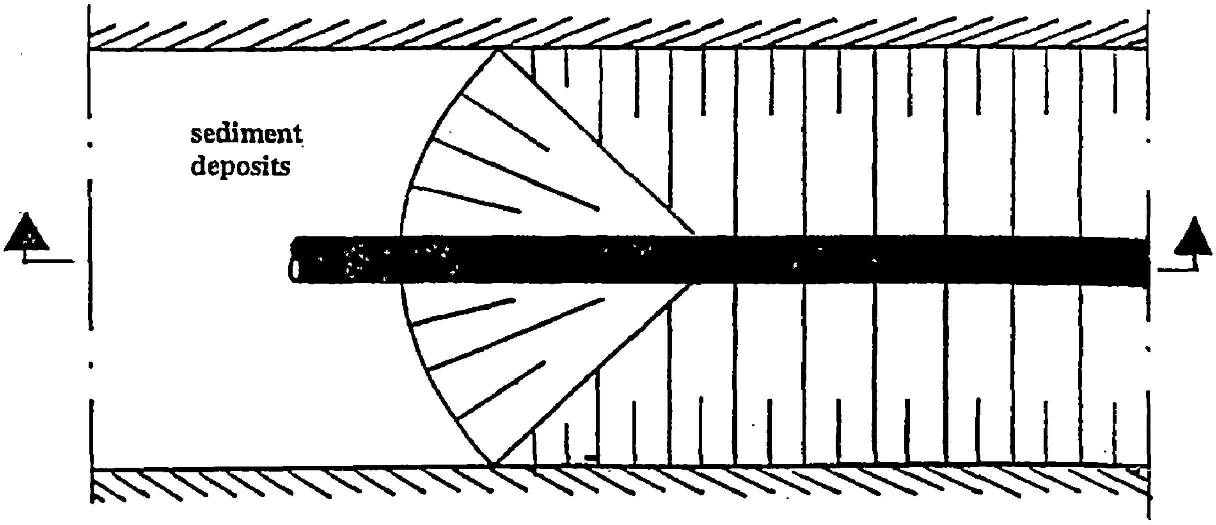


PLAN VIEW

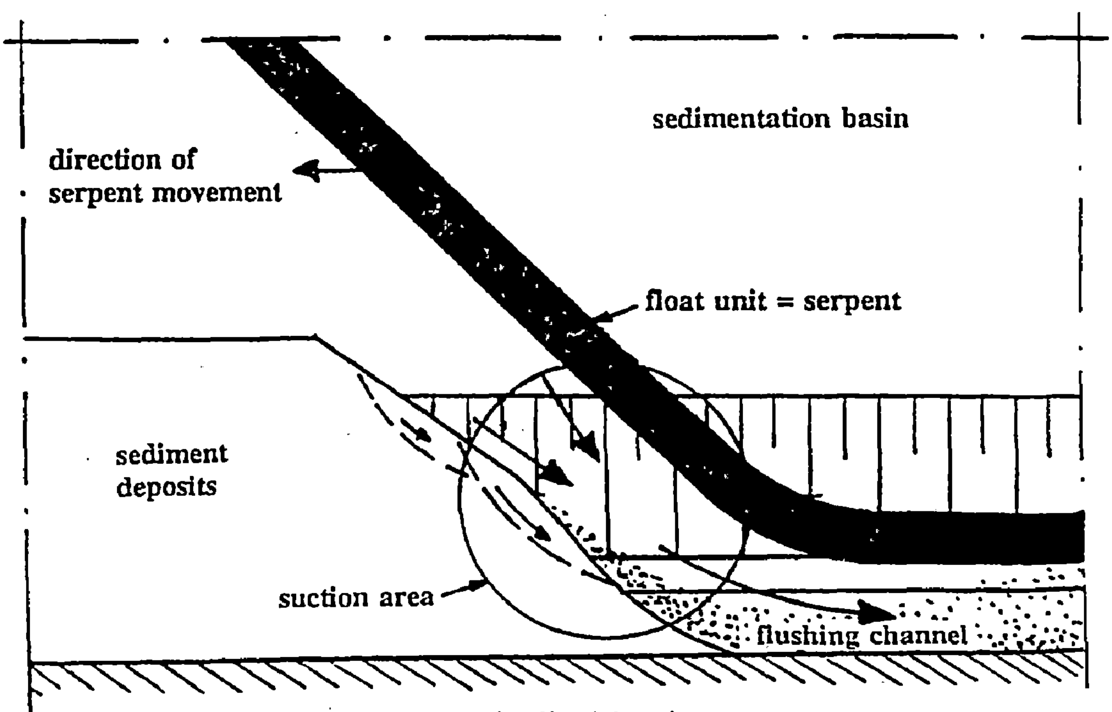


Longitudinal Section

Fig.4.14: SEDIMENTATION REMOVAL IN OPENING MODE

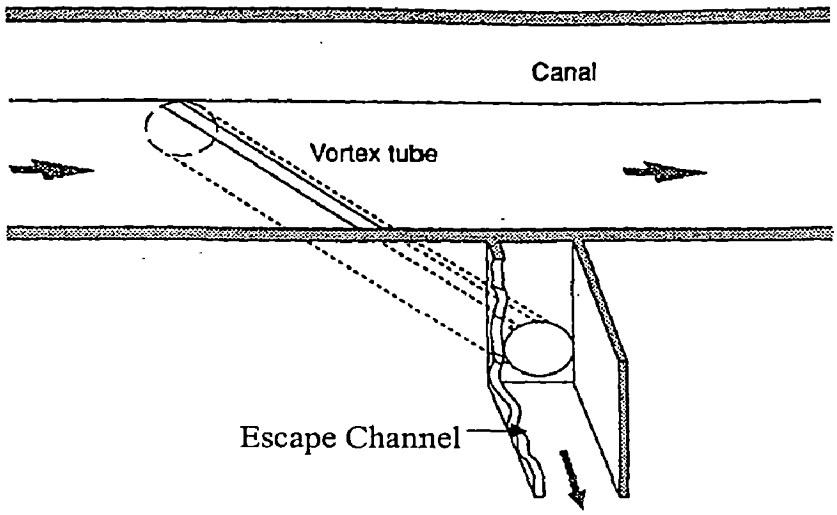


PLAN VIEW

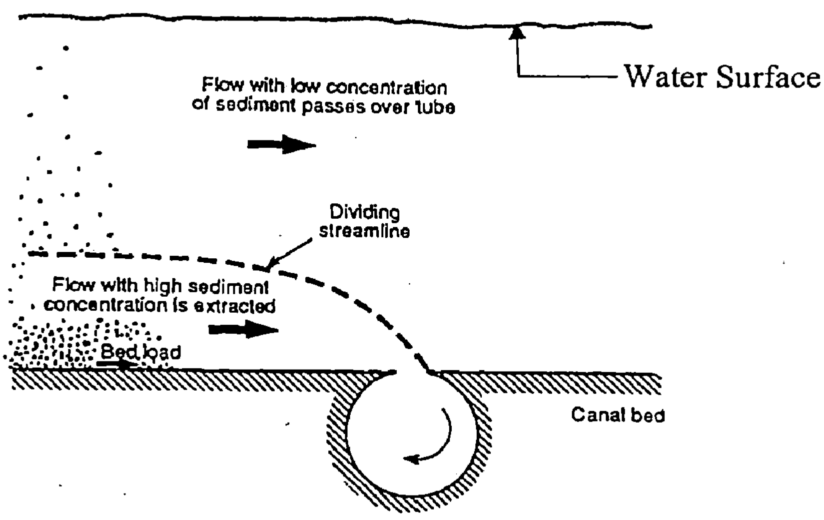


Longitudinal Section

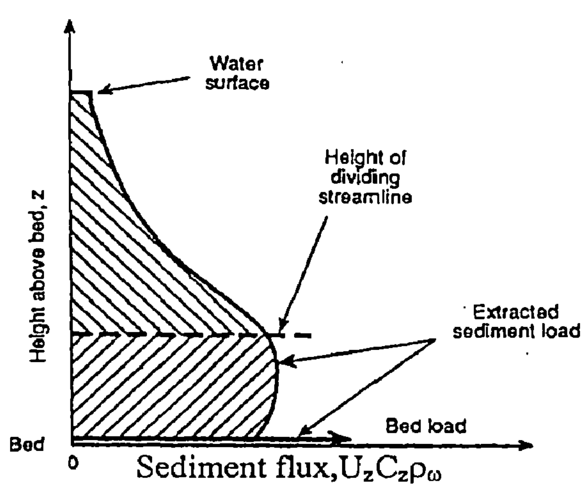
Fig.4.15: SEDIMENTATION REMOVAL IN CLOSING MODE



(a) Vortex Tube Sediment Extractor



(b) Elevation Showing Dividing Stream Line



(c) Sediment Flux Profile

Fig.4.16: VERTEX-TUBE SEDIMENT EXTRACTOR

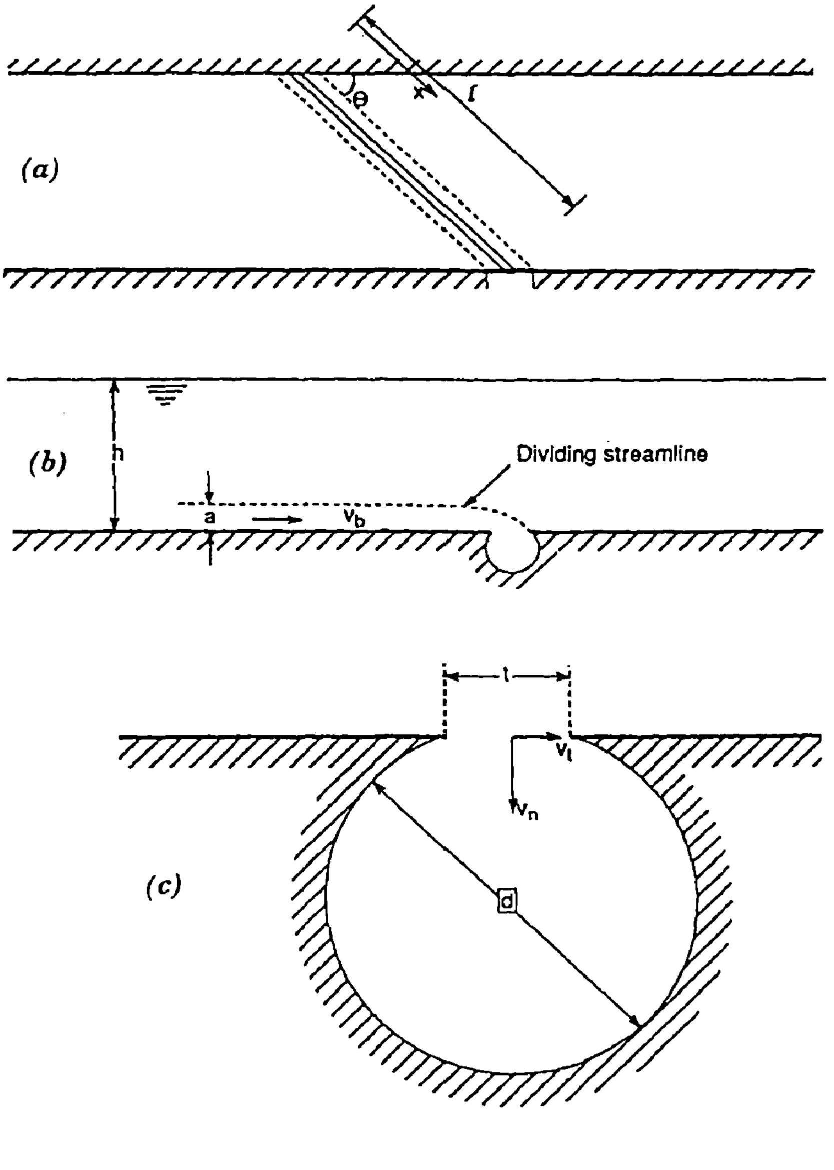
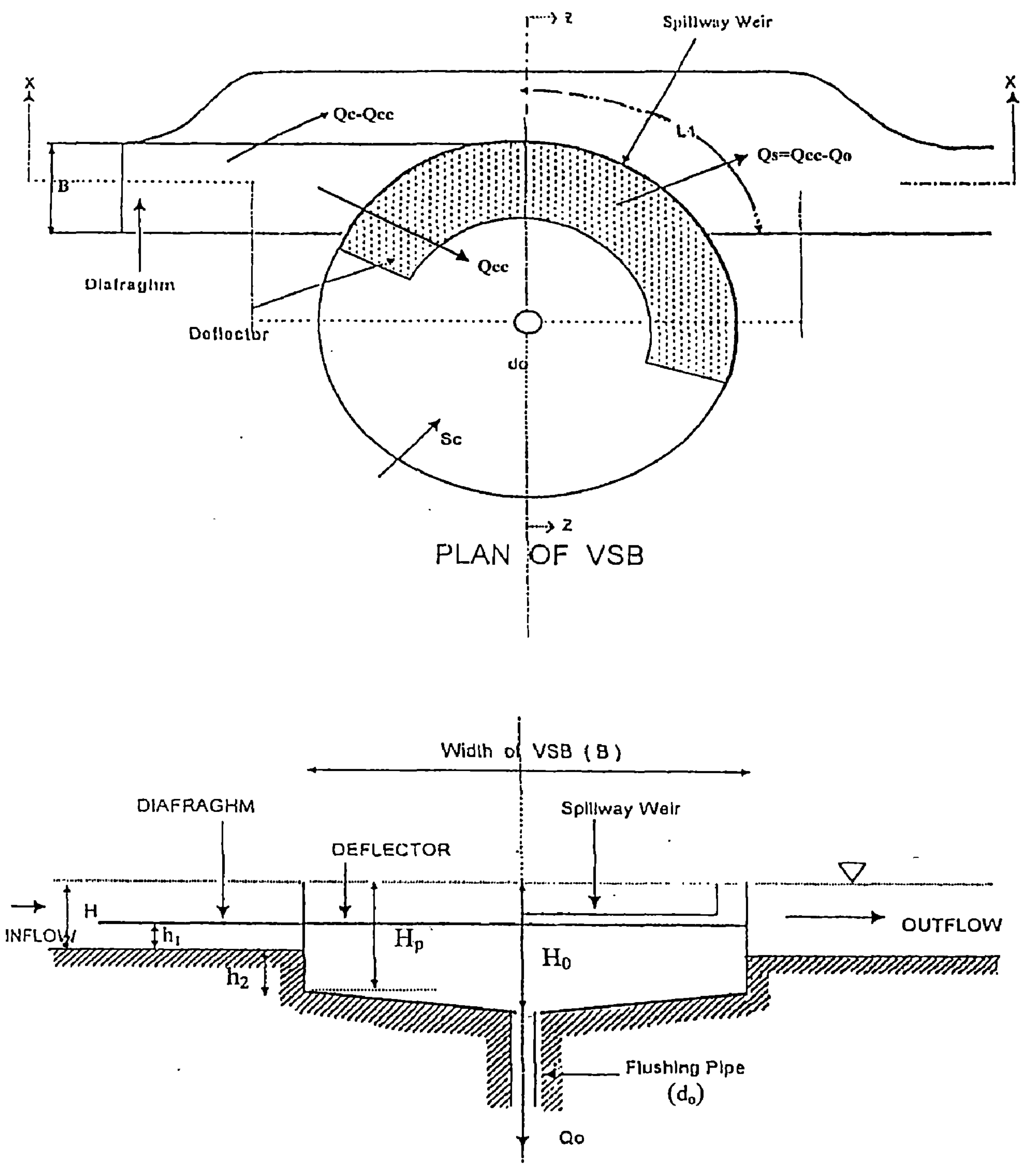


Fig.4.17: FLOW ANALYSIS IN VERTEX TUBE SEDIMENT EXTRACTOR: (a) PLAN; (b) ELEVATION; AND (c) SECTIONAL OF VERTEX TUBE NORMAL TO TUBE AXIS



Section X-X

Fig.4.18: PLAN AND SECTION OF VERTEX SETTLING BASIN

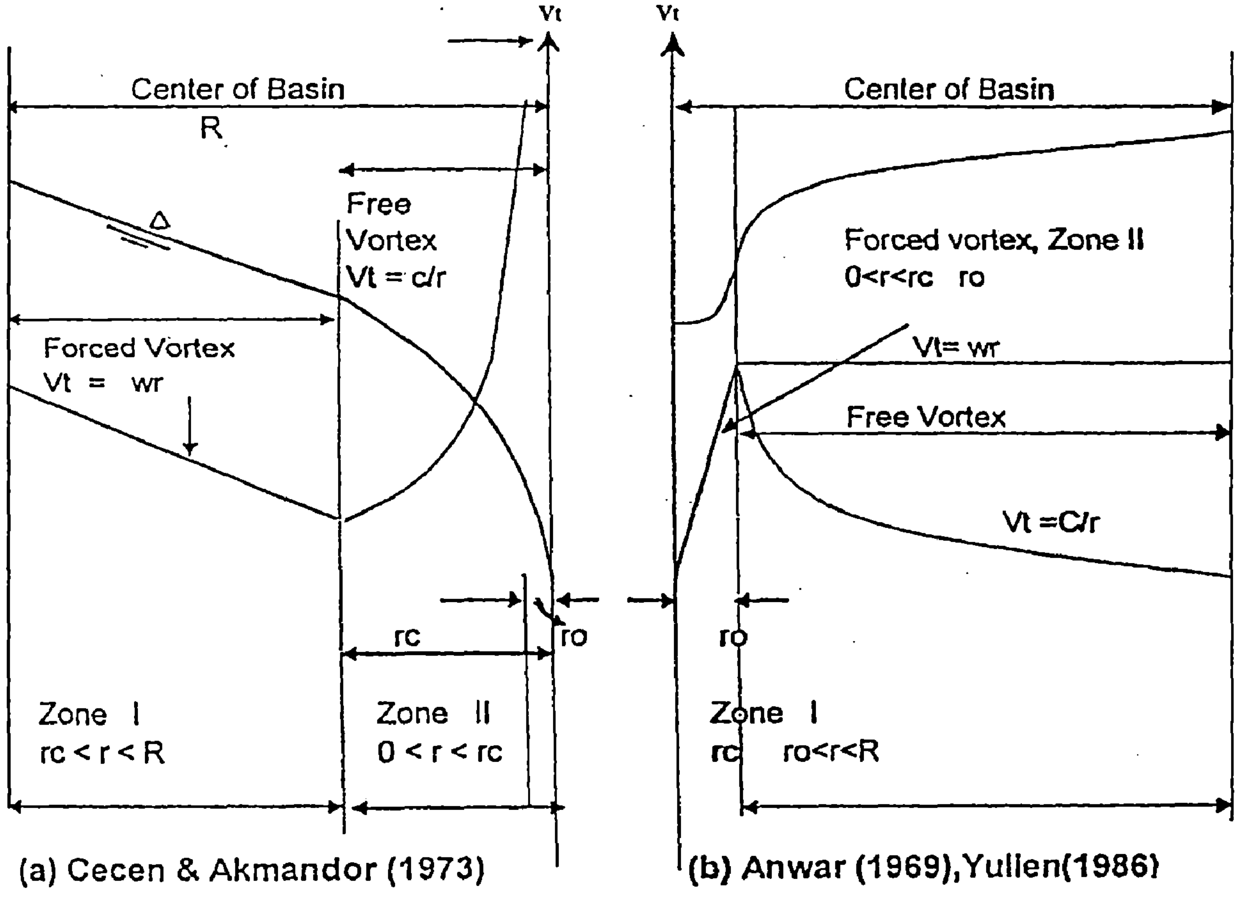
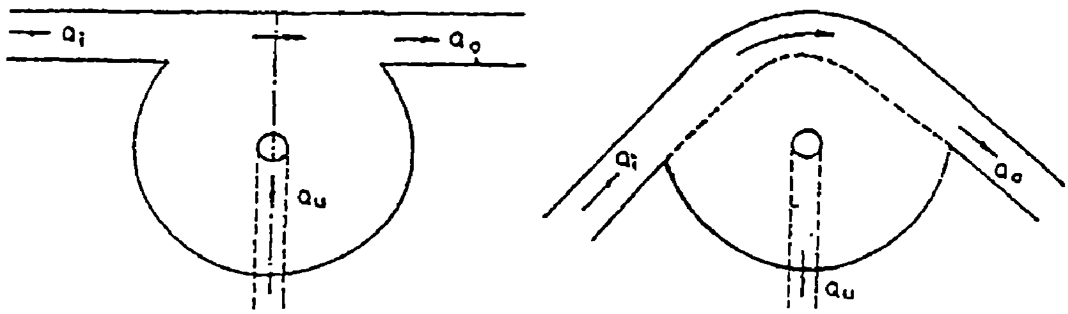
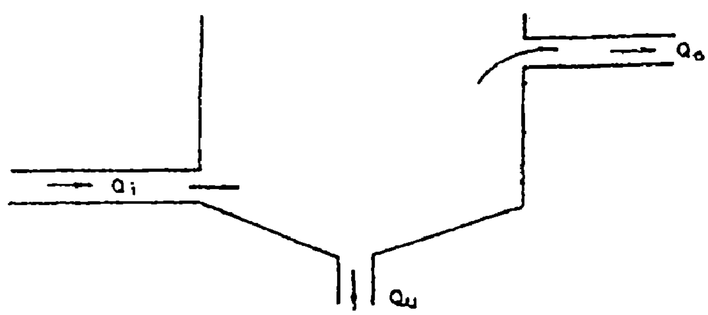


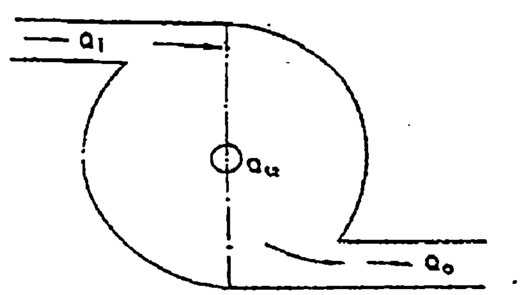
Fig.4.19: FLOW CHARACTERISTICS IN VERTEX SETTLING BASIN



PLAN VIEW OF GEOMETRIC CONFIGURATION - I



SECTIONAL VIEW OF GEOMETRIC CONFIGURATION - II



PLAN VIEW OF GEOMETRIC CONFIGURATION - III

Fig.4.20: TYPES OF GEOMETRIC CONFIGURATIONS OF THE VORTEX CHAMBER SEDIMENT EXTRACTOR

CHAPTER 5

HYDRAULICS AND DESIGN PARAMETERS OF WATER CONDUCTOR SYSTEM

5.1 GENERAL

The water conductor system in a small hydro may be in the form of a open channel, where the natural bed slope is somehow flatter or a closed conduit where it is supposed to place along the steep hill slopes. Small hydropower plants in mountain and hilly regions are generally combinations of relatively small discharges with medium to high heads. As the discharge is small, so also the size of the head race channel or conduit. Special care should be taken to safeguard the water conductor system from various adverse topographical and geological conditions. The headrace channel or conduit is frequently subjected to damage due to soil erosion, landslides and rock falls. Hence the headrace and its repair and maintenance costs a lot and as such affects the economic viability of the project.

The possible layouts of water conductor systems are described in the chapter 2. The present chapter describes the detailed design procedure of free flow headrace channel and the closed conduit as per the different site conditions.

5.2 DESIGN OF OPEN CHANNEL

A free flow lined power channel should be preferred when the alignment is close to the top of a ridgeline in the hilly region. The lining of the channel is normally necessary to control the loss of precious water by way of seepage which in turn may cause water logging and slope failure. Sufficient catch water drainage should be provided on the hill sides so that transverse flow from hill slopes during rains do not get into channels but are carried safely away to nearby cross drainage works. In heavy rainfall areas or where the soil is loose, the catch water drainage should be protected by stone pitching. A typical cross-section of power channel adopted in small hydro plants is shown in Fig. 5.1.

The design of power channel follows the same principles as any hydraulic channel. While designing the following points should carefully be considered [1], [16].

(a) The alignment of the channel should be in flatter slope in order to minimise the head loss.

(b) As far as possible the power channel should be lined, so that water losses during conveyance can be minimised with maximisation of power production.

(c) The flow velocity should be adequate (> 0.5m/sec) to prevent reduction of discharging capacity due to settling of silt if any [33].

(d) The velocity of the flow should be less than an upper maximum permissible velocity [Table 5.2] in order to prevent the erosion of lining material if the water carries abrasive materials in appreciable quantities.

(e) Power canals in plain areas are generally built to a slope varying between 5 and 20 cm/km. In rolling or mountainous country slopes a steeper value as steep as 1 or 2m /km can be adopted [33] .

Water discharge in power channel is obtained from the continuity equation

$$Q = AV \text{ in } m^3/sec \tag{5.1}$$

where, Q = Discharge in the power channel (m^3/s)

A = Area of cross section (m^2).

V = mean velocity in the channel (m/s).

For computing the mean velocity Manning-Stickler formula has been found to yield very reliable values for lined canals. The equation adopted is

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}. \tag{5.2}$$

where, V = Mean velocity of fluid in channel.

n = Manning's roughness coefficient and the value depends upon the type of lining. The values are given in Table 5.1.

R = Hydraulic radius.

S = Longitudinal slope of the channel bed.

TABLE 5.1: Types of Lining and the Values of Manning's Roughness Coefficient [1]

SL.NO.	TYPE OF LINING	MANNING'S n
1	Cement concrete lining	0.013-0.015
2	Brick lining	0.014-0.017
3	Asphalt lining	0.013-0.016

TABLE 5.2: Types of Lining and Maximum Permissible Water Velocity [1]

SL.NO	TYPE OF LINING	MAXIMUM PERMISSIBLE VELOCITY
1	Cement concrete lining	2.7 m/sec
2	Brick tile lining	1.8 m/sec
3	Boulder lining	1.5 m/sec

5.3 CROSS-SECTION OF LINED CANALS

For the most economical section, the hydraulic radius 'R' should be a maximum. Theoretically, a semi circular section is the best section for an open channel. However it is not practicable to adopt this type of section. From the practical considerations, a channel section of trapezoidal and rectangular type section is usually selected. The corners of these sections are rounded to have a more efficient section. The Indian Standard (IS 4745-1968) recommends the round cornered trapezoidal section for lined canals of all discharges. The rounding of corners is provided with a radius equal to depth of flow in the canal [Fig. 5.2].

5.4 GEOMETRY OF TRAPEZOIDAL SECTION

The Fig. 5.2 shows a round corner trapezoidal section having a base width 'B', radius of corner equal to the depth of flow 'D' and the angle 'θ', subtended at the water surface by the corner is equal to the angle made by the water surface and side slope of the channel. As the lining only rests on the side slopes it can't resist the earth pressure. Hence it is necessary to select the side slope equal to or less than the angle of repose of the natural soil. Table. 5.3 gives the suitable side slopes for channels excavated through different types of materials [1].

TABLE 5.3: Nature of Soil and Suitable Side Slope of Channel [1]

SL.NO.	TYPE OF MATERIAL	SIDE SLOPES (H:V)	ANGLE
1	Loose , sandy soil	2:1	26.56°
2	Firm clay	1.5:1	33.69°
3	Earth with stone lining	1:1	45°
4	Stiff clay with concrete lining	0.5 : 1 TO 1 : 1	63.43° TO 45°
5	Muck and peat	0.25:1	75.96°
6	Rock	0.125:1 TO NEARLY VERTICAL	82.87° TO 90°

The geometrical properties of the round corner trapezoidal section are given below

$$A = BD+\pi D^2(2\theta/2\pi)+(2/2)D^2\cot\theta .$$

$$= BD+D^2(\theta+\cot\theta) \quad (5.3)$$

$$P = B+2D(\theta+\cot\theta) \quad (5.4)$$

where, A = area of cross-section.

P = wetted perimeter.

For design of trapezoidal lined channel, following data should be collected or assumed and steps below are followed [14], 40], [46].

(i)	Bed slope (S),	(ii)	side slope, or the angle (θ)
(iii)	Rugosity coefficient 'n'	(iv)	Limiting velocity.

Following steps are adopted while designing a trapezoidal lined channel.

1. Determine the area of flow i.e $A= \dfrac{Q}{V}$
2. Determine the hydraulic radius, i.e $R=\left(\dfrac{Vn}{S^{1/2}}\right)^{3/2}$
3. Determine the wetted perimeter P = A/R

4. Determine the values of B and D from the computed values of A and P by utilising the geometrical properties given above for the section [equation 5.3 and equation 5. 4]

For practical consideration as per S. L. Atmapooja and R. N. Ingle [5] if the radius of rounding is less than the depth of flow then obtained the most efficient channel section. According to him the round cornered sections can be classified into two categories such as:

(i) The section with constant radius of curvature.

(ii) The section with variable radius of curvature.

The conditions for the most efficient round cornered section are given below for different criteria and types of section.

(a) **The section with constant radius of curvature:** In this case the radius is fixed as it is provided and it will not vary with the depth of flow. The cross section area is calculated as below.

$$A = BD + 2rDZ_1 + D^2Z - 2r^2Z_1 + r^2Z_2$$

$$P = B + 2D(1+Z_2)^{1/2} - 2rZ_1 + 2rZ_2$$

Where A = Cross-sectional area.

b = Central bottom width.

y = Depth of flow.

R = Radius of curvature.

1:Z = Side slope (1V:ZH)

$$Z_1 = \left[(1+Z^2)^{\frac{1}{2}} - Z\right]$$

$$Z_2 = (\pi/180)\tan^{-1}(1/Z) \text{ in radians.}$$

$$= \tan^{-1}(1/Z). \qquad \text{in degree.}$$

For the most efficient section the wetted perimeter should be minimum, hence two case arises, i.e.

(i) Constant side slope and variable depth condition , i.e. by simplifying the equation we have, $\frac{b}{y} = 2Z_1\left[1 - \left(\frac{r}{y}\right)\right]$

(ii) Constant depth of flow with variable slopes, i.e.

$$2Z\sqrt{1+Z^2} = \frac{\left[2\left(\frac{r^2}{y}\right)+y-4r\right]}{\left[\frac{r^2}{y}+y-2r\right]}\left(z^2+1\right)$$

(b) **Section with variable radius of curvature:** If the radius of curvature r is taken as function of y, i.e $r = k\,y$.

Where, k = Constant and value is less than unity.

y = depth of flow.

The area can be written as:

$$A = by+2kD^2Z_1+D^2Z-2k^2D^2Z_1+k^2D^2Z_2$$

$$P = B+2kDZ_2+2D\sqrt{1+Z^2}-2kDZ_1$$

Here also two cases arise for most efficient sections.

i.e (i) Constant side slope and variable depth i.e on simplifying we have

$$\frac{b}{y} = 2\left(\sqrt{1+Z^2}-Z\right)+4k^2\left[Z_1-\left(\frac{Z_2}{2}\right)\right]-6k\left[Z_1-\left(\frac{Z_2}{3}\right)\right]$$

(ii) For constant depth of flow and variable slopes we have

$$2Z\sqrt{1+Z^2} = \left[\frac{\left(2k^2+1-4k\right)}{\left(k^2+1-2k\right)}\right]\left(Z^2+1\right).$$

So for a particular condition the most efficient section can be obtained from the above equations.

5.5 TYPICAL SECTION AS PER CBIP RECOMMENDATIONS

The typical cross-sections of canal for discharge s upto 12.5 cumecs is shown in Fig. 5.1 and Fig. 5.3 [11]. The canal can be lined with LDPE film and with single tile cover, where it traverses through porous strata. Fig. 5.3 shows a general layout of single tile lined power channel with LDPE film. The tile can be constructed with 1:6 soil cement ratio with dimension as 30 × 15 × 5. The LDPE film may be adopted where canal passes through sandy soils without any stones or irregular projections.

The power channel should be confined within a formation cutting of 10 m. This formation width includes the construction of canal and service road. Table 5.5 indicates the dimensions of the water conductor system for discharge of 1.0 to 12.5 cumecs.

TABLE 5.4:Typical Dimensions Of Trapezoidal Sections For Different Discharges[11].

SL.No	DISCHARGE (m^3/sec)	WIDTH (m)	DEPTH (m)
1	UPTO 1	700	700
2	1.0 TO 5	1500	1500
3	5 TO 7.5	1950	1950
4	7.5 TO 12.5	2350	2350

A free board, more than the maximum height of surge should be provided above the water surface to prevent the possible over topping of water during sudden closer of turbine and opening of intake gate. According to E.Feifel [32], for gradual closer, if the time required to close the terminal discharge mechanism is shorter than that required for the first wave to travel twice the length of the channel. The maximum wave crest is

$$\Delta h_{max} = \frac{V^2}{4g} + V\sqrt{\frac{h}{g}} = \frac{k}{2} + V\sqrt{\frac{h}{g}}$$

Where V = mean velocity of flow.

g = 9.81

h = hydraulic mean depth = area of flow / top width of water surface $= \frac{A}{b}$

$\frac{V^2}{2g}$ = velocity head = k

Similarly for sudden closer

$$\Delta h_{max} = V^2/2g + [(V^2/2g)^{1/2} + 2(V^2/2g)h]^{1/2}$$

IS 7112-1973 recommended a minimum free board of 0.5m for channels with discharge less than 10 cumecs and 0.75m for channels with discharge greater than 10 cumecs.

5.6 TYPES OF LINING IN OPEN CHANNEL

The lining is provided in open channel to check the transmission losses. The suitable types of linings adequate for different sizes of canals are given below for general guidance [7].

a) Channels with bed width upto 3.0m
   - i) Single burnt clay tile or brick lining where seepage considerations are important
   - ii) P.C.C slab lining; and
   - iii) Flexible membrane lining with adequate earth /tile cover

b) Channels with bed width 3.0 to 8.0m
   - i) Lining of single burnt clay tile
   - ii) P.C.C slab lining
   - iii) Combination lining (flexible membrane lining in the bed and rigid lining on the sides).This may be adopted where the channels have become stable and no danger of scour is expected.

c) Channels with bed width greater than 8.0m.
   - i) In situ cement concrete lining in bed and sides in accordance with IS 3873 – 1978
   - ii) In situ cement concrete lining in bed and P.C.C slab lining on sides
   - iii) Burnt clay tile lining in accordance with IS 3872 – 1966 where aggregates for manufacture of concrete are not available economically.

5.7 CURVES IN ALIGNMENT OF OPEN CHANNEL

As far as possible the open channel should be aligned straight. If the curves are unavoidable, it should be quite gentle/simple circular curves. The greater the discharge, the larger the radius of curve. The radii of curves should be usually 3 to 7 times the water surface width subject to the minimum given in Table.5.4. On lined channels where the mentioned radii may not be provided proper super-elevation shall be endowed with.

TABLE 5.5: Radius of Curves [7]

SL.NO	DISCHARGE (cumecs)	LINED CANALS RADIUS (MINIMUM) (m)
1	280 and above	900
2	Less than 280 to 200	750
3	Less than 200 to 140	600
4	Less than 140 to 70	450
5	Less than 70 to 40	300
6	Less than 40 to 10	200
7	Less than 10 to 3	150
8	Less than 3 to 0.3	100
9	Less than 0.3	50

5.8 DESIGN CONSIDERATION OF CLOSED CONDUIT

Open channels are not recommended in steep hillsides, as the channel will fill up with debris and sediment from surface erosion. Crossing of creeks and gullies carrying storm water is costly and there is no flexibility in vertical alignment of the channel. In these cases the closed conduits in the form of hume pipes are best suited and could be buried in a cut and cover trench along the hill slopes.

A closed headrace conduit may be of unpressurised and low pressurised. Conduit located on the level of the hydraulic grade line will be unpressurised with a free water surface within the pipe. This layout lacks flexibility with respect to vertical alignment and a forebay connecting the headrace and the penstock is needed as per an open channel headrace. Low pressure closed conduit for water conductor system has following advantages than unpressurised one

(a) Flexibility in vertical alignment to avoid the most difficult terrain with respect to construction in general, slides or washout hazards.

(b) Smaller size of pipe as the headrace pipe will be designed according to minimum cost criteria and the velocity in the range 2 to 4 m/sec

(c) Head loss will be limited to pipe friction losses only

Low pressure closed conduit require a surge tank when L/H ratio exceeds 5 to 10.While aligning in the uphill grades less than 1:10 the flow will not create any siltation problems. However, when eliminating the forebay, provision for flushing should be provided at a safe spot, either towards the lower end of the headrace conduit or along the penstock. The low-pressure conduit must be placed on a steep hillside, which implies the side hill cuttings. If the inherent stability of the hillside is low as in the most Himalayan region, the cut will represent costly excavation and support works. It is therefore important to access the topographical and geological conditions of the hillside at an early planning stage.

5.9 EXCAVATION AND PLACING OF HEADRACE CONDUIT

The maximum slope of the cut for placing the headrace conduit must be less than the angle of internal friction including a factor of safety; otherwise the slope will be unstable [16]. Fig. 5.4 shows a natural hill slope with a probable excavation line depending upon the depth of cut. By penetrating different layers of sub-soil or rock, the maximum possible slope will vary and berms at higher levels may be needed. An alternative to excavating to a safe slope angle is to secure the cut by appropriate support work. A permanent sound design must be found for the headrace, as damage and/or washout may lead to extensive maintenance and reconstruction problems. A recommended design is shown in Fig. 5.5 to limit the amount of excavation and support works. The headrace pipe is buried for protection against rock fall or small slides. The cut must be stable during construction and if slides can be expected these should not endangered the foundation of pipe. When the alignment of the headrace is sloping (longitudinally) proper drainage of surface water must be provided in order to restrict the erosion. In case of crossing gullies the recommended design is shown in Fig.5.6. Here the pipe is embedded in concrete so that water, falling rock and trees will pass without affecting the pipe.

5.10 CHOICE OF PIPE FOR HEADRACE CONDUIT

Headrace conduit may be of steel pipe with or without encased concrete, ductile cast iron, concrete hume pipe. Asbestos-cement pipe, glass fibre reinforced epoxy etc. Steel pipe should not be buried due to corrosion problems unless it is suitably protected by bitumen. Steel pipe with thickness of about 3.2 mm encased in concrete and buried underground is best for the steeper slope of terrain (<1:1.5). Reinforced concrete hume pipe also suitable for low-pressure conduit. For all these types of conduit main disadvantages may be higher cost and transportation to the location of their placement. On the other hand the glass-fiber reinforced pipes are very light, transport in difficult terrain is relatively easy and this type of pipe is there fore often recommended. In the case of glass fibre reinforced pipe care should be taken to avoid points of local strain on the body of the pipe wall.

5.11 HEAD RACE TUNNEL- DESIGN CONSIDERATIONS

In small hydros the water conductor system in the form of tunnel is not economically suitable. However the provision of head race rock tunnel, in some cusses is inevitable due to excessive cost of surface alignment. The tunnels can be made as free flowing or under presser, lined or unlined. The following are the different shapes of tunnel [Fig.5.7], adopted on practical, hydraulic and geological reason.

Horse Shoe Section: This type should be adopted when the conduit is partly full and there is a significant external load.

D-Section : Suitable for tunnels located in massive igneous, hard compacted metamorphic and good quality sedimentary rocks where the external pressure due to water or unsound strata upon the lining is slight. Here the wide invert is useful for excavation and mucking operation.

Circular Shape : This section is most suitable from hydraulic and structural considerations.

Egg-Shaped Section: This section may be considered when the rock is stratified, soft and very closely laminated and where rock falls are caused due to high external pressure and tensile stress in rock.

The maximum and minimum diameter of tunnel is generally governed by the structural problem of supporting the roof during and after construction, method of construction employed and above all the relative economics of energy losses vs capital cost. However a minimum height of 2m is desirable for manual excavation and 2.5×2.5m or 3m diameter is about the minimum section for practicable machine excavation.

The governing criteria of hydraulic design of tunnel are as per follows:

(a) If the tunnel is free flowing the hydraulic gradient must be sufficient to give require discharge without causing excessive velocities or losses.

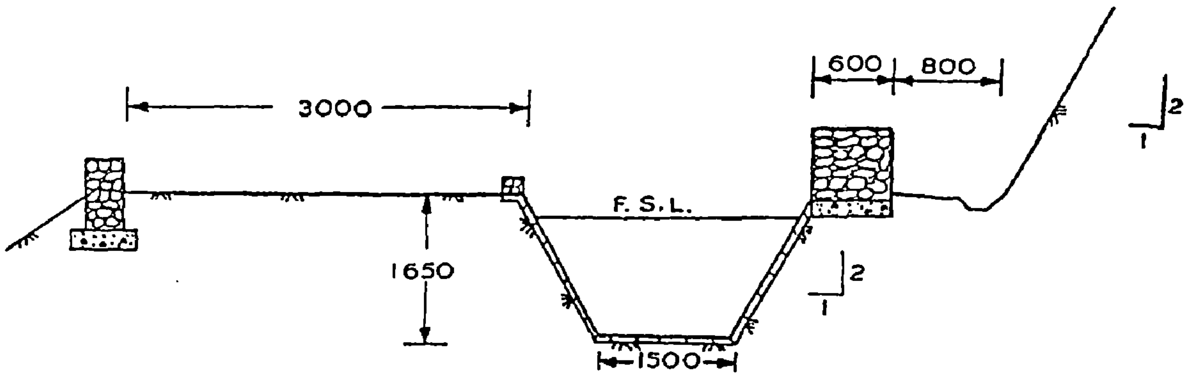
(b) In designing the profile of pressure tunnels care should be exercised that the roof of the tunnel remains in the entire length below the hydraulic gradient by 1.0 to 2.0m. in order to avoid creation of vacuum and consequent possibility of cavitation and possible collapse of lining.

(c) The permissible velocity of water in tunnel with concrete lining should be in the range of 3 to 4 m/s. But in the case of water carrying sharp-edged sand in significant quantities, it should be between 2.0 to 2.5 m/sec.

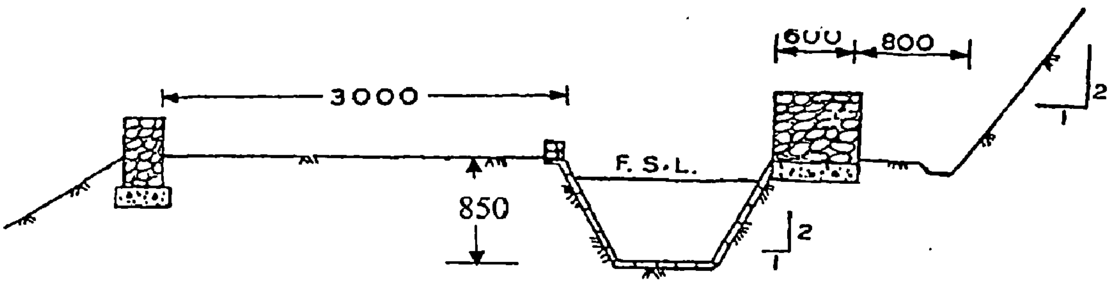
The head race tunnel has to be invariably lined with cement concrete- plain or reinforced or steel lined. The lining has one or more of the following function

(i) To reduce friction losses in the system.
(ii) To prevent the leakage of water.
(iii) Take care of the external forces which are not taken by the support

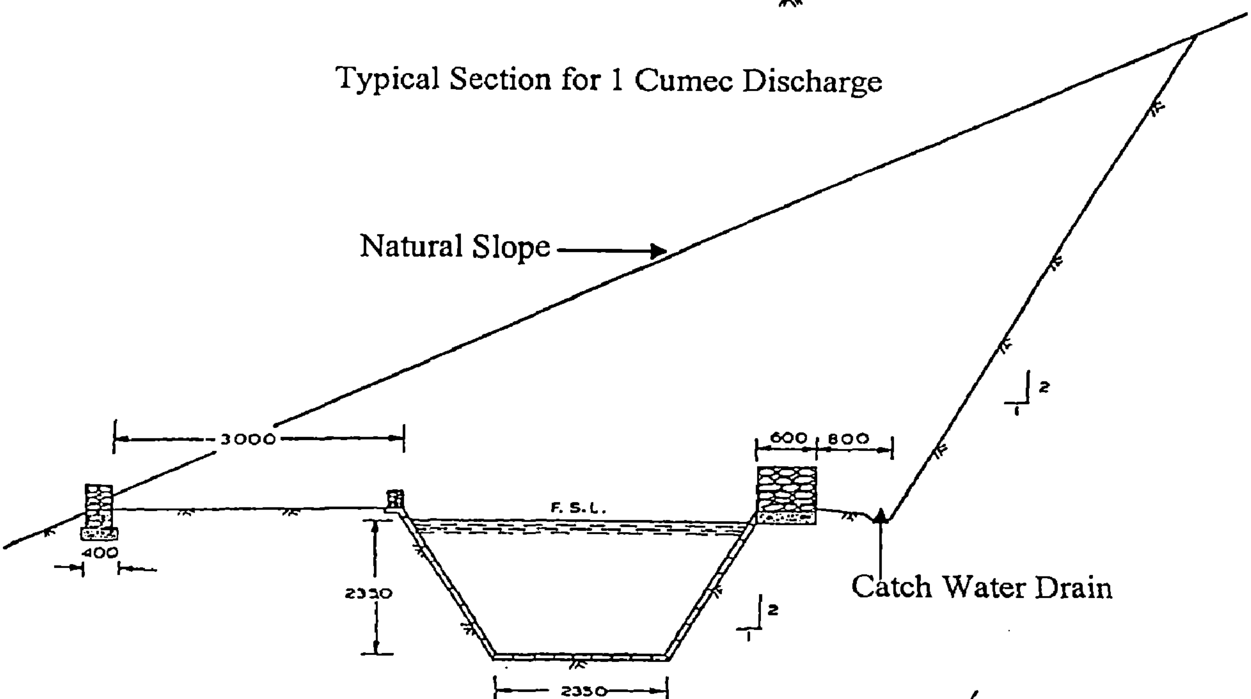
For free flowing tunnel, concrete lining without reinforcement should be provided. The reinforced concrete is generally provided wherever the depth of cover is less than the internal pressure head. In case of heavy seepage or high velocities in the tunnel Steel liner may be provided. The minimum thickness of unreinforced concrete lining of manual placement is 15 cm. Where lining is reinforced, a minimum thickness of 30 cm may be given. As a thumb rule a lining thickness of 6 cm per metre of finished diameter of tunnel should be provided [49].



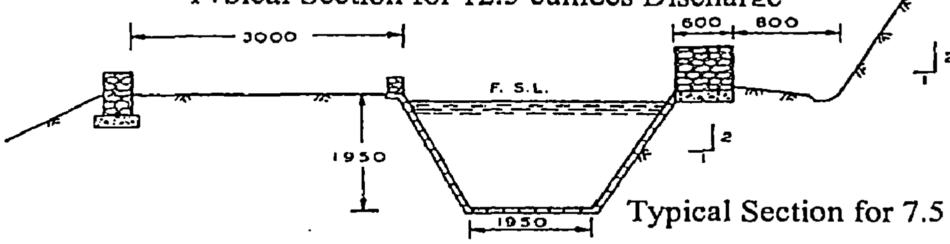
Typical Section for 5 Cumecs Discharge



Typical Section for 1 Cumec Discharge

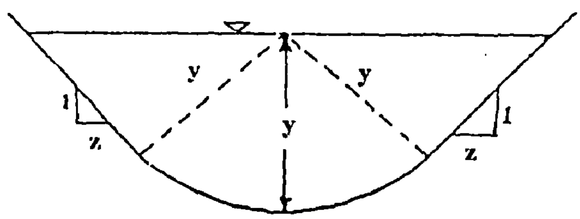


Tvpical Section for 12.5 cumecs Discharge

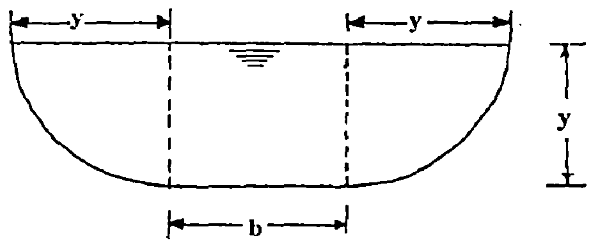


Typical Section for 7.5 Cumecs Discharge

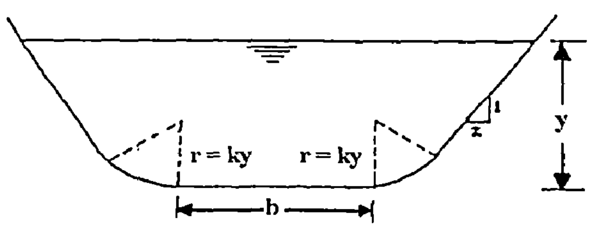
Fig.5.1: TYPICAL CROSS-SECTIONS OF POWER CHANNEL



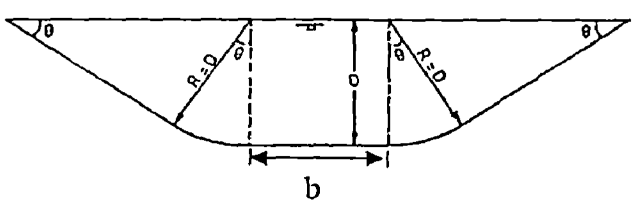
Rounded Bottom Section



Section With Circular Sides

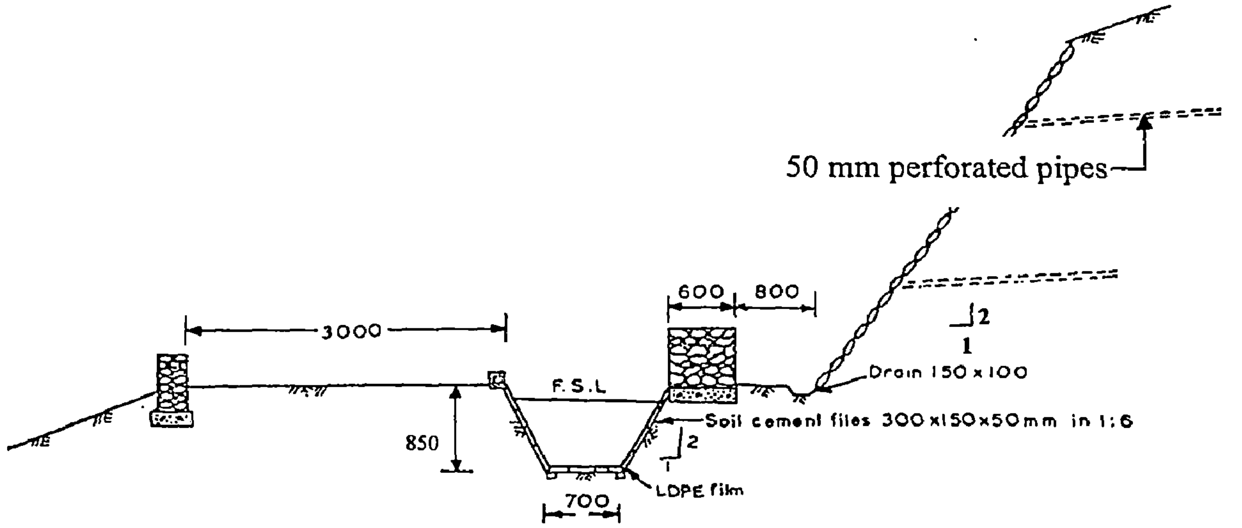


Rounded Corner
Trapezoidal Section



Trapezoidal Round Corner Section with Radius of Curvature Equal to Depth of Flow

Fig.5.2: ROUND CORNERED CHANNEL SECTIONS



Typical Section for 1 cumecs Discharge

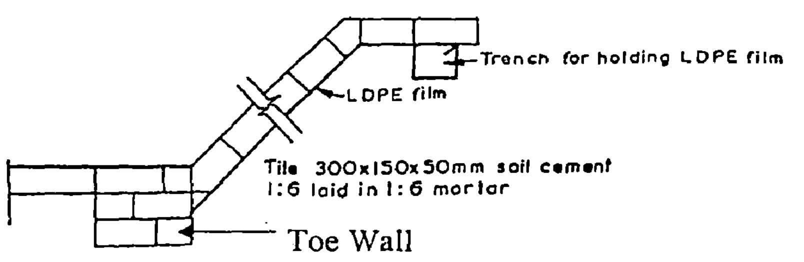


Fig.5.3: DETAILS OF TILE LINING WITH LDPE FILM

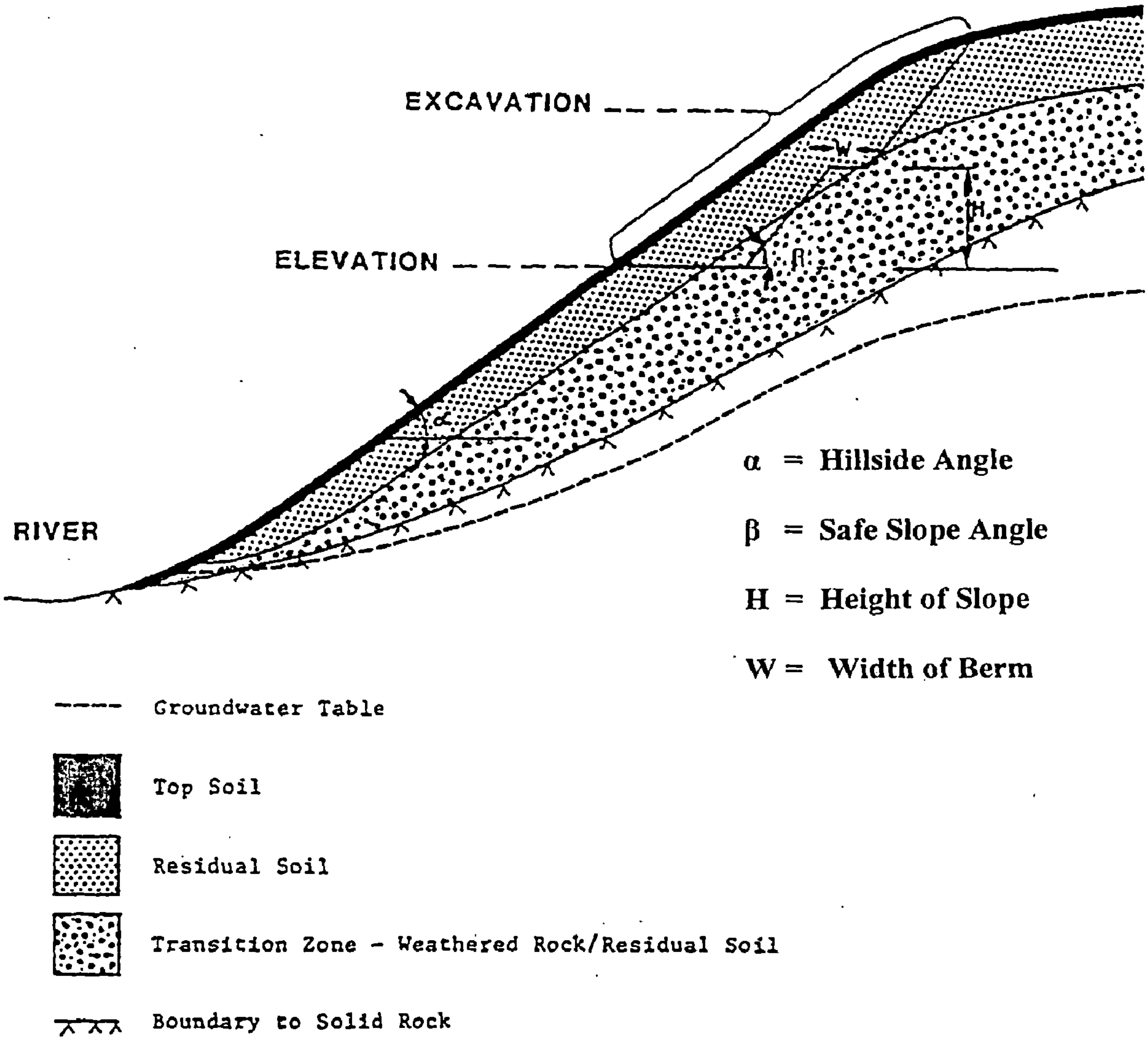


Fig. 5.4 SIDE HILL CUTTING IN WEATHERED ROCK

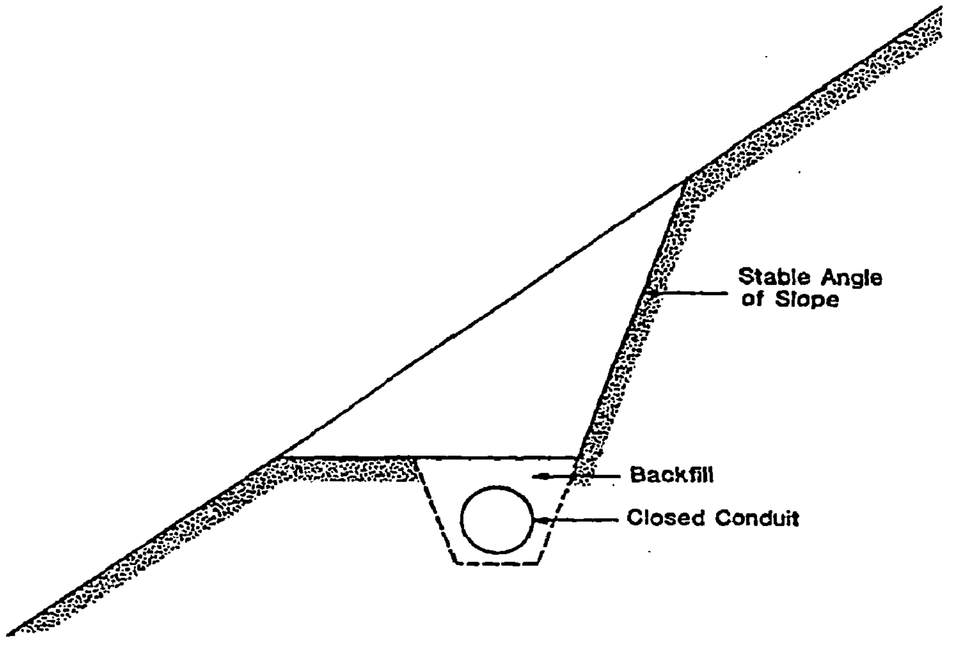


FIG. 5.5 RECOMMENDED DESIGN FOR CLOSED CONDUIT HEADRACE PIPE

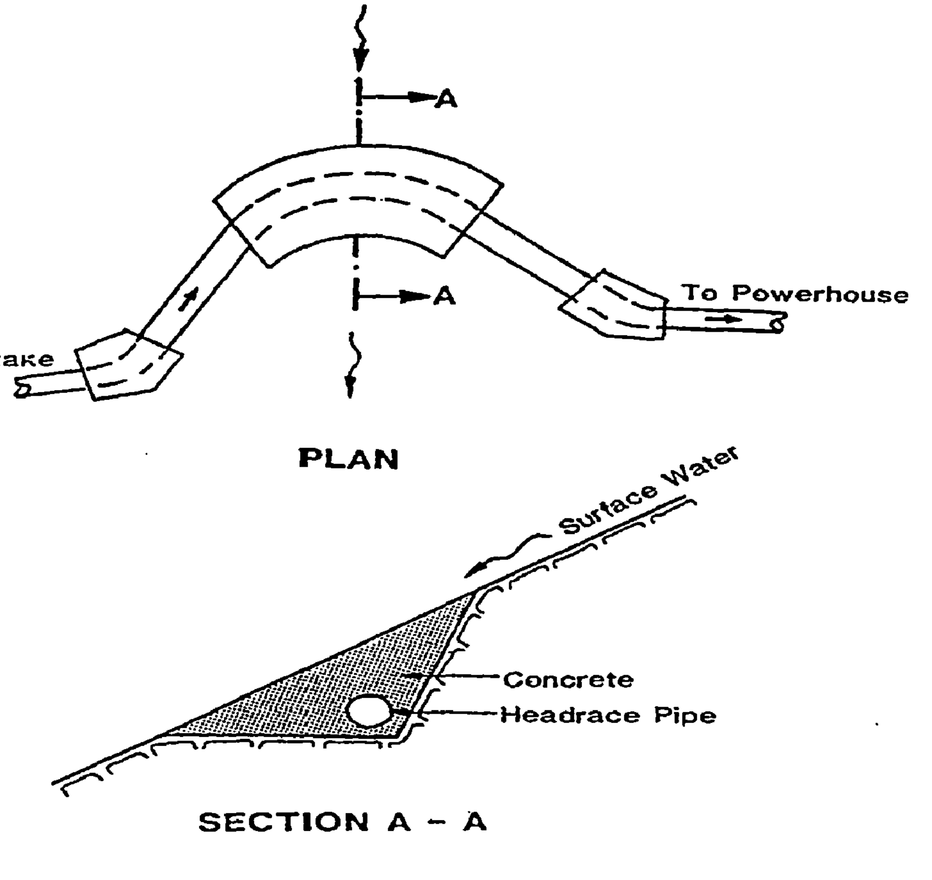
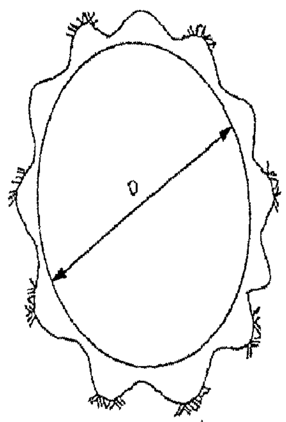
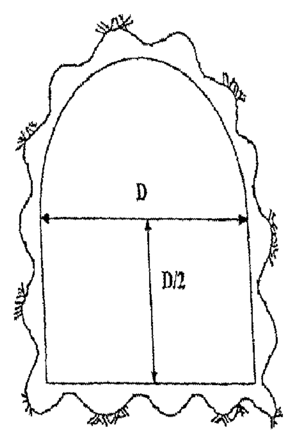


Fig. 5.6 RECOMMENDED DESIGN FOR CROSSING GULLIES



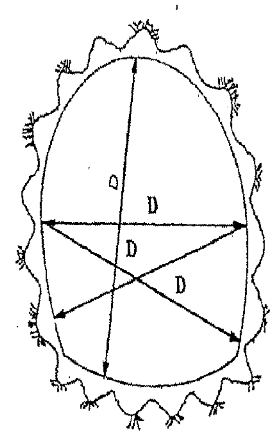
Circular

$A = 0.7854\,D^2$
$P = 3.1416\,D$
$R = \dfrac{A}{P} = 0.25\,D$



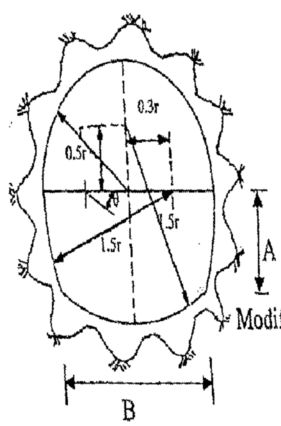
D- Shaped

$A = 0.905\,D^2$
$P = 3.58\,D$
$R = \dfrac{A}{P} = 0.2528\,D$



Horse-Shoe

$A = 0.8293\,D^2$
$P = 3.267\,D$
$R = \dfrac{A}{P} = 0.2538\,D$



Modified HorseShoe

$r = 0.9875\,R'$
R' = Radious of Hydraulically Equivalent Circle
$A = 3.253\,r^2$
$P = 6.426\,r$
$R = 0.506\,r$
$A = 0.78\,r$
$B = 1.56\,r$
$\Theta = 31o22'1''$

Fig.5.7: SHAPE AND GEOMETRIC PROPORTIES OF DIFFERENT TYPES OF TUNNEL SECTION

CHAPTER 6

HYDRAULICS AND DESIGN PARAMETERS OF FOREBAY AND SURGE TANK

6.1 GENERAL

In a run-of-river hydropower scheme, water is conveyed to the turbine from a stream or river through an open channel or closed conduit. The water requirement of turbine changes according to the load requirement. At the sudden load rejection condition the flow of water has to be stopped and if possible stored for a while, to be utilized at sudden load acceptance condition. This immediate water mass requirement at sudden load acceptance and storing of excess water, in the case of load rejection without affecting the water level in the power channel or variation of pressure in the entire headrace conduit, a mechanism in the form of free water surface reservoir is very much essential. This mechanism can be an artificial reservoir called forebay or a water tank called surge tank. In this chapter an effort is made to describe briefly the necessity and design parameter of the above said structures, especially suitable for small hydro schemes.

6.2 FOREBAY

The forebay, located at the end of the headrace in a small hydro power plant represents a small intake reservoir for penstock. It mainly consists of hydraulic structures that control the water level, a device for flushing the forebay and emptying the canal and an intake structure to direct water into the penstock.

6.2.1 Function

The main function of the forebay is outlined below.

(i) It stores some water to act as a regulating reservoir for penstock and provide Immediate water demand on starting the generating units

(ii) It provides enough depth of water over the penstocks to prevent vortex formation and air entry.

(iii) It safely disposes off the excess inflows during load rejection.

(iv) It also acts as a desilting basin/tank and helps to withdraw silt free water for the powerhouse.

Forebay is required only when the powerhouse is located away from the dam or diversion structure and the water is conveyed to the powerhouse through a power canal a free flow water conductor system. It is created by widening the downstream end of the power canal into the form of a small basin having storage for at least 2 minutes. It is always advantageous to combine forebay and balancing reservoir [Fig.6.1] wherever feasible and provide for storage of 1, 2 or 4 hours as may be suitable to the system justifying the economics and requirements for efficient operation of the power plant. The main purpose of balancing reservoir is to store up water during off-peak hours to supply the same during peak hours and thus meet variable demand. Hence wherever a flat terrace or depression is available the creation of a storage pond is advisable.

6.2.2 Design Aspects

The forebay is very often designed with sloping sidewalls and a bottom sloping towards the penstock intake [32],[16]. As some silt will be deposited in the forebay a simple flushing device is provided at the lowest point. A spillway, normally an overflow spillway, is provided to account for the difference in discharge from the intake and the discharge to the power station. A typical layout is shown in the Fig. 6.2.

The location of the forebay is at the top of the most suitable alignment for the penstock. The construction site is often found by cutting into the hillside, and the forebay may have to be built on undisturbed ground.

The damage to the forebay or balancing reservoir very easily leads to a complete failure of the power plant. As the forebay is normally located directly above the powerhouse, uncontrolled flow of seepage water will cause damages to the penstock, the powerhouse and various facilities nearby. The safe design and construction of a forebay involves the following main points.

- The overall stability of the hillside with the pond, filling area with embankment and possible seepage forces must be assured.
- Creation of pondage is preferred in cutting and if done by constructing embankment precaution should be taken so that water may not overtop it in any circumstances.

- For concrete slabs, attention must be given to joints and drainage systems to handle leakage as well as groundwater pressure during dewatering.
- The entire wetted surface should be lined to prevent undue seepage losses and direct waterlogging which could lead to slope instability.
- Surface drainage must be collected and safely discharged to avoid surface erosion, particularly of the freshly deposited fill material. For this purpose catch water drains should be provided on the slope with perforated pipes laid deep in the hills to divert the rainwater.
- The discharge over the spillway and from the sand sluicing facility must be handled in a way that prevents scouring .The spillway capacity should be equal to the design discharge of the channel/duct. A suitable spillway channel should be provided for safely disposal of excess inflows during load rejection. Some times a pipe is one of the choices for conveying the surplus water safely down the steep slope [16].
- The intake arrangement for the penstock should be carefully planed and designed with proper controlling arrangement.
- The stability of the intake retaining wall should be checked with utmost care with any angle.
- The length and breadth of forebay should be as per following considerations, Fig. 6.3.

  B = 1.2 to 1.5 of overall width of penstock arrangement

  L = 2.5 to 3 of overall width of penstock arrangement

  Mean velocity of water inside the forebay should be 0.5 m/sec.
- Depth of forebay should be worked out in relation with the diameter of the penstock pipe.
- The transition between the water conductor channel and forebay should be within 10° to 12° of angle.

Considering all the above design aspects of the forebay the following are the usual components, which require proper hydraulic and structural design consideration.

(i) An overflow concrete spillway with or without falling shutter type gates.

(ii) A screen usually composed of flat steel bars.

(iii) A vertical steel gate or siphon type intake gate for penstock.

(iv) Retaining wall and concrete block for anchoring the penstock and its intake.

(v) A complete lined forebay

(vi) Related embankment to guide the water towards the penstock intake or forebay.

6.3 SURGE TANK

Wherever a completely closed pressurised water conductor system is only feasible it may be necessary to provide a surge tank at the meeting point near the horizontal or gently sloping headrace conduit and steeply sloping penstock. It is designed as a chamber or as a high raised tower. However when the length of the water conductor conduit is less than 5 times the head of the machine, the surge tank may not be necessary. In case of those power stations where impulse turbines are used, the pressure surges arises due to sudden load acceptance or rejection by the machine can be avoided by deflection of the jet. This would eliminate the need of surge tank, though it would involve some wastage of water.

6.3.1 Function of Surge Tank

- It provides a free reservoir surface just upstream of the turbine intake to absorb and compensate water hammer effects. It also reduces the length of the high pressure conduits required to resist the water hammer effect.
- It stores the water at the time of load rejection until the penstock velocity is decelerated to the new steady–state value.
- It supplies extra water to the penstock at the time when the load increases until the penstock velocity is accelerated to the new steady-state conditions.
- It quickly dampens the water level fluctuations following the load changes.

From the above discussions it is well understood that the length of the penstock from the nearest water surface primarily governs the pressure variations in the penstock due to fluctuation of the turbine load. This is why a surge tank with a free surface should be connected to the headrace as close to the turbine as possible. The surge tank helps to obtain smaller changes of water velocity in the headrace than in the penstock, simply by balancing the difference by letting the surface of the tank rise or fall.

6.3.2 Common Types Of Surge Tanks

Surge tanks can be of different types as per the hydraulic design such as:

i) Simple surge tank: This is an unrestricted tank of constant cross-sectional area in which the maximum variation of water level is contained within the tank [Fig. 6.4(a)]. This is very sluggish in action and costly since it requires greatest volume.

ii) Restricted orifice surge tank: This decreases the amplitude of the surge and effects more rapid reduction of mass oscillation. It allows a part of the water hammer pressures into the conduit system. The distinctive feature of this type is a restricted orifice installed between the conduit and the tank [Fig. 6.4(b)].

iii) Differential surge tank: This type has got the combined advantage of a simple and restricted orifice type surge tanks. Here a internal riser pipe extends upward from the restricted orifice to near the top of the outside shell [Fig. 6.4(c)].

iv) Spilling surge tank: This type of tank is similar to simple type but has less capacity to contain the whole of the water during the upsurge and allows the excess water to waste[Fig.6.4(d).

Out of the above four types of surge tank the simple and spill surge tanks are commonly adopted in small hydros on account of its simple construction technique.

For SHP projects involving the use of reaction turbines cost studies may be conducted for the following alternative arrangement for surge requirement and the most economical alternative may be adopted.

(i.) **A vertical surge tank founded over a sound rock surface:** Fig.6.5 shows the typical layout of the vertical surge tank [16]. It consists of a circular reinforced high raised wall to accommodate the rise of water level within the tank, a retaining wall behind it wherever possible to avoid land slide, a proper and structurally stable penstock headrace connection with the surge tank. Hence this type requires costly structural arrangements.

(ii.) **An inclined pipe laid on the hill side slopes:** An inclined hume pipe laid on the hill side slopes with or without a suitable tank at a suitable height extending over the intake level could serve as a surge chamber [11]. A provision of surplus water spill channel may be provided at suitable height to reduce the cost. Fig.6.6 shows the layout of inclined pipe type surge tank.

(iii.) **Water wasting relief valves:** Fig. 6.7 shows the layout of water wasting relief valves.

(iv.) **Air vessels:** Fig. 6.8 indicates the layout of air vessel.

6.3.3 Functioning Of Surge Tank

Whenever there is a sudden increase of the turbine load it will require a similar sudden increase of the flow of water inside the penstock. The stream of water in the headrace represents, however, a considerable mass, which requires sufficient time for acceleration. This is why the sudden increase of flow is supplied from the surge tank itself. This makes the surface in the tank sink, thus increasing the net head between the intake and the surge tank and hence accelerating the flow of water in the headrace. At the new load situation, with increased flow of water, the friction loss in the headrace increases and the new steady state surface position in the surge tank will be lowered.

Because of the inertia forces, however, the surface will at first oscillate down, and then up past the new steady state position thus retarding the flowing water. The friction of the system damps these oscillations about the new steady state level in the tank and their amplitudes are relatively reduced quickly [Fig.6.9].

A sudden reduction of the turbine load, and hence the flow of water, will initiate a similar oscillation starting with an upsurge because the water in the headrace is only slowly retarded, thus filling the surge tank. After a load reduction, the new steady state surface level in the tank will be higher because of less friction loss in the headrace[Fig6.9].

6.3.4 Hydraulic Design Aspect Of A Surge Tank

The surge tank must be so dimensioned that it can take the passing difference in inflowing and outflowing water and limit the water hammer effects. The maximum upsurge and down surge in the tank may be estimated by simple formulae [16], [32],[35],[49] giving very good approximations of the maximum surface oscillations. The expressions are based upon the assumptions that the variation of turbine load takes place instantaneously. This is true if the closing / opening time of the turbine or the valve is shorter than one fourth (1/4) of the headrace periodic time 'T'.

So, Up surge : $h_{up} = Z - \frac{2}{3} h_f$.

Down surge : $h_{down} = Z + \frac{1}{6} h_f$.

where, $Z = V\left[\frac{LA_h}{gA_s}\right]^{\frac{1}{2}}$ = upsurge if friction losses are ignored.

$As = \frac{0.75LA_h V^2}{gH_n h_f}$ = Thoma area of surge tank.

$H_n = H - h_f - \frac{V^2}{2g}$.

where, h_{up} = Maximum rise of the water surface level in the surge tank (m) measured from the surface elevation at the intake. (Being the level in the tank at zero flow)

h_{down} = Maximum fall of the water surface level in the surge tank (m) measured from the surface elevation at the intake(Fig.6.5).

V = Water velocity in the headrace at maximum flow (m/Sec).

L = Length of headrace conduit (m).

A_h = Cross-sectional area of the headrace conduit, assuming it as a closedconduit running full in m^2.

As = Area of the surge tank in m^2.

g = Acceleration due to gravity in m/sec^2.

h_f = Friction loss in the headrace conduit at maximum load (m).

H = Gross head of power plant in 'm' = intake level – tailrace level.

The formula for calculation of A_s is based on the assumption that the penstock length from the surge tank to the turbine is relatively short compared to the length of the headrace and that the friction loss in the penstock is low compared with that of headrace conduit.

As per CBIP [11] recommendation the maximum surge level in simple surge tank can be computed by the formula

$$\frac{L}{2g\phi\beta^2 V_o^2} - \frac{Z_m}{\beta V_o^2} - \frac{L}{2g\varphi g^2 V_o^2} e^{-\frac{2g\varphi}{L}\left(Zm + \beta v_0^2\right)} = 0$$

Where, V_0 = Velocity of flow in tunnel corresponding to maximum steady flow upstream of surge tank.

Z_m = Maximum surge level above maximum reservoir level.

β = Coefficient of hydraulic losses, such that $hf = \beta V_1^2$.

V_1 = Velocity of flow in tunnel at any instant upstream of surge tank.

h_f = Total head loss in the headrace tunnel.

Φ = A_s/A_h = Ratio of area of surge tank to that of headrace conduit.

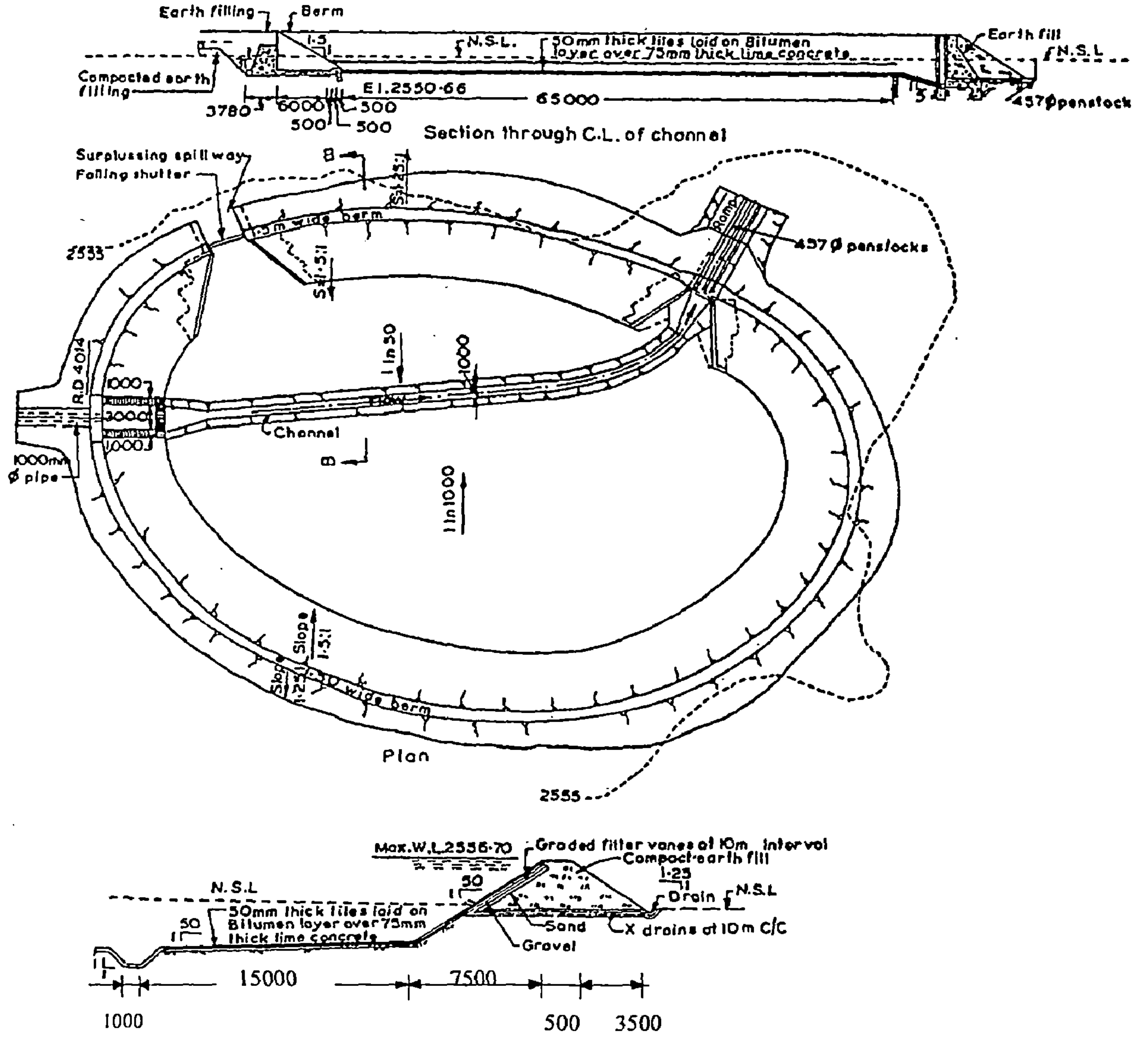
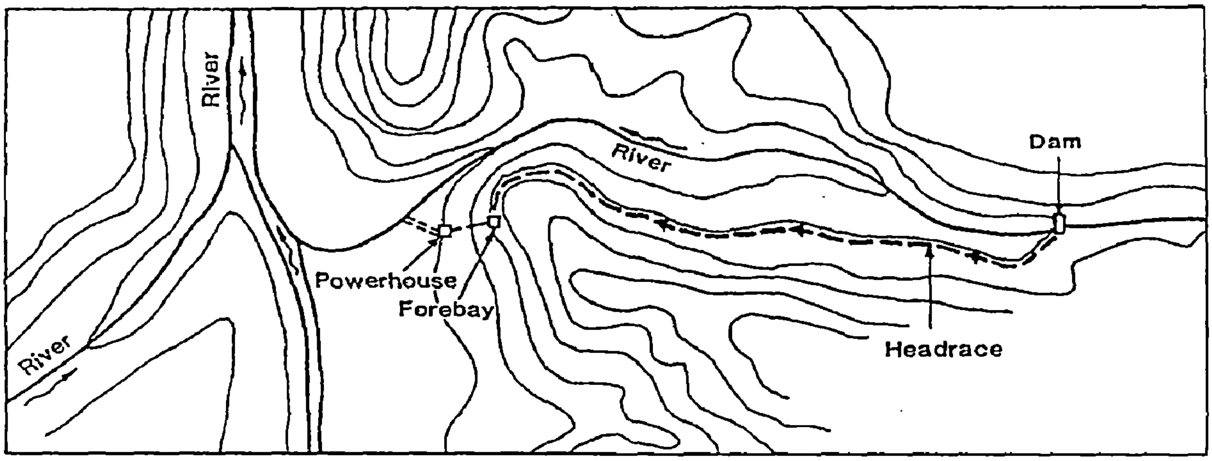
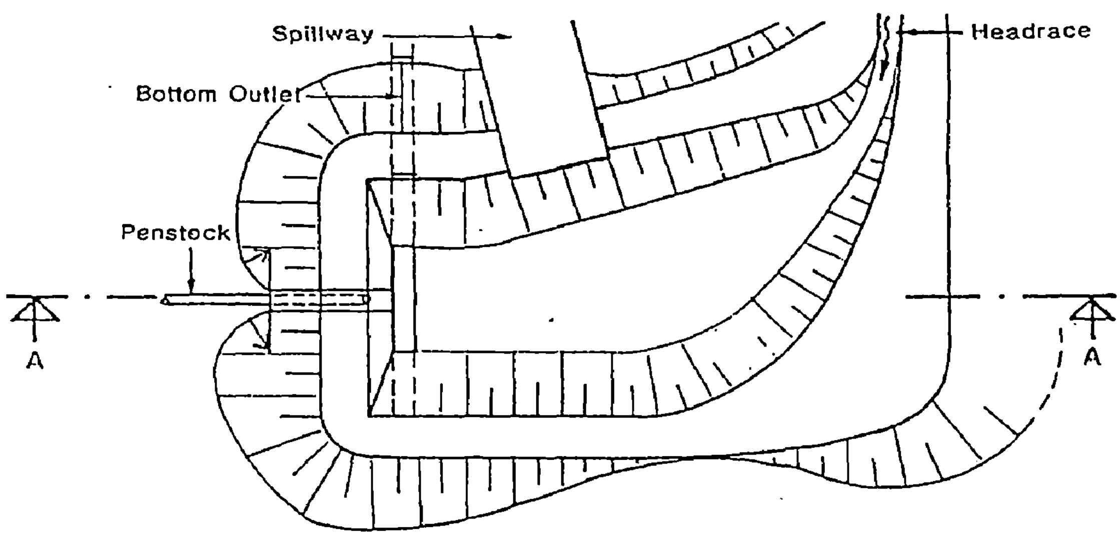


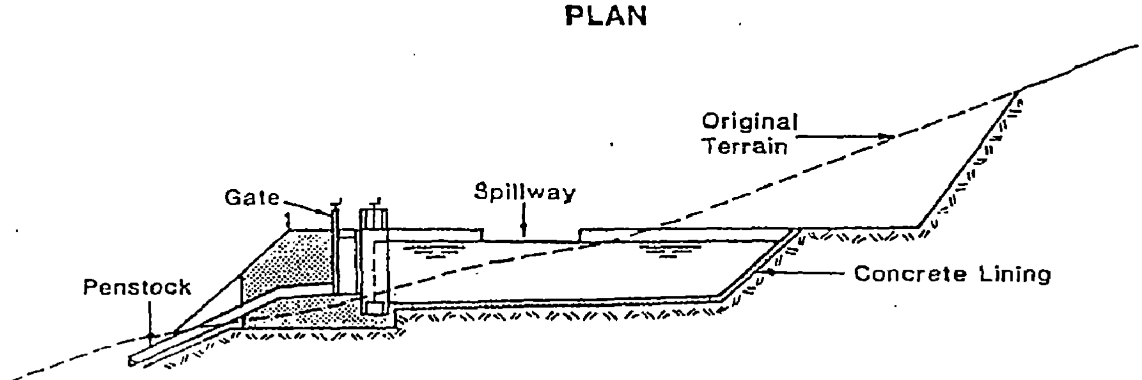
Fig.6.1: TYPICAL LAYOUT OF COMBINED FOREBAY AND BALANCING RESERVOIR



TYPICAL LAYOUT

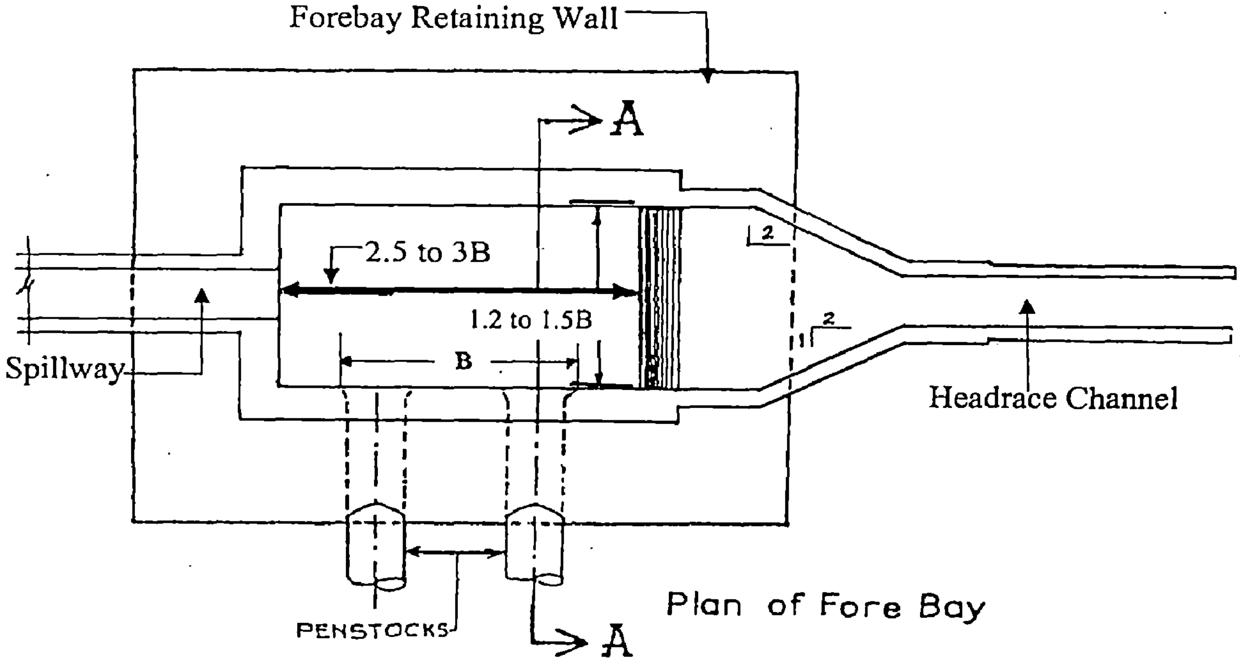


PLAN

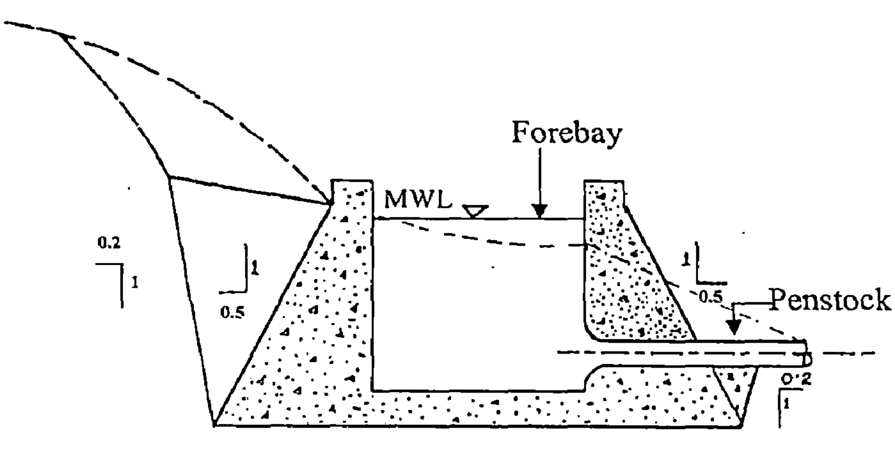


SECTION A - A

Fig. 6.2 TYPICAL LAYOUT OF FOREBAY



Plan of Fore Bay



Section A - A

Fig.6.3: TYPICAL FOREBAY DESIGN

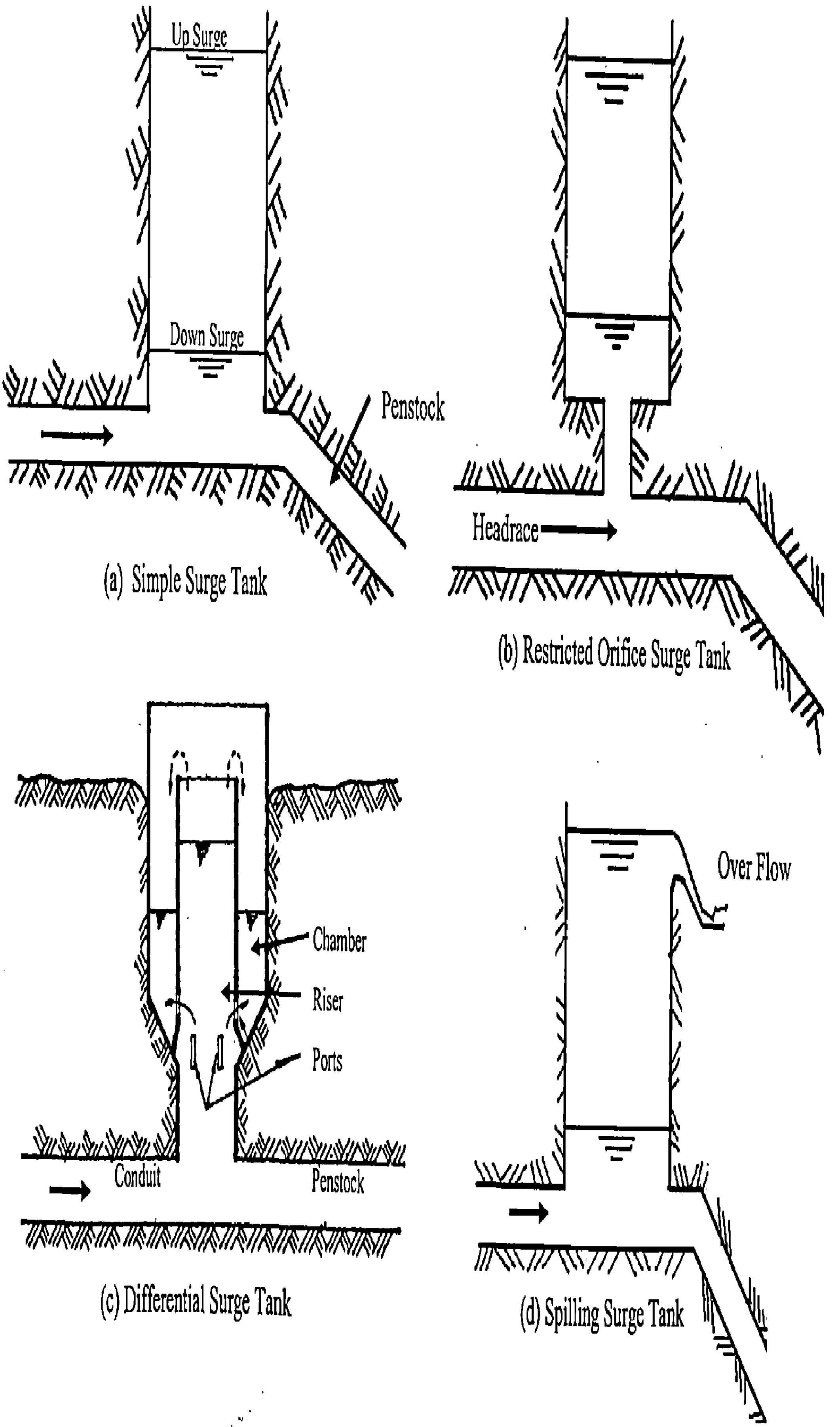


Fig.6.4: TYPES OF SURGE TANKS

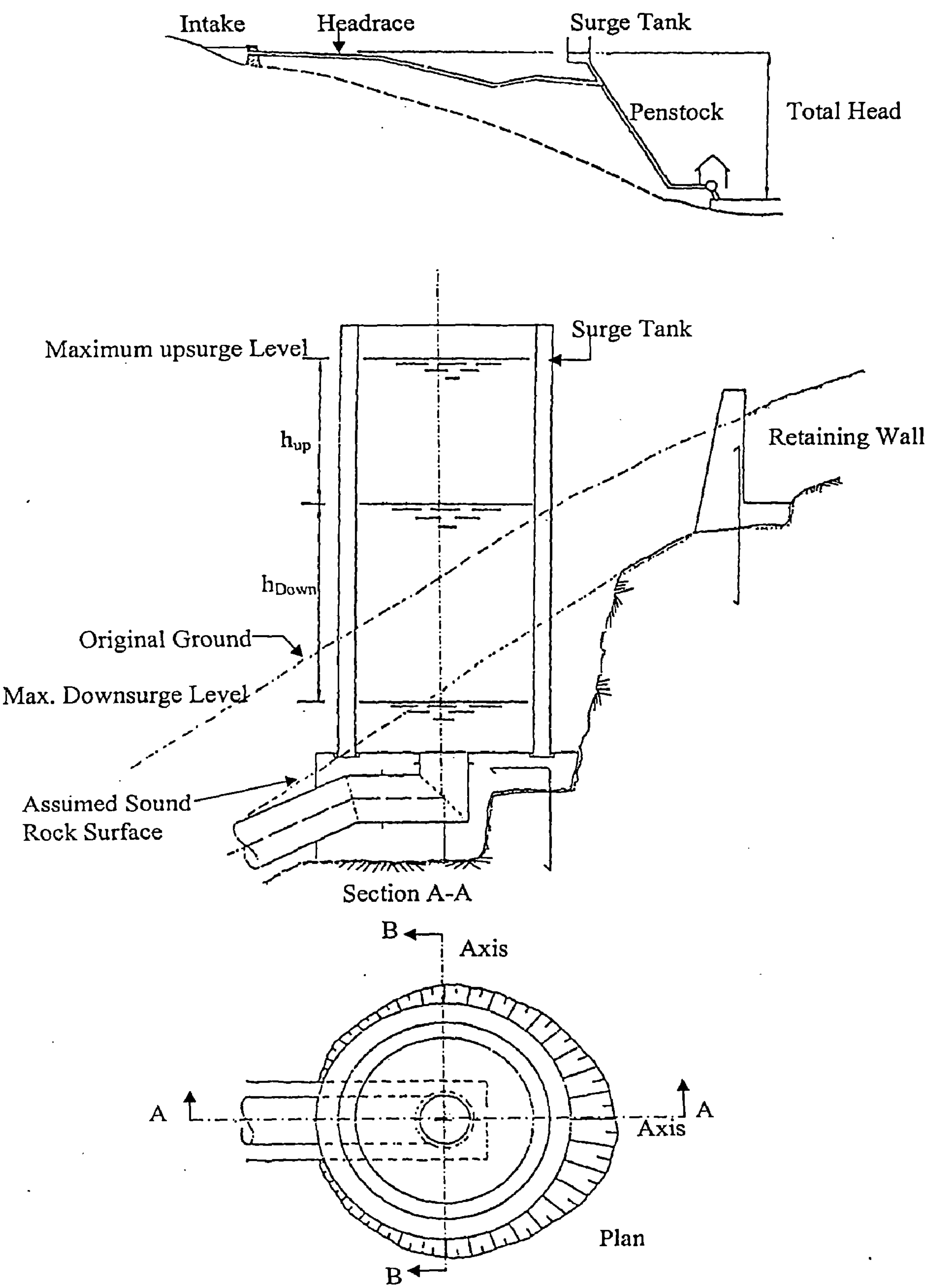
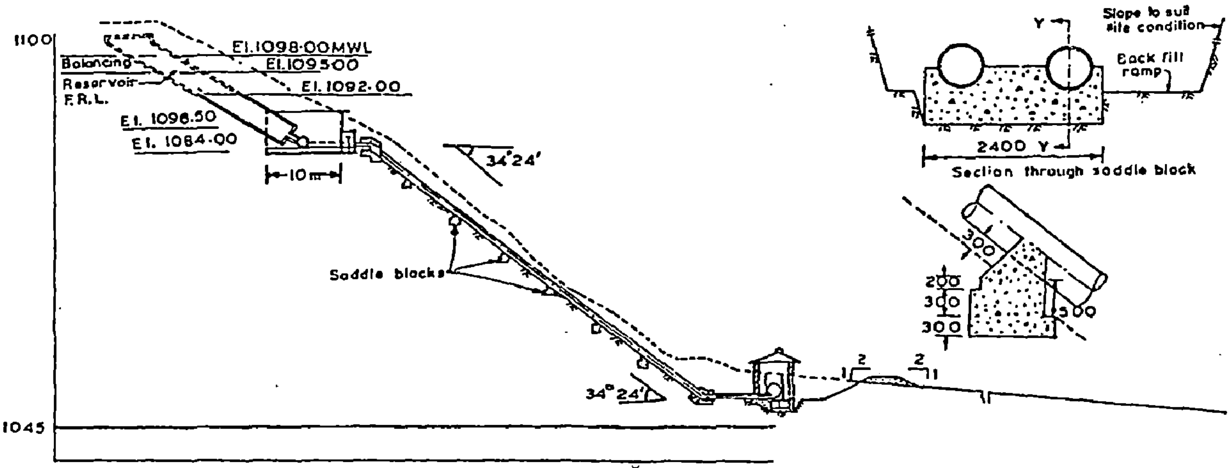


Fig.6.5: TYPICAL LAYOUT OF VERTICAL SURGE TANK



Section of A - A

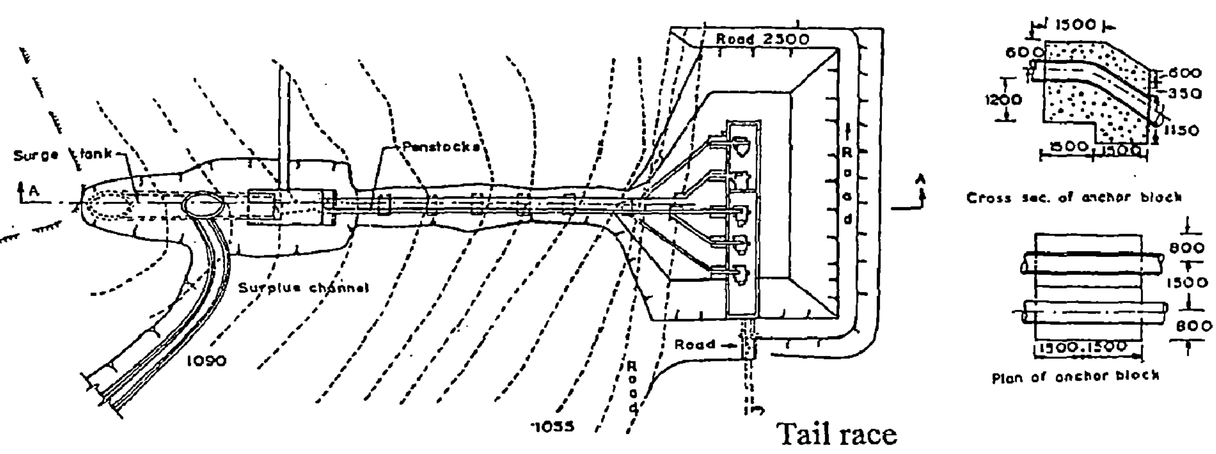


Fig.6.6: TYPICAL LAYOUT OF INCLINED SURGE TANK AND PENSTOCK ANCHORAGE BLOCK DETAILS

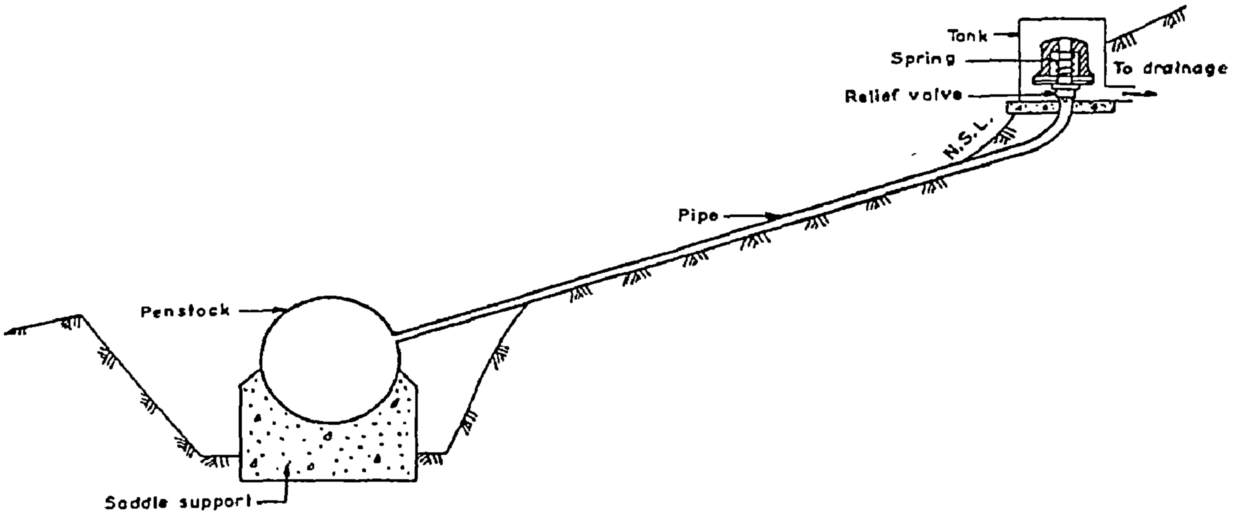


Fig.6.7: LAYOUT OF WATER WASTING RELIEF VALVES

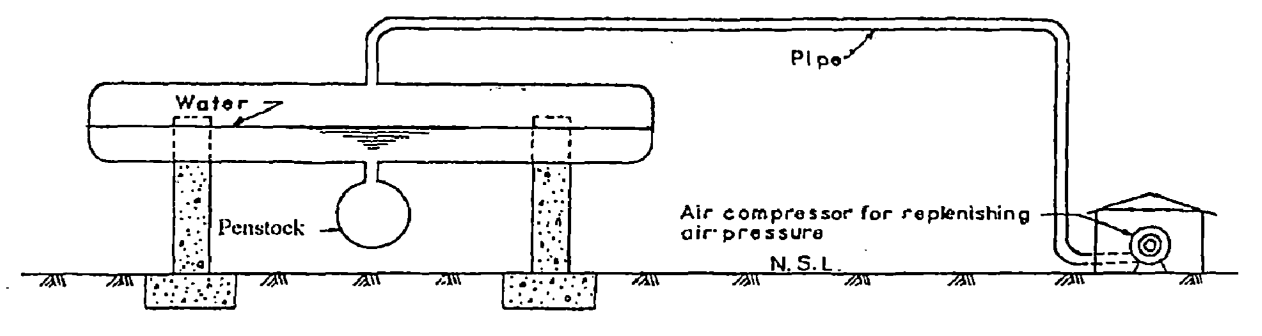


Fig.6.8: LAYOUT OF AIR VESSEL

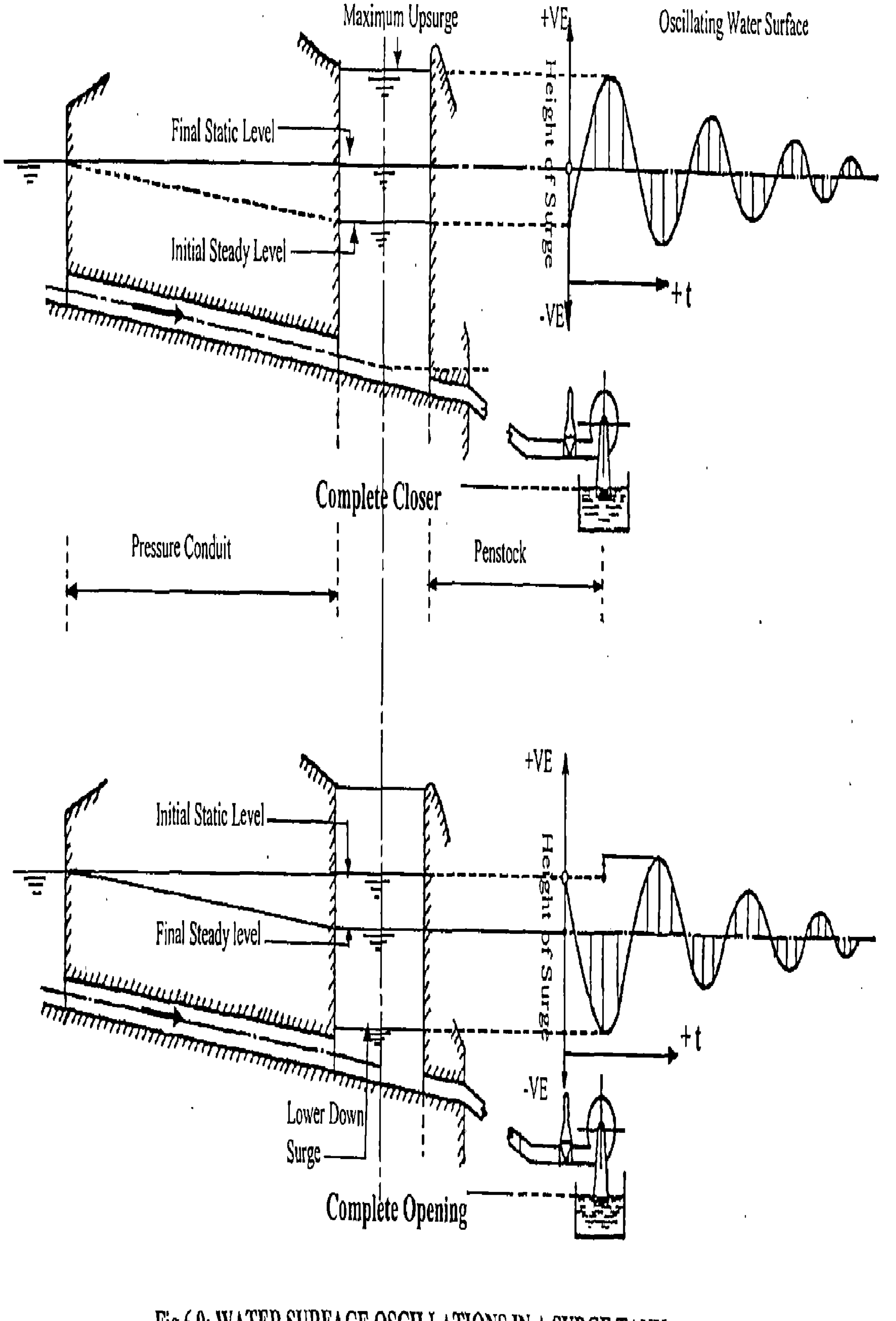


Fig.6.9: WATER SURFACE OSCILLATIONS IN A SURGE TANK

CHAPTER 7

PENSTOCK

7.1 GENERAL

The water is taken from the forebay or surge tank to the turbines through the penstocks. It carries water to the turbines with the least possible loss of head consistent with the overall economy of the project. They are most commonly made of steel, ductile cast iron, glass fiber reinforced epoxy, reinforced concrete hume pipes and EWR pipes. The penstock may be of buried or exposed depending on the foundation and geological conditions.

7.2 PENSTOCK INTAKE

At the penstock intake the bell mouth entry is provided to reduce the head losses and to ensure a smooth entry of water .The intake has a rectangular section and is followed by a transition zone to convert the rectangular to circular shape where from the penstock liner is embedded. Control gate is provided at the throat section with minimum span. Suitable gate grooves are provided as the part of the intake arrangement. A typical section of bell mouth entry and transition in terms of pipe diameter are indicated in Fig. 7.1. The bell mouth and transition is provided as per following equation [49],[35], and [11].

$$\frac{X^2}{D^2}+\frac{Y^2}{0.4286D^2}=1 \qquad (7.1)$$

where, D = diameter of penstock.

X = distance from entry of intake along center line

Y = Height from centerline.

The intakes are provided with trash-racks, which prevent trash, debris, and in cold climate, ice from entry into the waterways. The flow velocity through trashrack is kept at 0.6 to 0.9 m/sec so that head losses are not significant. Trash-rack area closing to an extent of 50 % of total area may be considered in the design without materially decreasing the head on turbine. It may be aligned in a slope to facilitate clearing of debris. The bar spacing should be close enough to restrict the passage of any object that should not pass through the turbine. The rack is designed for an unbalanced head of 1.5 to 2m, which permit some clogging of the screen without causing the structural damage [33], [35]. The trash rack bars should be rounded off to avoid ice formation on the intake screen, which results in clogging of the racks. Deep level intakes should be provided in the cold regions to allow for the necessary cushion for surface ice formation. Some new generation material like HDPE trash racks may be used in place of common steel racks. These have higher hydraulic efficiency than steel racks.

7.3 ALIGNMENT OF PENSTOCK

The penstock should follow a ridge rather than a depression to reduce drainage problems. The vertical alignment can in general follow the slope of the terrain in order to avoid excessive excavation. Sharp bends should be avoided to reduce head losses and the size of supports required to anchor the pipe.

The penstock may be buried or exposed depending on the foundation conditions. If the penstock is exposed, then all supports should, if possible, be founded on rock. If not, sufficient protection against erosion must be provided. The vegetation cover will be damaged or removed during construction; this allows the possibility of extensive erosion especially at depressions sloping towards the penstock. Therefore the vegetation must be replaced as soon as possible after installing the penstock. Fig. 7.2 shows typical general profile of exposed penstock. The pipe rests on Anchor blocks and saddle support with contact angle varying from 90^0 to 120^0. The bearing plate on support is provided with suitable lubrication devices to reduce friction between pipe and supports during contraction/expansion. The anchor blocks should be provided at every change in penstock running direction and at intermediate points to limit penstock length to 100-150 m between consecutive anchors. Suitable expansion joints may be provided to permit elongation/contraction due to temperature variations. A typical arrangement is shown in the Fig. 7.3.

In case of buried penstock the friction between the pipe and the soil will keep the pipe in place provided the slope is stable. Cut-off walls are needed at intervals along the pipe to avoid erosion and /or washout. The buried penstock should be advisable whenever site condition is suitable. Fig. 7.4 shows a typical cross-section of buried penstock.

7.4 CLOSING DEVICES

At the penstock intake it is recommended that an emergency shut off valve be installed combined with an air inlet valve to undertake any repair for penstock. For penstock upto 1000mm diameter sluice valves are better suited. An upstream stop-log should also be provided to repair the valves whenever required. Slide gates could also be adopted along with upstream stop-log arrangements. Similarly an inlet valve just upstream of each turbine is necessary when more than one turbines are operated on one penstock, in order to facilitate maintenance work on one generating unit at a time. Even when only one turbine is installed the inlet valve is usually recommended to avoid draining and refilling of the complete penstock for every stop in production.

7.5 CHOICE OF PIPE FOR PENSTOCK

The penstock can be of Indian hume pipes, ERW pipes, welded steel, ductile cast iron or glass fibre reinforced epoxy pipes for small hydel schemes [27]. Some of the new materials like high-density polyethylene (HDPE), unplasticized polyvinyl chloride (UPVC) or (PVC-U) are also used in worldwide. The new generation pipe materials are cost effective, better hydraulic efficiency, ease in installation and transportation as compared to steel and concrete hume pipe penstock. In India generally the steel pipes are used. The RCC, PSC and other new generation pipe material can be used in India in different site condition. The characteristics data for some of these types of pipes are given in the Table 7.1.

TABLE 7.1: Some Characteristics Data for Various Types of Pipe [29]

Characteristics	Reinforced epoxy	Steel	Ductile iron
Specific weight	1.5	7.9	7.1
E modulus, kg/cm^2	70×10^3	2000×10^3	1700×10^3
Longitudinal expansion coefficient 10^6	20	12	11
Maximum diameter, m	2.0	5-6	1.2
Maximum pressure in m (approximately)	160	1000	400
Friction Coefficient h_f	0.014	0.015 increase with age	0.017
Durability against wear	Good	Good but frequent painting is required	Good
Durability against chemical substances	Good	Low, painting is required	Good
May be buried	Yes	Yes if specially protected	Yes

Expansion joints should be provided on the lower side of every anchor block. The joint should be designed for a temperate range between 0°c and 60°c, [16],[29].

The penstock should be designed for a head corresponding to the maximum water level at the intake plus an allowance for water hammer of at least 25% of the total head, or more if the water hammer analysis indicates higher pressure. Penstocks and headrace pipes must also be dimensioned to withstand full vacuum (-10m) if an absolutely safe aeration arrangement is not made. The common formula used for thickness of penstock is

$$\sigma = \frac{pd}{2t\eta}$$

where, σ = allowable hoop stress (t/m^2).

p = design pressure including water hammer pressure (t/m^2).

d = diameter of pipe (m).

t = thickness of pipe (m).

η = efficiency of longitudinal joint

For steel penstock 1 to 2 mm of plate thickness should be added for corrosion allowance. As a rule for rigidity in transport 6 mm should be the minimum thickness for small diameter pipes and 10 mm for larger ones. The penstock section so obtained should also be pressure tested upto 1.3 to 1.5 times the highest operating pressure.

At least two manholes, one at the upper and another at the lower end, should be provided if the length of the penstock exceeds approximately 50m. The maximum distance between manholes should not be exceeding 150m.

7.6 PIPE SIZE OPTIMISATION

As per CBIP [11] recommendations the table 7.2 shows a possible size of penstock with maximum velocity ranging between 6 to 7 m per sec.

TABLE 7.2: Recommended Penstock Diameter

SL.NO.	DISCHARGE (cumecs)	PENSTOCK DIAMETER (mm)
1	Upto 1	500
2	1 to 2.5	700
3	2.5 to 5	1000
4	5 to 7.5	1200
5	7.5 to 10	1400
6	10 to 12.5	1600

7.6.1 Selection of Diameter of Penstock

In selecting the diameter of penstock for small hydro- power scheme following factors should be considered [32].

(i) Friction loss along the penstock.

(ii) Aberration due to water conveyed.

(iii) Governing conditions of the turbine.

(iv) Cost of the penstock with relation to that of the whole project.

(v) Cost of the turbine governer.

(vi) Technical possibilities of the pipe manufacturer.

The above factors are by no means independent. So the most favorable solution should be arrived at by carefully balancing all the above factors against each other. These are discussed briefly below.

(i) Friction loss along the penstock: A comparison may be obtained by adopting greater penstock diameter with that of lower diameter in terms of energy regained, converted to price and the cost increased due to the adaptation. So the investigation of several diameters provides a satisfactory basis for selecting the reasonable and economical size of the pipe. The following formula can be used for economical size of conduit by taking in account only the variation of friction losses with variation of diameter for the same discharge or the discharge that of the cubic middle[29],[35] .

- $$D \leq \left(\frac{f\sigma k_2 Q_c^3}{1100 k_1 H} T \right)^{\frac{1}{7}} \text{ in m.}$$

- $$Qc = \frac{1}{T}\left(Q_1{}^3 t + Q_2{}^3 t + Q_3{}^3 t + \dots\dots + Q_n{}^3 t\right)^{\frac{1}{3}}$$
- $$n = \frac{T}{t}$$

where, Qc = Equivalent discharge in cumecs.

H = Design head in meter.

σ = Allowable stress for penstock material in kg/cm^2

k1 = Annual cost of penstock per kg.

k2 = Value of one KWh at generator terminals in the same unit.

T = Annual duration of operation in hours == $\sum_{n=1}^{n} t_n$

f = Friction coefficient of the pipe material.

t = Length of each period.

n = Number of period.

(ii) Abrasion due to water conveyed: In order to eliminate the abrasion caused by high-velocity flow of water conveying more or less suspended sediment, a limit velocity depending in part on the pipe material and in part on the amount of sediment transported may be established, with respect to abrasion, permissible velocities for properly settled water are in general between 3 to 5 m/sec. The higher values apply to water carrying finer sediments, while lower values are to water conveying coarser and sharper materials. The diameter of pipe can be obtained from the formula

$$d = 1.13\sqrt{\frac{Q}{V}}$$

where d = Diameter of penstock

Q = Discharge in penstock

V = Water velocity inside penstock

(iii) Governing conditions of the turbine: The time of opening or closing of the governor of the turbine should be larger than the time of acceleration i.e.

$$T_a = \frac{LV}{gH}$$

where, T_a = Time of acceleration (Sec).

L = Length of penstock (m).

V = Velocity in the penstock (m/Sec).

g = Acceleration due to gravity (m/Sec^2).

H = Gross head over turbine (m).

However to some extent the time of opening or closing can be extended by increasing the flywheel effect upto a certain limit without a parallel increase of the cost of the mechanical equipment.

(iv) Cost of penstock: A relative magnitude of penstock costs in comparison with the other investments should be analysed. If the ratio is less, then a greater diameter may be allowed for more efficient utilization of available energy.

(v) Cost of Governer: The cost of governer is inversely proportional to the diameter of penstock. So an analysis in this respect should be made to have an economical arrangement.

(vi) Technical possibilities of pipe manufacturer: The difficulties in manufacturing, transporting, and erecting especially large pipes of great shell thickness has an impact in the adaptation of pipe size. Similarly the available pipe material and manufacturing plant equipment are also defining the greatest possible pipe diameter.

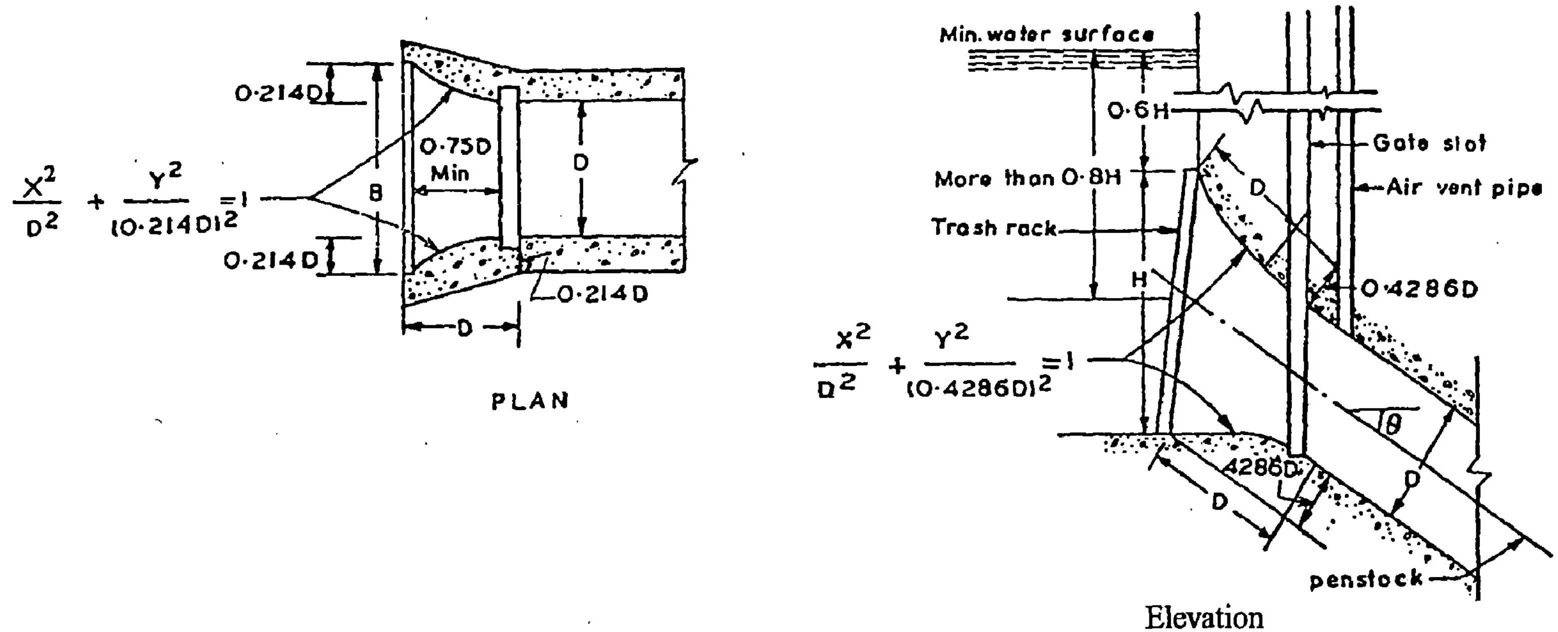


Fig.7.1: TYPICAL SECTION OF BELL MOUTH ENTRY

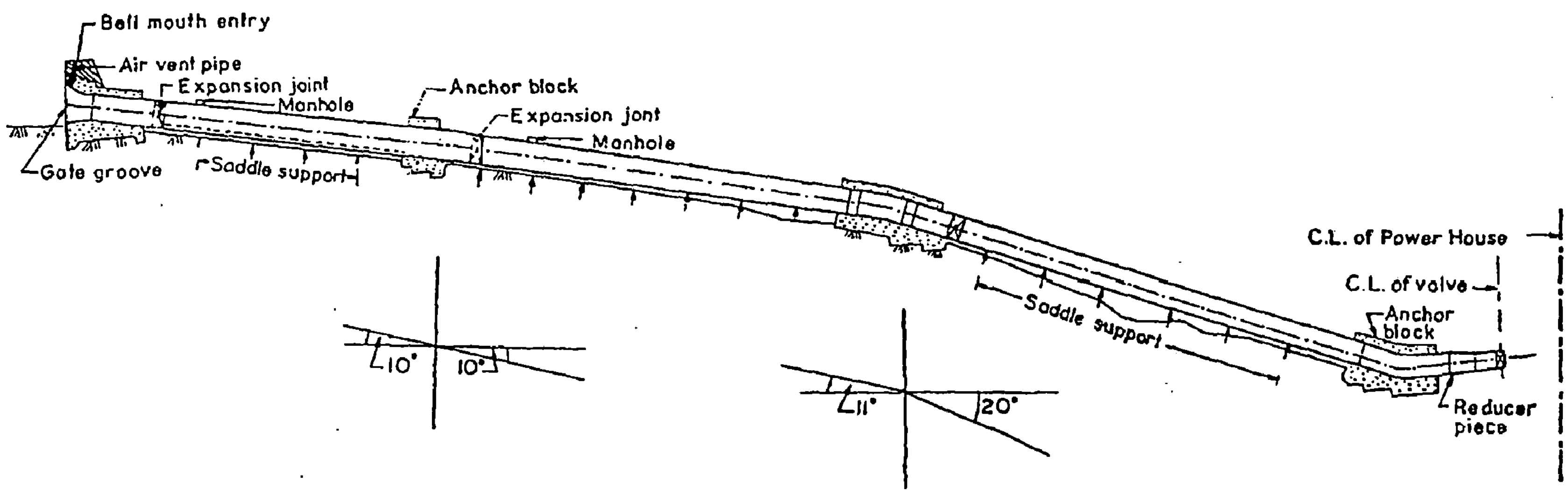
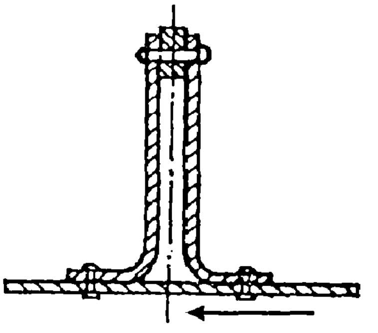
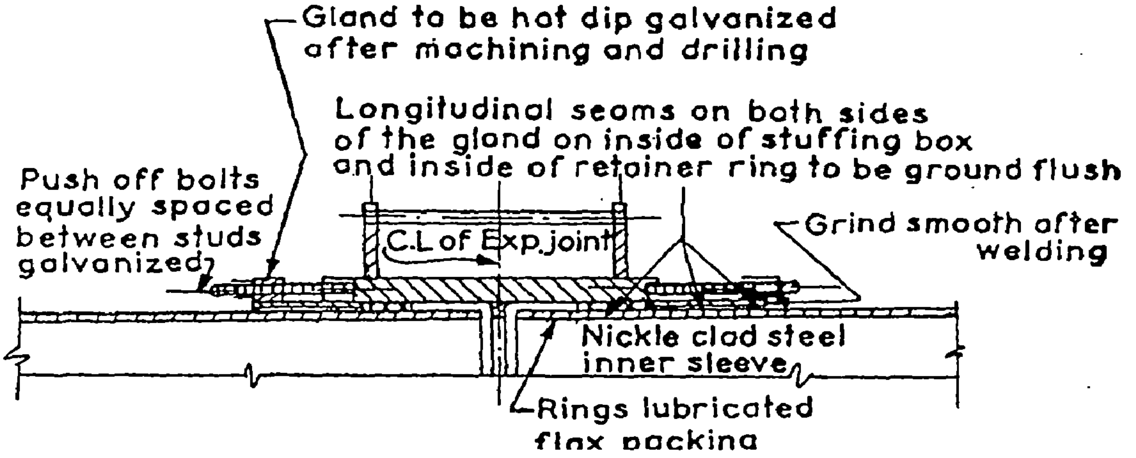


Fig.7.2: LONGITUDINAL SECTION OF EXPOSED PENSTOCK PROFILE


Direction of Flow

Bellow Type Joint



Sleeve Type Expansion Joint

Fig.7.3: TYPICAL EXPANSION JOINTS

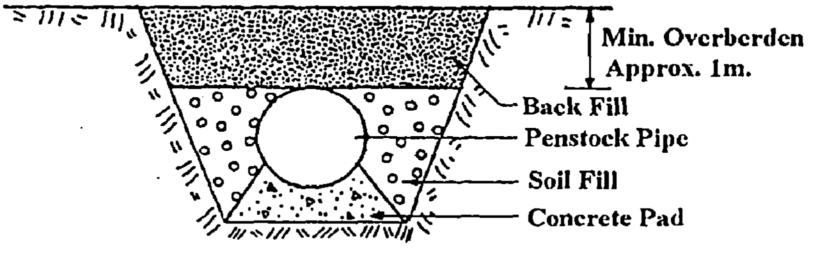


Fig.7.4: TYPICAL CROSS-SECTION OF BURIED PENSTOCK

CHAPTER 8
HYDRAULIC DESIGN OF TAILRACE CHANNEL

8.1 GENERAL

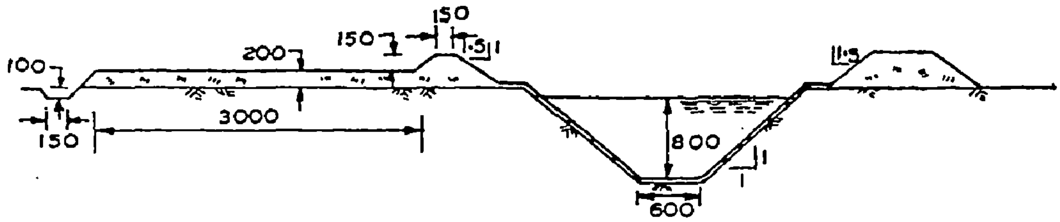
The purpose of the tailrace is to convey the water leaving the power plant back to the river. Tailrace should be designed to maintain the water surface at the elevation specified by the turbine manufacturer and to protect the power plant against flooding by the expected design flood level in the river. In small hydro these are normally accomplished by a weir at the outlet of the power plant with the crest level determined by the expected flood level in the river.

Effort should be made to obtain correct flood level. In many cases the water level versus discharge curve is not available at the stream site. So hydraulic calculations should be made and relied upon, to get the maximum flood level.

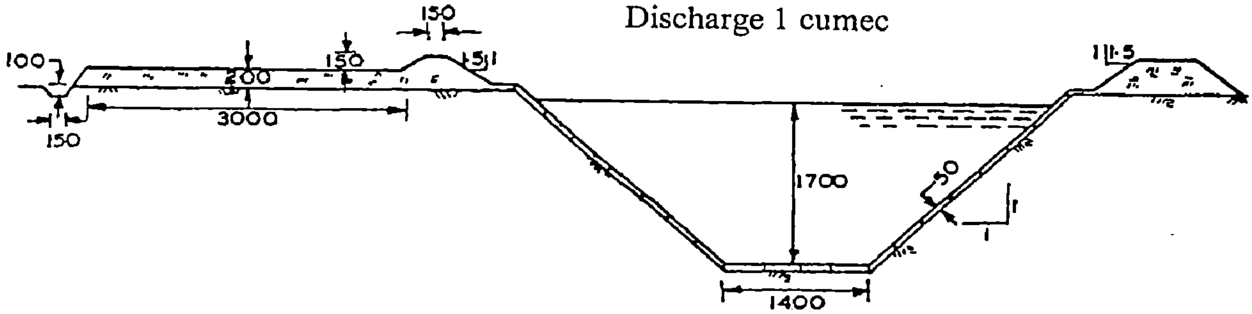
8.2 THE HYDRAULIC DESIGN ASPECT

The design aspect of tailrace is similar to that of headrace channel. The tailrace may be simply a water channel or a cut and cover conduit with an allowable water velocity of 1 m/sec [16].

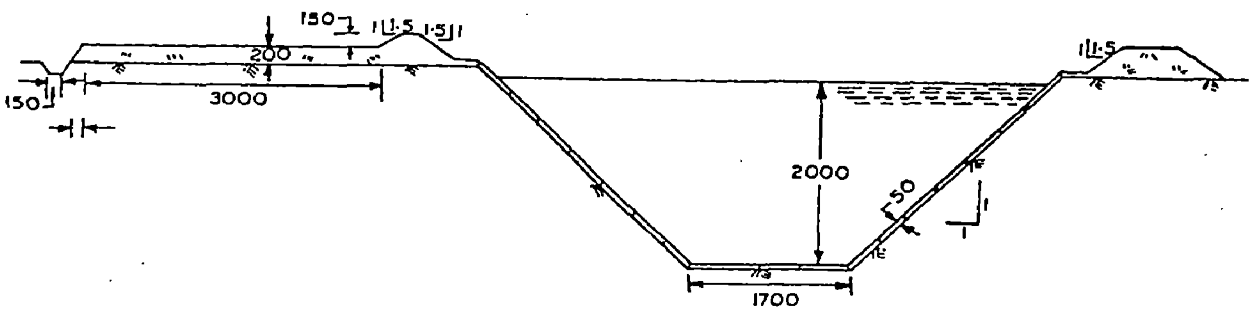
The overflow weir may create relatively high velocities in the tailrace channel and erosion protection will normally be needed. If the base material is sand or silt, then a properly designed energy dissipater at the toe of the weir may be needed. Various dimensions [11] of tailrace in cutting for discharges upto 12.5 cumecs are shown in Fig. 8.1. The canal should join the watercourse at a smaller angle, if possible on the concave side of a bend. It is important to observe that the canal should discharge into the river as close to main current as feasible to avoid the bed formation.



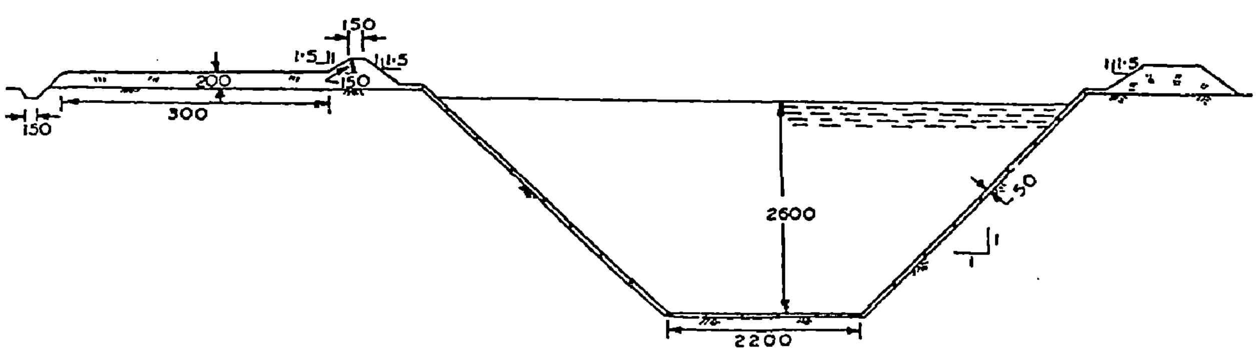
Discharge 1 cumec



Discharge 5 cumecs



Discharge 7.5 cumecs



Discharge 12.5 cumec

FIG.8.1: TYPICAL DETAILS OF TAIL RACE CHANNEL FOR DIFFERENT DISCHARGES

CHAPTER 9
CASE STUDY ON SHP, ASIGANGA STAGE -I

9.1 GENERAL

In this chapter a case study is made on design of hydraulic structures, using the principles and practices of design discussed in earlier chapters appropriate for the proposed SHP on river Asiganga, a tributary of the river Bhagirathi in Uttarkasi District in the state of Uttaranchal.

9.2 THE PROJECT FEATURE

Asiganga stage-I Small hydel project is a run-of the river scheme for power generation on the river Asiganga, a tributary of Bhagirathi. Some of the important project features are discussed below.

9.2.1 Geographical Position

The project is located in district Uttarkasi of Uttaranchal state.The geographical co-ordinates of the project are latitude 30°48'37"N and longitude 78°27'5"E.

9.2.2 Topography and Physiography of the Basin

Asiganga is a perennial snow-fed stream, meeting Bhagirathi river. The altitude of the scheme is about 1360m above NSL. The catchment area of the river at weir site is about 148 sq km. The stream bed slope is steeper in the upper reaches and is about 1:20 (V: H) at the project site.

9.2.3 Geology

The project is situated in a highly seismic area and falls within zone V of seismic map.

9.2.4 Location of the Project Area.

The power channel, forebay and powerhouse are located on the left bank of Asiganga. The proposed diversion site is near Sangamchatti. The scheme will provide power supply to the villages situated within a radius of 5km from the powerhouse location and extra power will be fed to the grid.

9.2.5 Communication Facilities

The nearest railhead is Rishikesh about 174km from the project site. The nearest township, Uttarkasi, an important industrial trade Centre and district headquarter is 11 km from the project site. The project is well accessed by metal road. However a small link/approach is required upto powerhouse location.

9.2.6 Climatic Condition

The area has experienced an average mean minimum temperature of 6°C.The month of January remains as the coldest and May remains the hottest month with average mean temperature of 27°C. There is a deviation of 6° to 8°C from the mean temperature on either side during the day/night. The mean annual rainfall in the project area is 1680 mm.

9.2.7 Hydrology

The discharge measurements of Asiganga have been carried out by U.P.Jal Vidyut Nigam .The analysis of available data indicates that 50% and 75% dependable discharges are 5.63 cumecs and 4.09 cumecs respectively. The lean discharge generally occurs in the month of March. As per historical data available the high flood level is RL.1497.8 m at intake site. The maximum design flood is estimated 594 cumecs.

9.2.8 Construction Materials

The construction materials like coarse aggregate and sand are locally available. The forest comprises of mixed jungles, mainly pine, banjh, burans, surai etc.

9.2.9 Population

The area is sparsely populated with a low literacy rate. Agriculture and cottage industry are the mainstay of the local people.

9.2.10 Socio-Economic Aspect

The socio-economic condition of the people is very poor. They work either on their terraced fields or on the state development projects. Their part time employment is on handicraft goods and woollen khadi dresses. The majority of population is woollen khadi farmers.

The final layout of the project as per the detailed project report prepared by U.P.Jal Vidyut Nigam is shown in Fig.9.1.

9.3 HYDRAULIC DESIGN

The different hydraulic structures that are required for the scheme are as follows.

(a) Diversion weir.

(b) Intake structure on the left bank of the river.

(c) A desilting tank to remove silts and sands particles.

(d) A 2500 m long water conductor system upto the forebay.

(e) A forebay with spilling arrangement.

(f) Penstock about 175m long bifurcating into two leading to the powerhouse.

(g) A tailrace channel.

The civil structure of powerhouse is not considered for this study as it relates to the structural aspect of the powerhouse. The hydraulic design aspects of the above structures for this SHP scheme have been dealt in the present study.

9.3.1 Design of Diversion Weir

Asiganga river is in the hilly region of the Himalayas. The conventional type weir obstructing the stream flow is not considered suitable to this site. Considering the stream characteristics a trench type weir is selected for this scheme. The choice has been made for the following reason.

(i) The Asiganga a tributary of the Bhagirathi in Himalayas carries lots of bed loads and rolling boulders in the rainy season.

(ii) The river is subjected to flash flood during rainy season.

(iii) The scheme is a run-of-river type.

(iv) The bed slope of the river at the site is 1:20 (V:H)

(v) The riverbed is full of fluvial sediments of varying size. Sound rock is not available at reasonable depth.

9.3.1.1 Design Discharge of Trench weir

Assuming a 20% of water is required for flushing of sediment through desilting chamber, discharge required = $1.2 \times 5.63 = 6.756$ m^3/sec.

Take an additional discharge of 15% for flushing through intake well at the end of trench weir. So total water required to be diverted = $1.15 \times 6.756 = 7.7694 \approx 7.8$ m^3/sec.

9.3.1.2 Design of Trench weir

For design of Trench the procedure discussed in chapter 2 should been followed. Let us use 25mmϕ round steel bars with spacing of 30 mm C/C in the trash rack. The trash rack is provided with a slope of 1 in 12 and rack bars are kept parallel to the direction of the stream. The length of the trench weir is taken as 22m (width of river). Now the length of the rack required for complete withdrawal of non monsoon flow through the rack

$$L = \frac{Q_l}{\in \in_c C_d B\sqrt{2gE}}$$

where, $Q_l = 7.8$ m^3/sec

$$\in = \text{Contraction coefficient} = \frac{s}{s+t} = \frac{30}{25+30} = \frac{30}{55} = .545$$

Let us assume a 10 % of reduction for framework.

Hence $\in = .545 \times .9 = .49$

$\in_c = 0.5$ (Assuming 50% clogging of trash rack with pebbles and trash carried by the stream)

B = length of trench weir = 22m.

g = 9.81

E = Specific energy of stream over the rack

$$= \left[\frac{Q_l}{C_d' B}\right]^{2/3}$$

For a broad crested weir, coefficient of discharge C_d' is taken as 1.5 (1.46 to 1.64)

So $$E = \left[\frac{7.8}{1.5 \times 22}\right]^{.67}$$

$$= .491$$

Now $$L = \frac{7.8}{.49 \times 0.5 \times 0.47 \times 22\sqrt{2 \times 9.81 \times .491}}$$

$$= .992\text{m say } 1.0 \text{ m.}$$

In the above analysis a uniform intensity of discharge for entire length of waterway has been assumed. For streams with less discharge and considerable width, it is desirable to be liberal in providing the rack width to ensure complete withdrawal as there could be considerable variation in intensity of discharge along the length of the rack i.e. width of the stream or river. Hence as per CBIP standard a width of 3m is recommended for these discharges [Fig.9.2].

(b) **Depth of Trench and Water surface profile**

Assuming the shape of the trench to be rectangular, the depth of trench can be worked out as per following data.

Top width of trench, T = 3m

Design discharge, Q = 7.8 cumec

Depth of flow at any section inside the trench, d = to be calculated.

Taking water surface profile as straight, n = 0.5

Hence by using the equation 3.16 and 3.17 the velocity head

$$h_v = \frac{n}{n+1}\left(\frac{A}{2T}\right)$$

$$= \frac{0.5}{0.5+1} \times \frac{3 \times d}{2 \times 3} = \frac{d}{6}$$

Again $\quad Q = A \times V = A\sqrt{2gh_v}$

$$\Rightarrow \quad 7.8 = 3 \times d\sqrt{2 \times 9.81 \times d/6}$$

$$\therefore \quad d = 1.274$$

Velocity at the end of the trench weir, $V = \dfrac{Q}{A}$

$$\therefore \quad V = \frac{7.8}{3 \times 1.274} = 2.04 \text{ m/sec}$$

Required velocity $= \sqrt{gd_{max}} = \sqrt{9.81 \times \dfrac{30}{1000}} = .542$

Hence the design is ok.

Again from equation 3.16 the ordinate y of water surface curve measured downwards from the datum, the bed level of the entrance to trench weir is given by

$$y = \frac{n+1}{n} h_v$$

at the end of the trench weir, $y = \frac{0.5+1}{.5}\frac{d}{6}$

$$= \frac{1.5}{.5} \times \frac{1.274}{6}$$

$$= 0.637$$

Depth of trench with permissible submergence limit

$$y + d = 0.637 + 1.274$$

$$= 1.911\text{m}$$

Let us provide a 0.3m of clearance (optional) from streambed level to water level in the trench weir.

The velocity-distance relationship along the trench is assumed as

$$V = ax^n$$

Where, a = an arbitrary constant

x = distance along the trench weir.

n = 0.5 depends upon the water surface profile

So $2.04 = a\ 22^{.5}$

$\therefore$ $a = \frac{2.04}{22^{0.5}} = 0.43$

Hence the equation can be represented as

$$V = 0.43\ x^{0.5}$$

The velocity, cross-sectional area, depth of flow, bed and water surface elevations are calculated along the longitudinal profile of the trench weir and shown in Table. 9.1. The bed and water surface profile inside the trench weir is shown in Fig. 9.3.

9.3.2 Design of Rectangular Type Intake Well

The water collected in the trench leads to a rectangular intake structure at the left bank of the river. The location should be so fixed that the regime of the stream is not disturbed. The intake well should be a rectangular shape with its top above RL 1497.8m (HFL level at the site). The opening of the trench is restricted by construction of a wall above the U/S of the crest level so that the flow is an open channel flow and when the discharge is more, the opening acts as a flow through an external mouthpiece. The intake

SL.No.	X (m)	Q=qx	x^n	$V=ax^n$	A=Q/V	d=A/3	$y=\{(n+1)/n\}(v^2/2g)$	d+y	Bed level of channel	Water surface elevation	Remark
1	0	0.00	0.00	0.00	-	-	0.00	-	1495.84	1495.84	
2	1	0.35	1.00	0.43	0.82	0.27	0.03	0.30	1495.54	1495.81	
3	2	0.71	1.41	0.61	1.17	0.39	0.06	0.45	1495.39	1495.78	
4	3	1.06	1.73	0.74	1.43	0.48	0.08	0.56	1495.28	1495.76	
5	4	1.42	2.00	0.86	1.65	0.55	0.11	0.66	1495.18	1495.73	
6	5	1.77	2.24	0.96	1.84	0.61	0.14	0.76	1495.08	1495.70	
7	6	2.13	2.45	1.05	2.02	0.67	0.17	0.84	1495.00	1495.67	
8	7	2.48	2.65	1.14	2.18	0.73	0.20	0.93	1494.91	1495.64	
9	8	2.84	2.83	1.22	2.33	0.78	0.23	1.00	1494.84	1495.61	
10	9	3.19	3.00	1.29	2.47	0.82	0.25	1.08	1494.76	1495.59	
11	10	3.55	3.16	1.36	2.61	0.87	0.28	1.15	1494.69	1495.56	
12	11	3.90	3.32	1.43	2.73	0.91	0.31	1.22	1494.62	1495.53	
13	12	4.25	3.46	1.49	2.86	0.95	0.34	1.29	1494.55	1495.50	
14	13	4.61	3.61	1.55	2.97	0.99	0.37	1.36	1494.48	1495.47	
15	14	4.96	3.74	1.61	3.09	1.03	0.40	1.42	1494.42	1495.44	
16	15	5.32	3.87	1.67	3.19	1.06	0.42	1.49	1494.35	1495.42	
17	16	5.67	4.00	1.72	3.30	1.10	0.45	1.55	1494.29	1495.39	
18	17	6.03	4.12	1.77	3.40	1.13	0.48	1.61	1494.23	1495.36	
19	18	6.38	4.24	1.82	3.50	1.17	0.51	1.67	1494.17	1495.33	
20	19	6.74	4.36	1.87	3.59	1.20	0.54	1.74	1494.10	1495.30	
21	20	7.09	4.47	1.92	3.69	1.23	0.57	1.79	1494.05	1495.27	
22	21	7.45	4.58	1.97	3.78	1.26	0.59	1.85	1493.99	1495.25	
23	22	7.80	4.69	2.02	3.87	1.29	0.62	1.91	1493.93	1495.22	

TABLE.9.1: Bed Level and Water Surface Profile in The Asiganga Trench Weir

well accommodates two pipes, one is for flushing shingle and silt and the other one is the power conduit [Fig.9.2]. The shingle excluder pipe is designed to pass the surplus water to the river as well as to flush the shingle load.

9.3.2.1 Design of Shingle Excluder Pipe

Discharge required for flushing of desilting well

$$= 0.15 \times 6.756 = 1.013 \text{ m}^3\text{/sec.}$$

Taking a minimum velocity of water as 2.5 m /sec in the conduit to flush the sediment we have, $Q = A \times V$

i.e. $\quad 1.013 = \frac{\pi}{4} \times d^2 \times 2.5$

$\Rightarrow \quad d \quad = 0.718\text{m}$

Hence provide a diameter of hume pipe as 700mm for intake well flushing conduit [Fig.9.4]. Taking the flow inside the conduit as external mouthpiece flow, the velocity inside the conduit will be

$$V = C_d\sqrt{2gh}$$

$$\Rightarrow \quad 2.5 = 0.64\sqrt{2 \times 9.81 \times h}$$

$$\Rightarrow \quad h = 0.77\text{m}$$

For minimum stream flow condition, the average available head will be -

Rl of water level at the end of the trench inside – Rl of centerline of flushing conduit

$$= 1495.22\text{-}(1492.82\text{+}.35) = 2.05\text{m} > 0.77\text{m (hence ok)}$$

If we provide 650mm standard I.D. Hume pipe the velocity generated will be

$$V = \frac{Q}{A} = \frac{1.013}{\left(\frac{\pi}{4} \times .65^2\right)} = 3.052 > 2.5 \text{ (hence safe)}$$

Again $\quad V = C_v\sqrt{2gh} = 0.64\sqrt{2 \times 9.81 \times h} = 3.052$

$\therefore \quad h = 1.159$

Hence the diameter 650mm can also adopted with minimum net head at exit point of order 1.159m.The discharge level of the flushing pipe should be as far as possible above high flood level and the discharge has a free fall. If there is any susceptible that the exit end is below down stream HFL during flood then there should at least a net head of .77m or 1.159m between U/S HFL and D/S HFL respectively for the area of conduit use at the exit end. As far as possible the exit site should be so chosen that there should be a free fall from conduit into the river during high floods.

9.3.2.2 Design Of Power Conduit

The discharge required for initial reaches of power conduit = 6.756 cumecs

The conduit can be made of channel of circular or rectangular cross-section with cut and covers arrangement upto desilting chamber. If we consider it as a rectangular channel of equal depth and breadth, for free flow condition the design is as follows.

Data: $Q = 6.756$

$n = 0.016$

$s = 1:1000$

So $$V = \frac{Q}{A} = \frac{Q}{b \times d} = \frac{Q}{d^2} = \frac{6.756}{d^2}$$

$$R = \frac{A}{P} = \frac{d^2}{3d} = \frac{d}{3}$$

From Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\Rightarrow \quad \frac{6.756}{d^2} = \frac{1}{0.016} \left(\frac{d}{3}\right)^{2/3} (.001)^{1/2}$$

$$\Rightarrow \quad d = 2.08\text{m}.$$

So, $V = 1.56 < 2.7$ m/sec (hence ok)

Provide 2.08 × 2.3m channel of power conduit from intake well [Fig.9.4].

9.3.3 Design of Desilting Basin

Water in the hilly stream is silt free except during monsoons. Most of the bed loads entered into the trench weir settle in the intake well and flushed out through the shingle excluder pipe at the intake. The suspended silt particles, which is not settled in the settling well entered into the power conduit along with intake water. So to trap these suspended silt particles further desilting arrangement is required. As discussed in chapter 3 the provision of desilting basin is the best one in the present context as

- It is widely accepted in different SHP scheme in India and working successfully.

- Although the continuous type required more water, but is a full proof method with respect to other desilting arrangement.
- Easier to construct, time tested and may not require vigorous model study like other type as far as SHP scheme is concerned.

Just before the desilting tank a weir should be provided to spill over the water in excess of design discharge back to the original river. The desilting tank is located at a relatively flatter ground keeping in view the structural safety, economy in design, operation, easy accessibility and availability of natural drainage.

Data available:

Design discharge for desilting basin = 6.756 cumec.

Density of water = 1.033 t/cumec.

In the present case the desilting basin should be designed to exclude 0.2 mm size (d) of silt particles from intake water owing to the fact that the Himalayan streams carries bed and suspended loads having greater concentration of sub angular sand and quartz particles (hardness greater than 7) responsible for damage of underwater component of hydraulic turbines and conduit system .

The minimum flow through velocity inside the desilting basin as per camp is

$$U = a\sqrt{d} = 44\sqrt{0.2} = 19.67 \text{ cm/sec}$$

The flow through velocity as obtained above is less and will create hydraulic short circuit inside the basin. So as per mosonyi [31] the velocity should be between 0.4 to 0.6 m/sec.

Let us adopt an average velocity of 0.4 m/sec inside the basin.

Settling velocity of particle size 0.2mm from sudry's curve = 2.7 cm/sec

Assuming a depth of desilting basin as 4m,

we have, $$L = Ut = U \times \frac{h}{\omega} = 0.4 \times \frac{4}{0.027} = 59.26\text{m}$$

Again $$Q = bhU = b \times 4 \times 0.4$$

$$\Rightarrow \quad b = \frac{6.756}{4 \times 0.4} = 4.22\text{m}$$

Using Velikanov equation i.e $L = \dfrac{\lambda^2 U^2 \left(\sqrt{h} - 0.2\right)^2}{7.51\omega^2}$

By substituting all the known values, $\lambda = 0.8$, and by referring the Fig. 4.2 the efficiency is come out as 87.5 %

Let us reduce the flow through velocity to 0.3 m/sec and by repeating the above steps we have

$$L = U \times \frac{h}{\omega} = 0.3 \times \frac{4}{0.027} = 44.44\text{m and,}$$

$$b = \frac{Q}{hU} = \frac{6.756}{4 \times 0.3} = 5.63$$

and, λ obtained from velikanov equation as 0.91 with efficiency from fig. 4.2 as 90%.

Coming in a reverse way by using Fig.4.2, for $\eta = 95\ \%$

$$\lambda = 1.2.$$

By substituting all the known values as just previous we have

the length, $$L = \frac{(1.2)^2 \times (.3)^2 \left(\sqrt{4} - 0.2\right)^2}{7.51 \times (.027)^2} = 76.69\text{m.}$$

The designed length is 172% of that obtained from Mosonyi method. Taking the turbulence effect in consideration

$$\omega' = \frac{0.132}{\sqrt{h}} U = \frac{0.132}{\sqrt{4}} \times 0.3 = 0.0198$$

$$\therefore \quad \omega_c = \omega - \omega' = 0.027 - 0.0198 = 0.0072$$

So the revised length will be

$$L = \frac{Uh}{\omega_c} = \frac{0.3 \times 4}{0.0072} = 166.67\text{m}$$

Changing the depth of basin to 5m and 6m and taking all other data as usual we have the following values

TABLE 9.2: Basin Size, Taking Turbulence In Consideration

SL.No.	DEPTH OF BASIN ASSUMED	BREADTH OBTAINED	LENGTH OBTAINED
1	4	5.63	166.67
2	5	4.5	161.458
3	6	3.75	166.67

The different trial cases for fixation of size of desilting basin are given in Table 9.3 for ready reference.

SL.NO.	DENSITY (T/Cumec)	PERMISSIBLE VELOCITY BY CAMP (cm/Sec)	VELOCITY TAKEN (cm/Sec)	SETTLING VELOCITY (Cm/Sec)	DEPTH ASSUMED (m)	LENGTH (m)	BREADTH (m)	EFFICIENCY (%)
1	1.033	19.68	40	2.7	3	44.44	5.63	87.5
2	1.033	19.68	40	2.7	3.5	51.85	4.82	87.5
3	1.033	19.68	40	2.7	4	59.26	4.22	87.5
4	1.033	19.68	40	2.7	4.5	66.66	3.75	87.5
5	1.033	19.68	40	2.7	6	88.89	2.815	87
6	1.033	19.68	30	2.7	4	44.44	5.63	90
7	1.033	19.68	30	2.7	4	65.00	5.63	93
8	1.033	19.68	30	2.7	4	70	5.63	94
9	1.033	19.68	30	2.7	4	76.69	5.63	95
10	1.033	19.68	30	2.7	4	88.88	5.63	95
11	1.033	19.68	30	2.7	5	55.55	4.5	90
12	1.033	19.68	30	2.7	6	66.66	3.75	90

TABLE. 9.3: Dimensions and Efficiency of Desilting Basin by Mosonyi And Velikanov Approach

Camp and Dobbin's approach:

Using the known values of different parameters and the Fig. 4.3, we have

$$\frac{\omega D^{1/6}}{nU\sqrt{g}} = \frac{0.027 \times 4^{1/6} \times 100}{0.016 \times 30 \times \sqrt{9.81}} = 2.26$$

Corresponding to the value of 2.26 and for efficiency 100%, $\frac{\omega L}{UD} = 1.5$

Now substituting the known values of different parameters, the length of basin will be

L = 66.66m < 76.69m as obtained above. Hence ok.

U.S.B.R Approach:

As per this approach the efficiency will be, $\eta = 1 - e^{-\omega L/UD}$

Substituting the value of ω,L(as obtained from velikanov approach),U,D we have

$$\eta = 1 - e^{\left(-\frac{0.027 \times 76.7}{0.3 \times 4}\right)} = .82 \text{ i.e. } 82\ \%$$

By increasing the basin length the following different efficiencies are obtained

TABLE 9.4: Basin Length and Its Efficiency by USBR Approach

SL.NO	LENGTH OF BASIN L(m)	EFFICIENCY η(%)
1	90	86.8
2	100	89.46
3	120	93.28
4	150	96.58

Sumer Approach:

As per sumer the relation between the settling length and efficiency η is

$$L = \frac{-6\left[\frac{U}{U_*}\right]D}{k\lambda} \ln(1-\eta)$$

where, $\lambda = f(\beta)$

$$\beta = \frac{\omega}{kU_*}$$

$$U_* = U\sqrt{\frac{f}{8}} = 0.3\sqrt{\frac{0.02}{8}} = 0.015$$

$$k = 0.4$$

$$\therefore \quad \beta = \frac{0.027}{0.4 \times 0.015} = 4.5$$

Using the fig. 4.6 for $\beta = 4.5$, $\lambda = 50$

So keeping $\eta = 95$ % and other known values we have

$$L = \frac{-6\left[\frac{0.3}{0.015}\right]4}{0.4 \times 50} \ln(1 - .95) = 71.89\text{m.} < 76.69 \text{ (Hence ok.)}$$

For $\eta = 99$ %, L = 110m

From all the above considerations the value obtained by valikanov approach is most reasonable and considered for final dimensions of the desilting basins [Fig.9.5], although for ensuring efficient functioning a model study is very much essential.

TABLE. 9.5: Comparative Statement Showing Dimensions and Efficiency of Desilting Tank by Different Approaches.

SL.NO.	NAME OF APPROACH	LENGTH (m)	BREADTH (m)	HEIGHT (m)	EFFICIENCY (%)
1	Mosonyi (using Velicanov efficiency curve)	76.69	5.63	4	95
3	Camp and Dobbins	66.66	5.63	4	100
4	U.S.B.R	150	5.63	4	96.58
5	Sumer	71.89	5.63	4	95

9.3.4 Design of Water Conductor System

Water conductor system can be of closed conduit or open channel (with or with out cut and cover arrangement). The system depends upon the layout, the topography, the availability of cheap construction material, etc. Although the adoption of closed conduit is more advantageous in layout and maintenance consideration, but it is a costly affair. So if the topography permits the construction of open channel is advisable. A systematic design procedure is outlined below. The dimensions of other sections with rounded corner can be obtained by following the principles outlined in chapter 5.

In general for small discharges, the rectangular channel is most preferred for economical, constructional and maintenance point of view.

As per V.T.Chow[14] for small discharges in rectangular channel $\frac{\text{Breadth}}{\text{Depth}}$ ratio should be one (1).

Here the other datas are n =0.016

$$s = 1:1000$$

$$B = D$$

So $$V = \frac{Q}{A} = \frac{Q}{D^2} = \frac{5.63}{D^2}$$

$$R = \frac{A}{P} = \frac{D^2}{3D} = \frac{D}{3}$$

From manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\frac{5.63}{D^2} = \frac{1}{0.016} \times \left(\frac{D}{3}\right)^{2/3} (.001)^{1/2}$$

$$\Rightarrow \quad D = 1.948$$

So $$B = 1.948$$

$$V = \frac{5.63}{1.948^2} = 1.48 \text{ m/sec} < 2.7 \text{ m/sec (hence ok)}$$

9.3.4.1 Free Board

By using the method mentioned in the chapter 5, the area of the channel

$$A = \text{wetted area} = 1.948 \times 1.948 = 3.794 \text{ m}^2$$

Mean velocity, V = 1.48 m/sec

For gradual opening of intake gate, $\Delta h_{max} = \frac{V^2}{4g} + V\sqrt{\frac{h}{g}}$

$$= \frac{1.48^2}{4 \times 9.81} + 1.48\sqrt{\frac{1.948}{9.81}}$$

$$= .715\text{m}$$

let us Provide a free board of 0.75m

So the total height of the channel will be, h = 1.984+.75 = 2.698m. ≈2.7m

Hence the size of the rectangular channel will be 1.948m × 2.7m and shown in Fig. 9.4.

If a trapezoidal shape of open channel will be provided then the surge height may be calculated in similar fashion by using effective height by trial and error with initial approximation of surge height [45].

9.3.5 Design of Forebay

The present water conductor system is a rectangular open channel; hence a forebay is required for penstock intake. As per design aspect discussed in chapter 6, for single penstock intake the following calculations are made.

Let us assume the storage capacity of fore bay is for 2 minutes. So the volume required

$$= Q \times t = 5.63 \times 2 \times 60 = 675.6 \text{ m}^3$$

Let us provide a 5m width and 3m depth of water above minimum water level in the forebay as calculated in section 9.3.6. So the required length of the forebay will be

$$L = \frac{675.6}{5 \times 3.0} = 45.04 \text{ m}$$

Hence provide a 46 m of forebay length.

Now the total height of water in the forebay

$$= 3+3.840 \text{ (as calculated in Section 9.3.6)} = 6.840 \text{ m.}$$

So the velocity inside forebay will be

$$V = \frac{Q}{A} = \frac{5.63}{5 \times 6.840} = 0.165 \text{ m/sec} < 0.5\text{m/sec (hence ok)}$$

During sudden closer of the turbine the height of surge will be

$$\nabla h_{max} = \frac{V^2}{2g} + \sqrt{\left(\frac{V^2}{2g}\right)^2 + 2\left(\frac{V^2}{2g}\right)\frac{A}{B}}$$

$$= \frac{.165^2}{2 \times 9.81} + \sqrt{\left(\frac{.165^2}{2 \times 9.81}\right)^2 + 2\left(\frac{.165^2}{2 \times 9.81}\right)\frac{5 \times 6.840}{5}}$$

$$= 0.139$$

The detailed dimensions of forebay are shown in Fig.9.6, however the dimension of forebay may be altered as per the site condition by assuming suitable length, breadth and height of water above dead storage level in the forebay subject to a minimum as mentioned in chapter 6 .

9.3.6 Design of Penstock

The available design net head as per the field condition is 96.15 m. Considering 25 % of head for water hammer effect, the total head required for design of penstock will be

$$H = 1.25\times96.15 = 120.187\text{m}$$

Using steel as penstock material, the thick ness of penstock shell will be

$$t = \frac{Pd}{2\sigma\eta} = \frac{\gamma Hd}{2\sigma\eta}$$

where , t = Shell thickness

P = Pressure in the penstock

γ = Density of water

σ= Hoop stress in steel

d = Diameter of penstock

η = Efficiency of longitudinal joint = 95%

Again restricting the velocity in the penstock as 5 m/sec and testing pressure as 1.5P, we have

$$d = 1.13\sqrt{\frac{5.63}{5}} = 1.199 \approx 1.2\text{m}$$

So $$t = \frac{1000\times1.5\times120.187\times1.2}{2\times1.2\times10^6\times0.95} = 9.48\text{ mm} \approx 10\text{ mm}.$$

Let us add an allowance of 2 mm of penstock shell thickness for corrosion, the total shell thick ness required = 10 +2 = 12 mm.

As the other aspect of selection of diameter is depends upon the cost factor, governing condition of turbine and technical aspect of manufacturing and transportation, it can be revised at the specific prevalent condition.

The bell mouth entry of penstock intake can be provided as per the following equation [11] i.e.

$$\frac{X^2}{D^2} + \frac{Y^2}{0.4286D^2} = 1$$

Substituting the value of D as obtained above we have the following equation

$$\frac{X^2}{1.44} + \frac{Y^2}{0.617} = 1$$

By substituting the different values of X the values of Y can be obtained from the above equation. The table shows the different values of X and Y for the transition of converting rectangular section to circular section.

Semi major axis of ellipse = D = 1.2

Semi minor axis of ellipse = 0.4286D = 0.514

TABLE 9.6: Profile of Bell Mouth Entry of Penstock Intake.

Sl. No.	X	Y
1	0.1	0.513
2	0.2	0.507
3	0.4	0.485
4	0.8	0.383
5	1.0	0.284
6	1.2	0.0

The penstock intake may be inclined or horizontal according to the profile of the terrain. The following parameters are calculated for fixing the positioning of the penstock intake

The height of the bell mouth entry, $H = 2D = 2\times1.2 = 2.4$m

The minimum water cushion above the bell mouth entry = $0.6\ H = 0.6 \times 2.4 = 1.44$m

So minimum total height above floor of penstock intake

$= H + 0.6\ H = 2.4 + 1.44 = 3.84$m

The depth between the centerline of the penstock and minimum water surface

$= 3.84-(H/2) = 3.84 - 1.2 = 2.64$

As per CBIP[10] the minimum depth of water above centerline of penstock should be more than 0.8H. So, $0.8H = 0.8 \times 2.4 = 1.92 < 2.64$ (hence ok).

Let us fix the minimum drawdown level of head pond at 1.053 m below the bed level of headrace. So maximum water height from floor level of penstock intake will be

$= 3.84 + 1.052 + 1.948 = 6.84$ m

By providing 0.5m of water cushion for flushing and space for silt accumulation, the total height will $= 6.84 + 0.5 = 7.34$ m.

The detail dimensioning of penstock intake is shown in Fig. 9.7.

9.3.7 Design of Forebay Spillway

The ungated ogee spillway in the forebay is designed for a design discharge

$$Q = 5.63 \text{ cumecs}$$

Let us provide the spillway waterway, L = 4 m.Neglecting the end contraction and using the basic equation of flow over weirs with coefficient of discharge as 2.2, we have

$$Q = C_d L_e H_e^{3/2}$$

$$\Rightarrow \quad 5.63 = 2.2 \times 4 \times H_e^{3/2}$$

$$\Rightarrow \quad H_e = \left[\frac{5.63}{2.2 \times 4}\right]^{2/3} = 0.742 \text{ m}$$

Taking end contraction in account with a square abutment and headwall is at 90^0 to the direction of flow, the coefficient of contraction will be 0.2. So, we have the effective length, $L_e = L - 2 \times K_a \times 0.742$

$$\Rightarrow \quad L_e = 4 - 2 \times 0.2 \times 0.742 = 3.7 \text{ m.}$$

So, $\quad 5.63 = 2.2 \times 3.7 \times (H_e)^{3/2}$

$$\Rightarrow \quad H_e = \left(\frac{5.63}{2.2 \times 3.7}\right)^{2/3} = 0.782 \text{ m}$$

Let us take the design head for spillway = 0.8 m

So, velocity of approach $= \dfrac{Q}{A} = \dfrac{5.63}{4 \times (6.84 + 0.8)} = 0.184$ m/sec

Now the velocity head will be , $\dfrac{V^2}{2g} = \dfrac{0.184^2}{2 \times 9.81} = 0.0017$ m , hence neglected.

Taking a free board of 0.5m, the height of forebay sidewall above the spillway crest will be $= 0.5 + 0.8 = 1.3$ m .

So for safety operation maximum height of forebay = 7.34 + 1.3 = 8.64 m

The size of spillway may be altered as per the desired depth of flow over it and site condition.

9.3.8 Design of Tailrace Channel

The tailrace is design for a discharge equal to design discharge of the scheme with maximum tail water level condition. The data on tail water level is not available; however the design principle is just similar to the design of water conductor system.

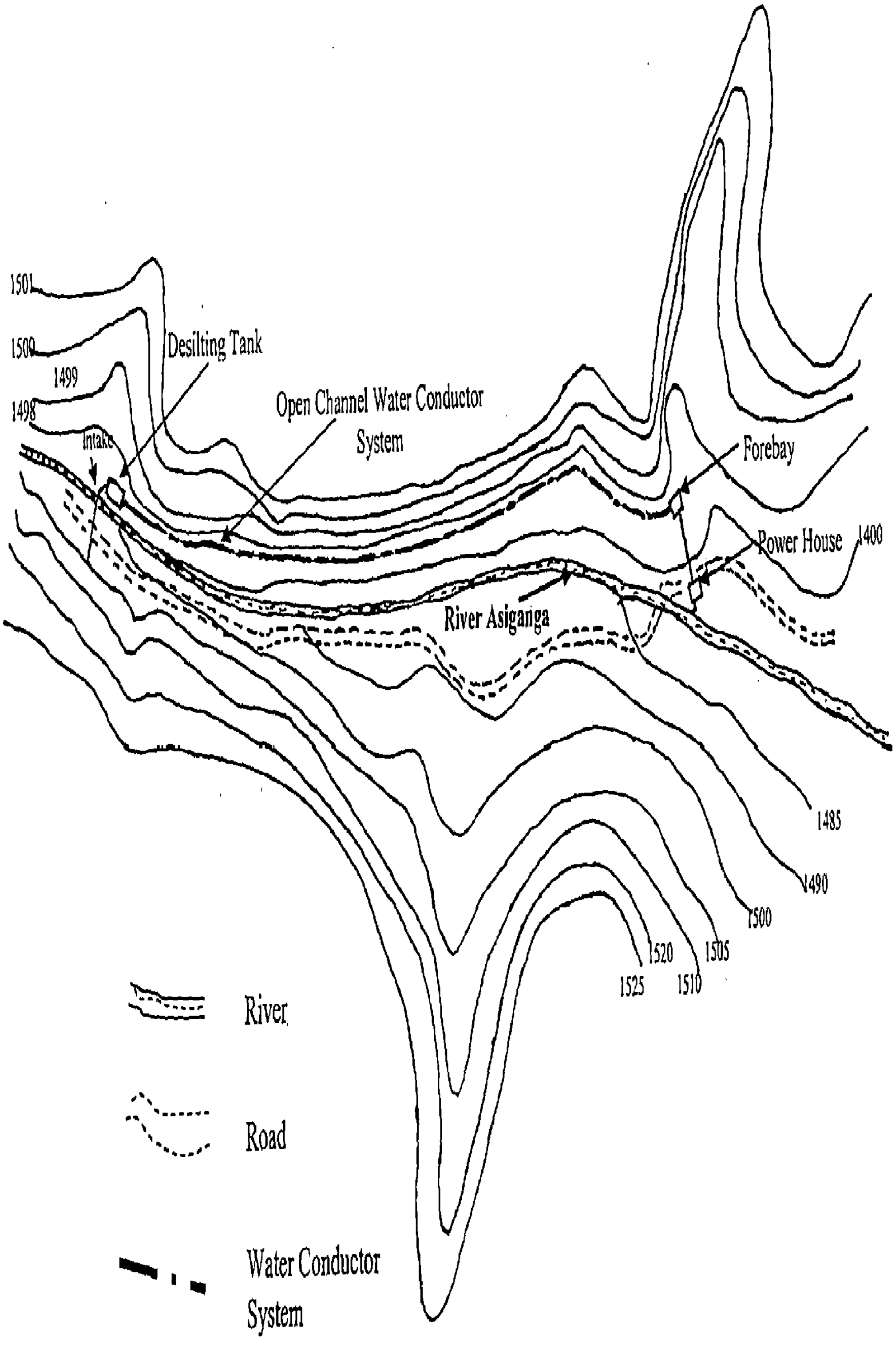


Fig 9.1: LAYOUT OF ASIGANGA STAGE - I SMALL HYDRO PROJECT

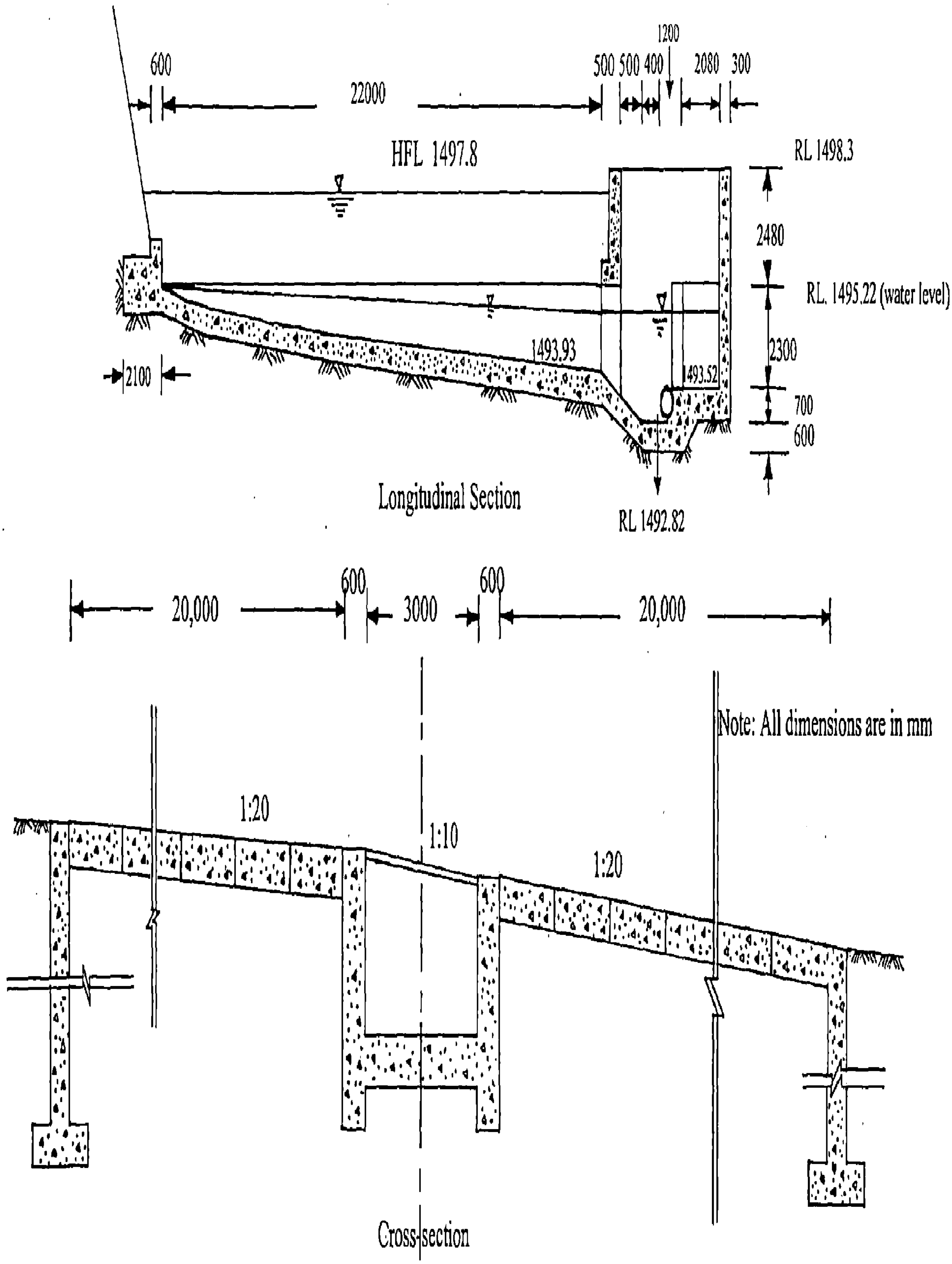


Fig. 9.2: LONGITUDINAL AND CROSS-SECTION OF PROPOSED TRENCH WEIR

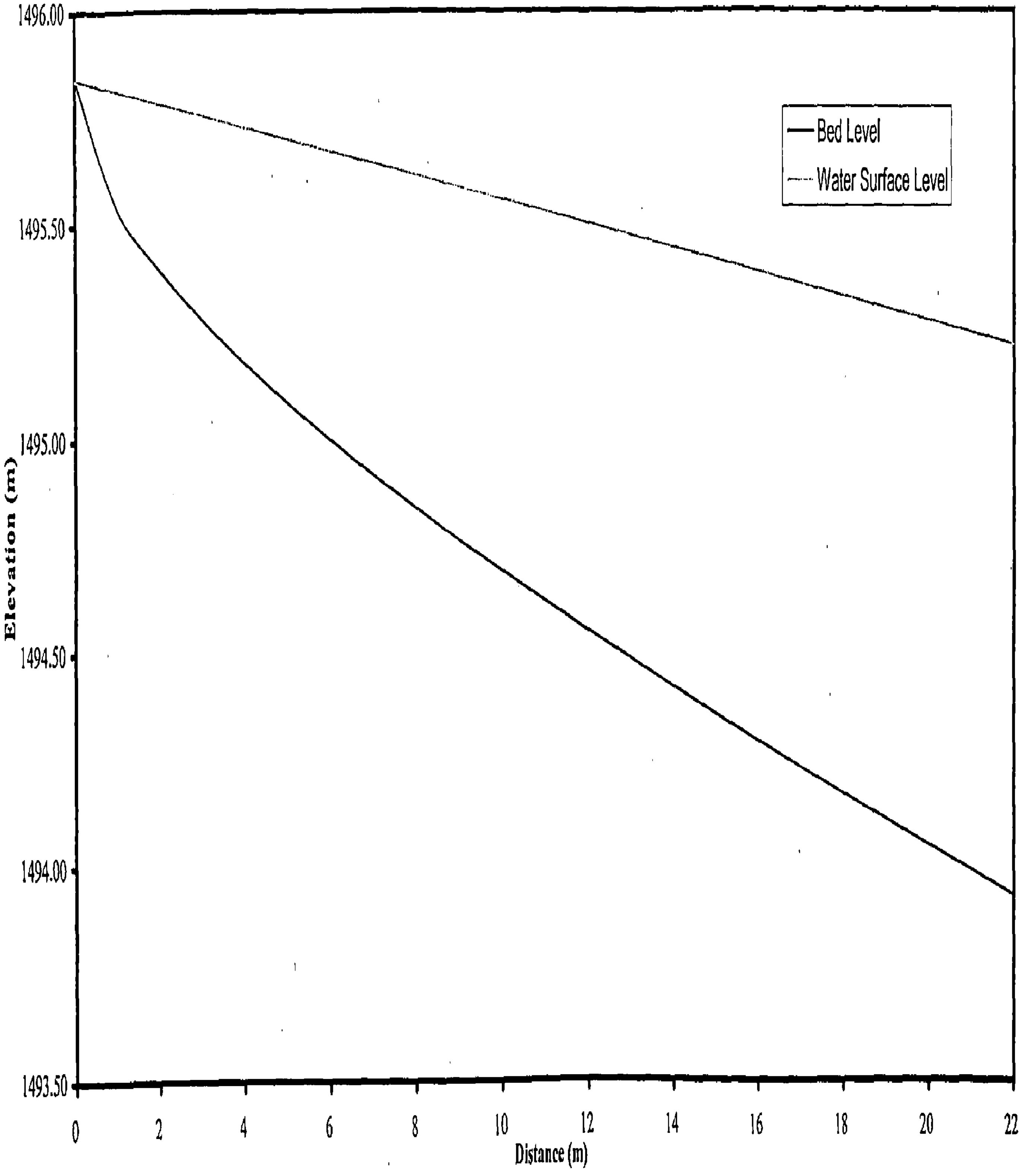
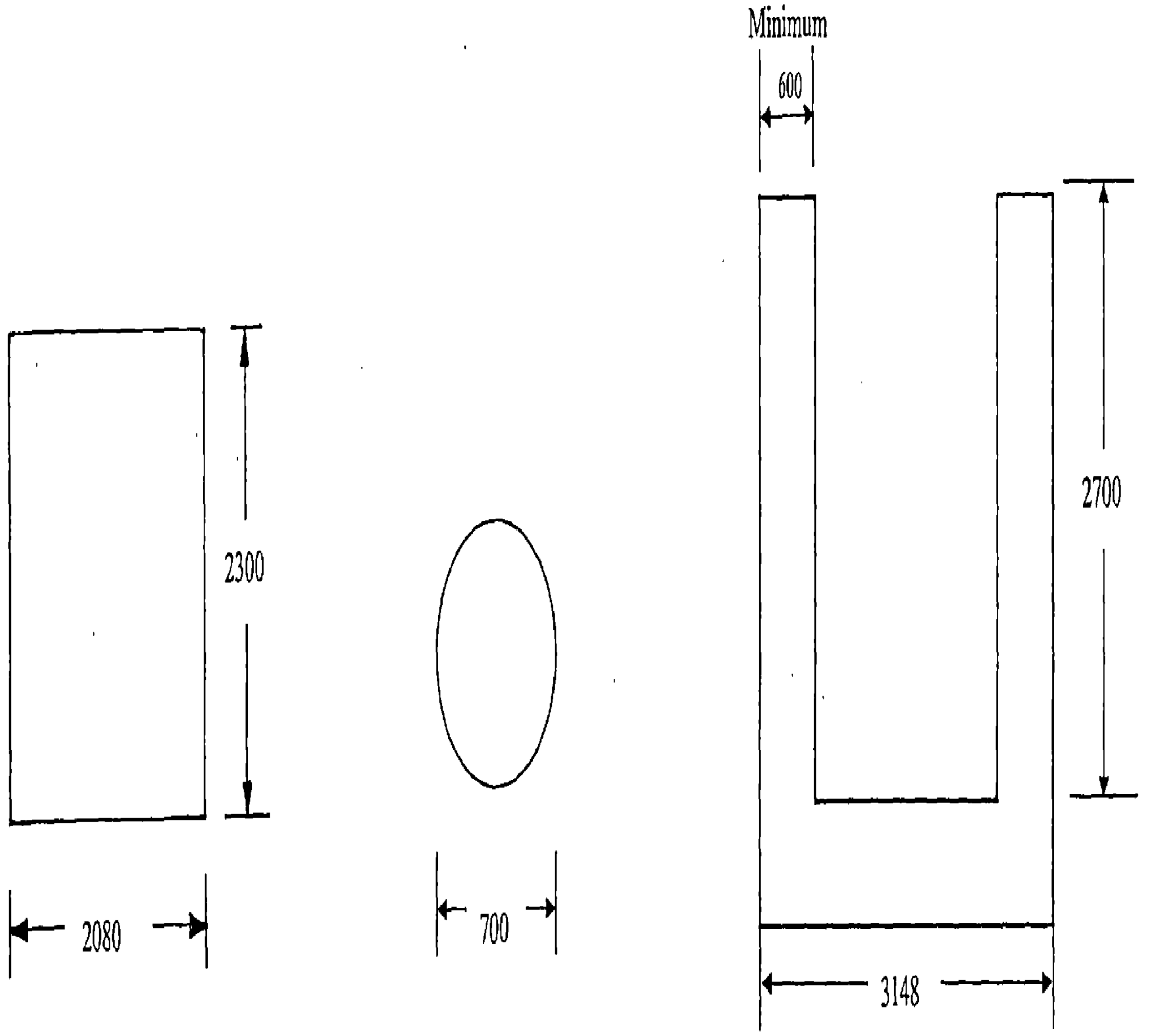


Fig.9.3: Bed and Water Surface Level of Proposed Asiganga SHP Trench Weir



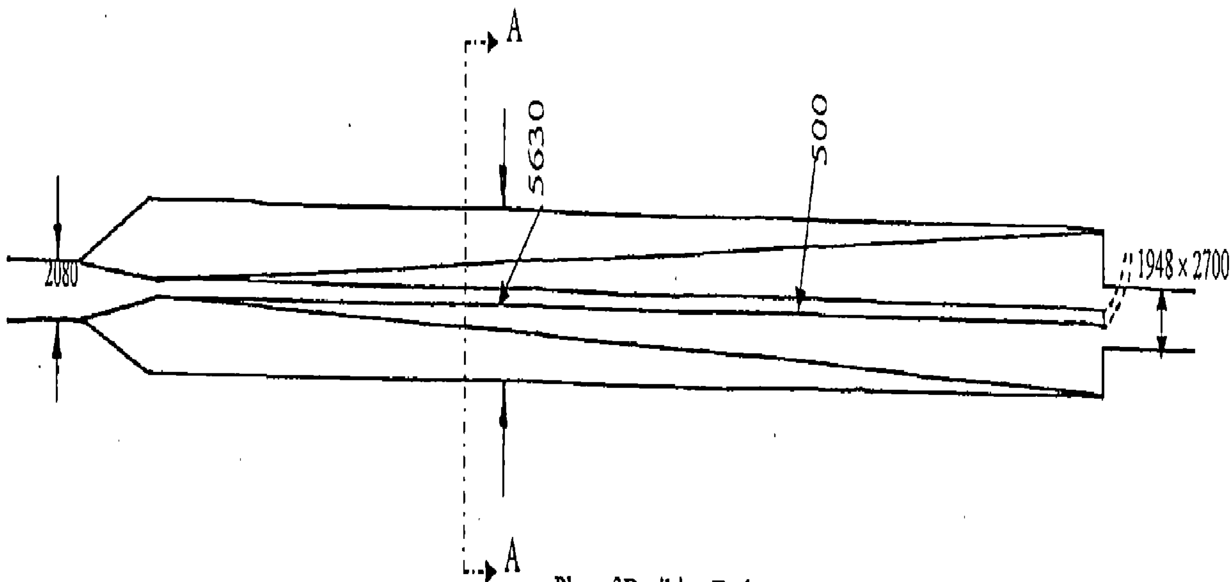
(a) Section of Power Channel from Intake Well to Desilting Basin

(b) Section of Shingle Excluder Steel Pipe

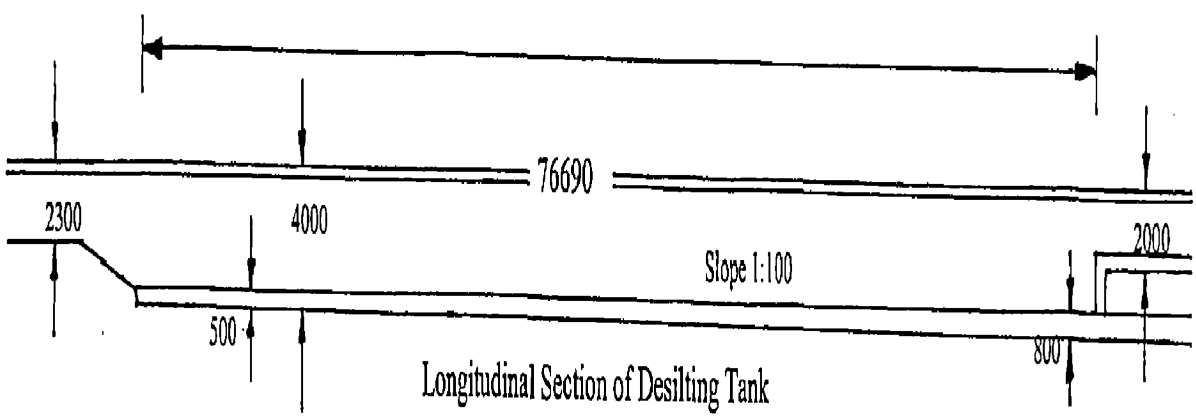
(c) Section of Power Channel / Headrace Channel

(All dimensions are in mm)

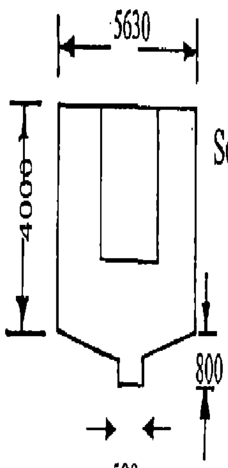
Fig. 9.4: SECTION OF POWER AND FLUSHING CONDUIT



Plan of Desilting Tank



Longitudinal Section of Desilting Tank



Section at A-A

Note-All dimentions are in mm

Fig.9.5:DESIGN DETAILS OF DESILTING TANK

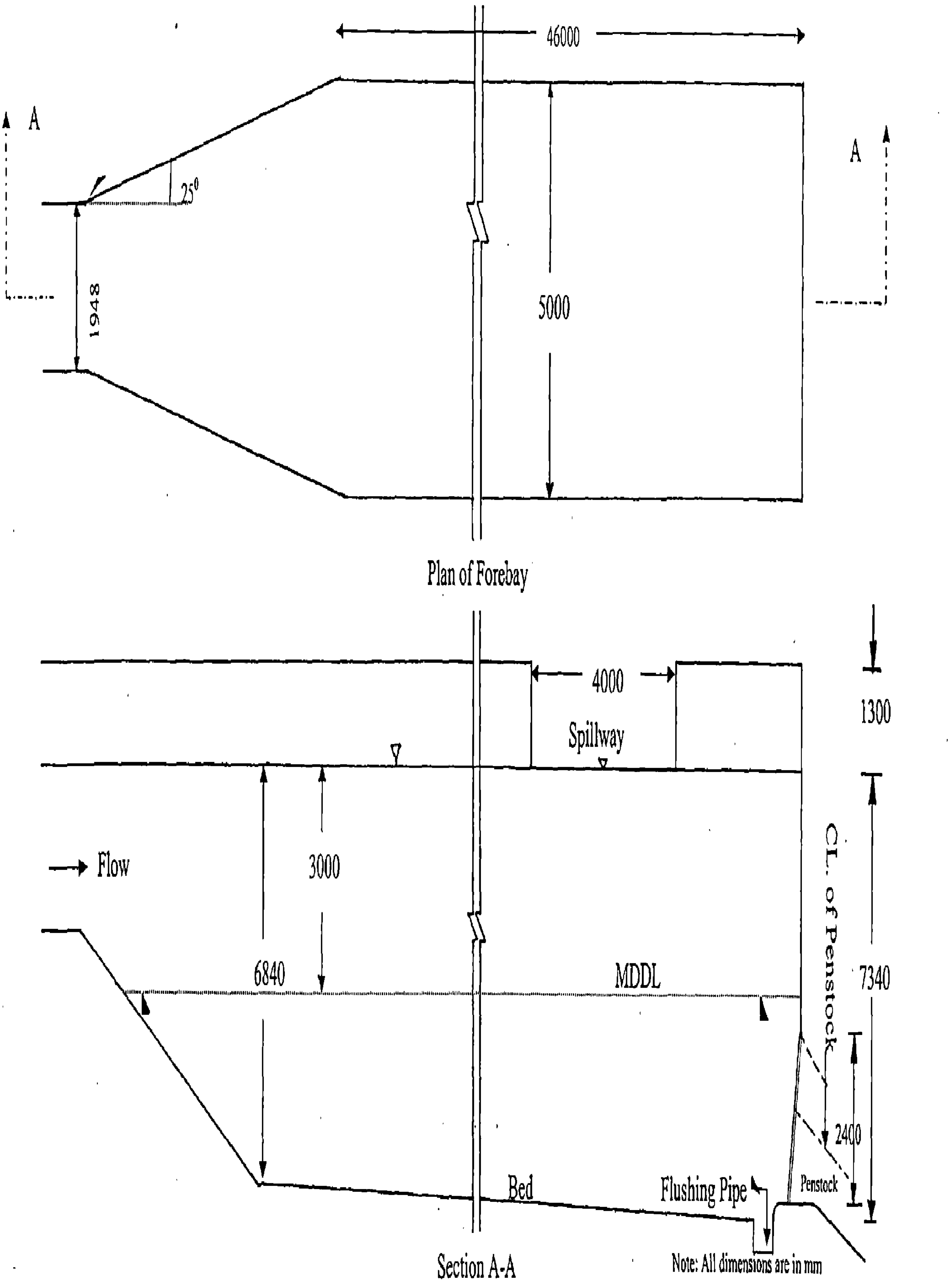


Fig.9.6: DESIGN DETAILS OF FOREBAY

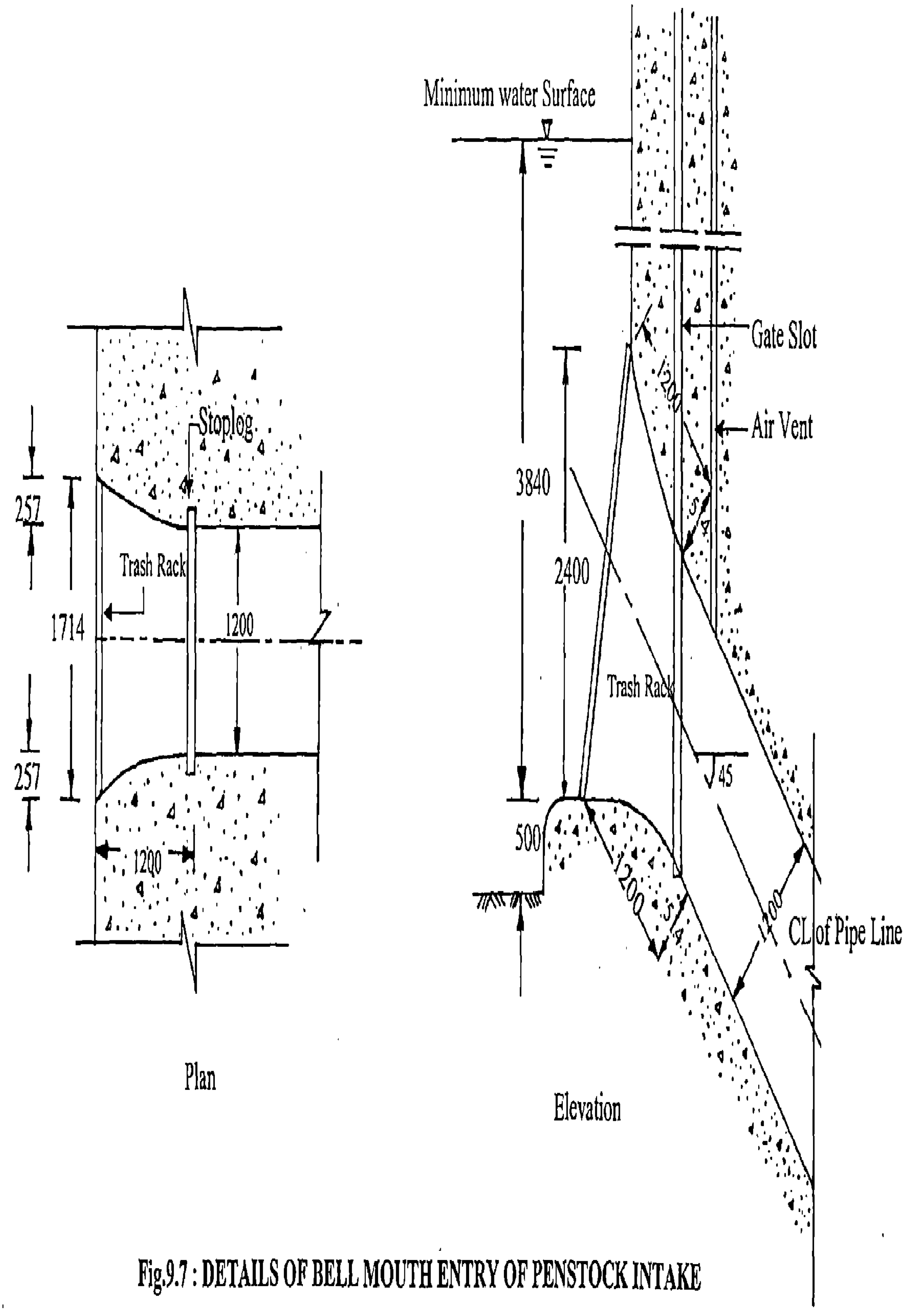


Fig.9.7 : DETAILS OF BELL MOUTH ENTRY OF PENSTOCK INTAKE

CHAPTER 10
CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDIES

10.1 GENERAL

In the foregoing chapters a detailed study on design procedure of different hydraulic components of civil structures in run-of river type small hydro schemes adopted by different organizations and countries in varying site conditions has been made. It is observed that the designs of hydraulics of civil structures are mainly site specific. Although its standardization is not prudent, but a general idea of standardized dimensions for different discharges and site conditions is vital for planning and popularizing the SHP schemes in Public as well as private sector. In the Himalayan region there is ample scope for development of SHP schemes on every perennial water source i.e. streams/rivers. So an economical standardized design of civil structures is very much required.

10.2 CONCLUSIONS

The review of design principles and practices with the design worked out for the study of SHP scheme on river Asiganga, a tributary of river Bhagirathi has given the following conclusions.

1. In the Himalayan region the conventional raised crest type diversion weir is not appropriate for head works in run-of-type SHP scheme owing to its inherent characteristics and stream behavior. The tyroller or bottom intake type headworks is the most suitable one for the above specific region. It not only extracts the design discharge in dry weather periods from the perennial sources of stream but also doesn't interfere with the flow in the stream in flood situations. Coming to the shape of the trench, the literature and model study indicates the suitability of trapezoidal section. But it is after all the designer's preference keeping in view different aspects like structural, ease of construction skill, site condition, economy and the life expectancy of the project.

2. The choice of suitable type of desilting arrangement is of vital importance both in functional efficiency and economy. The matter of suspended and bed load in stream / river water is still a subject of research , however the result of various study on desilting arrangement can guide one to have an proper choice . The desilting basin

/chambers are the most efficient and simple one in construction technique with a disadvantage of massive excavation and construction. The use of vortex type desilting basin in comparison to settling basin is a matter of designer's choice and depends on site conditions. However not much data on field performance of vortex type is available. As a fact, the choice of desilting arrangement depends upon the size of particles to be removed from water and the head available for power generation. So a judicious choice is essential keeping in mind various factors such as capacity of project, volume of work, economy, future maintenance and above all the ownership (Govt/private) for running of SHP scheme.

3. The choice of water conductor system is purely a case of relative economy, existing site condition, topography, layout, repair maintenance cost and the local engineering skill available. The open channel is the best suitable one for stable hill slopes; otherwise the low pressure closed conduit is advantageous in vertical alignment and maintenance point of view.

4. In small hydel, depending on type of water conductor and length the provision for forebay or surge tank is decided.

5. Penstock in SHP scheme should be exposed one for easy maintenance. The material should be of structurally stable and durable. In general steel penstock is suitable for high head SHP stations. In case of low head low capacity SHP as in micro hydel system steel penstock may be replaced by concrete hume pipe, HDPE, UPVC pipes.

10.3 FURTHER SUGGESTIONS

(i) A study on usefulness of coanda effect screen in SHP diversion headworks may be carried out for cost reduction and simplicity of the scheme.

(ii) A study on the aspect of providing siphon type intake at small barrage and forebay may be carried out with an objective of less maintenance and automation of SHP scheme.

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