A CRITICAL REVIEW OF WATER CONDUCTOR SYSTEM FOR MINI-MICRO HYDEL PROJECTS

A DISSERTATION

submitted in partial fulfilment of the requirements for the award of the degree of

MASTER OF ENGINEERING

in

WATER RESOURCES DEVELOPMENT

(Civil)

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled, **"A CRITICAL REVIEW OF WATER CONDUCTOR SYSTEM FOR MINI MICRO HYDEL PROJECTS"** in partial fulfilment of the requirement for the award of the degree of **MASTER OF ENGINEERING** in Water Resources Development, submitted at the Water Resources Development Training Centre, University of Roorkee, Roorkee (India) is an authentic record of my original work carried out during the period from July 16, 1996 to Feburary 21, 1997 under the supervision of **Er. D.K. Agarwal** and **Prof. Gopal Chauhan**, WRDTC, University of Roorkee, Roorkee.

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

(RAJENDER KUMAR)

Place : Roorkee

DATED : February 21, 1997

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

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SYNOPSIS

Power is the prime mover of economy in any country. In fact, the world over, per capita availability of power is considered to be one of the indices of economic growth and prosperity of the country.

In a developing country like India, with ever increasing population, continuous growth in agriculture and power has become indispensable for meeting the escalating needs of food and fibre as well as for the growth of multifarious industries to have a self reliant economy.

India has an appreciable Hydro-Power-potential. As per assessment of the Central Electricity Authority in 1987-88, the hydro-electric potential of the entire country is about 84,000 MW at 60% load factor. The schemes executed so far and those under construction on completion will exploit only about 20% of the assessed ultimate potential.

Hydropower is attractive because it is a renewable energy resource that can never exhaust and avoids the pollution associated with burning fossil fuels. However, most of the large hydro schemes involve massive dams, impounding enormous volumes of water in man-made lakes, in order to provide year round power by smoothing out the fluctuations of river flow. But construction of the structures attract problems like submergence of land, rehabilitation of large population, heavy initial expenditure, long completion time etc. Because of all these causes big hydroelectric schemes such as Tehri dam and Sardar Sarovar dam are facing lot of resistance from a few so called environmentalists resulting in delay in the completion of project.

In view of all this mini-micro hydro concept has found widespread accpetance in many parts of the world primarily for providing electric power sources at locations not served by transmission from larger or conventional generation centres.

Once the mini-micro hydroelectric potential has been estimated the planning engineers are then faced with the task of making these shcemes economical so that benefit cost ratio could be agreeable. Viewing all this an attempt has been made in this dissertation how to critically review the design aspects of the various components of the "Water conductor system of mini-micro hydel projects, which practically cover all main components from diversion to tail race", so as to arrive at an economical, practical and safe design.

CONTENTS

CHAPTER - 1

INTRODUCTION

1.1 GENERAL

India has large potential of hydro energy in medium and big projects. After independence when pace of power development had been stepped up, large hydroelectric projects were planned. Many of these had been executed and are being implemented. For development of power in small scale, cheaper alternative of generation by utilising fossil fuel was available. Thus development of small hydro went into background to some extent. After some time the uncertainty of availability of fossil fuels in future coupled with steep rise in their prices compelled many countries especially the developing ones to seek recourse to renewable source of energy. Among these renewable sources of energy, development of mini-micro hydel projects has been found most suitable. The mini-micro has its own merits vis-a-vis development of major hydro potential.

1.2 HYDROPOWER DEVELOPMENT

1.2.1 Hydropower a Reliable Source

Hydropower is a renewable, non polluting and a reliable source of energy. Hydropower stations have much larger life and have over 85% of availability as compared to 25 years life of a thermal power plant with an availability of around 70-75%. The cost of generation as well as cost of operation and maintenance are also lower as compared with the thermal projects. Hydropower stations do not put any added burden on infrastructure for supply (mines) and transportation (railways) of coal. Moreover such plants, with inherent ability for quick "start and stop" operations are ideally suited for meeting load variations and peaking requirements and play a crucial role in enhancing system reliability.

1.2.2 Power Scenario in India

India is endowed with a vast hydropower potential. As per the latest assessment 1987-88 (mentioned in CBIP, Publication No. 226, July, 1992) carried out by the Central Electricity authority, exploitable hydropotential in India has been estimated at about 84000 MW at 60% load factor, which can yield an annual energy generation of 400 billion units of electricity and with additional seasonal energy, the total energy potential is about 600 billion units a year. The bulk of potential is available in Arunachal Pradesh i.e. 26,756 MW, followed by Himachal Pradesh 20,000 MW then U.P. 9,744 MW and J & K 7,487 MW. Only 14.5% of this potential is under operation and 7.2% of the potential is under execution. Thus the bulk of the potential amounting to 78.3% is yet to be developed (Table 1.1). The development of hydropower has rather been neglected during last few decades and even today it is not receiving the due attention from the government. It may be useful to note that the developed countries harnessed their hydropower resources before concentrating on other sources of power generation.

Table 1.1 : Status of Development of Hydro Potential

Region	PA* at 60% load factor	PAD*		PUD*		TPD*		Balance potential yet to be developed	
	MW	MW	%	MW	%	MW	%	MW	%
Northern	30,155	3955	13.1	2308	7.6	6263	20.7	23892	79
Western	5,679	1764	31.1	1585	27.9	3349	59.0	2330	41
Southern	10,763	5274	49.0	1098	10.2	6372	59.2	4391	40
Eastern	5,590	868	15.5	726	13.0	1594	28.5	3996	71
N.E.	31,857	314	1.0	305	1.0	619	2.0	31238	9
Total	84044	12175	14.5	6022	7.2	18197	21.7	65847	78.3

PA* = Potential assessed

PAD* = potential already developed

PUD* = Potential under development

TPD* = Total potential developed

1.3 TREND IN HYDROPOWER DEVELOPMENT IN THE COUNTRY

The first hydropower project in India dates back a century ago when a small station with a capacity of 2x65 KW was commissioned in 1897 at Darjeeling. At the time of Independence, the total installed capacity of hydro-power was only 508 MW (out of total capacity of 1362 MW). The total hydropower capacity progressively increased to 20,366 MW by March 94 comprising of 1814 MW in the Central Sector. Despite this growth in the installed capacity of hydropower plants, the share of hydropower in the total capacity, which was around 50% in 1963 has gradually declined to less than 27% upto year 1993-94 (Table 1.2).

Table 1.2 : Share of Hydro Power in Total Installed Capacity

Year	Year installed Capacity	Hydro Capacity	Percentage share of Hydro capacity
1962-63	5,801	2,936	50.6
1969-70	14,102	6,135	43.5
1979-80	28,448	11,384	40.0
1989-90	63,636	18,308	28.8
1991-92	69,070	19,189	27.8
1993-94	76,718	20,366	26.6

In the 7th Plan period (1985-90), hydropower potential of 3827 MW was commissioned as against a target of 5541 MW. The actual development of hydropower capacity in the 7th Plan was 69% as compared to 107% in case of thermal capacity. Even during 1990-91 and 1991-92 the actual achievement in the hydropower capacity had been only 44% and 58% respectively of the target. A hydropower capacity of 9282 MW (3260 MW in Central Sector, 5842 MW in the state sector and 180 MW in the private sector) have been envisaged in the 8th Plan, out of the total of 30,538 MW. As per present indication the actual capacity addition of hydropower plants may just be around 3500 to 4000 MW. As such the percentage share of the hydro capacity is likely to decline further.

1.4 DEFINITIONS

1.4.1 Mini, Micro Hydro Pojects

The classification of mini and micro hydro projects has been varying in different times and in different countries and it is not strictly defined. In Kathmandu, SHP (Small Hydro Power) conference UNIDO defined hydro station capacity below 100 KW as Micro, between 101 and 1000 KW as Mini, but in Hangzhou SHP conference the mini hydro capacity was increased to 2000 KW and the capacity of 2001-10000 KW was defined as small hydropower.

The capacity range of SHP stations is co-ordinated with the development of the national economy, and is especially related to the development of the rural economy and the level of rural electricity consumption. The different methods of classification only reflected the degree of the industrial development of a country at a certain period and proportions of hydropower in the whole power sector of a country and have no fundamental difference. So different countries have different definitions for SHP according to their own features as shown in Table 1.3.

1.5 ADVANTAGES OF MINI-MICRO HYDEL PROJECTS

The potential impact of mini-micro hydropower system is being rediscovered by government agencies and international institutions working in the field of power sector. Mr. Ellis Arm

Table 1.3

Country or Organisation	Micro	Mini	Small
			Unit-KW
UNIDO	100	101-1000	1001-10000
HRC	100	101-500	501-10000
OLADE	50	51-500	501-5000
Bolivia	100	101-1000	-
China	100	101-500	501-25000
Colombia	-	-	>20000
Dominica	100	101-1000	1001-5000
Ecuador	50	51-500	501-5000
Finland	200	201-2000	-
France	500	501-2000	2001-8000
Greec	100	101-1000	1001-15000
India	1000	1001-3000	3001-15000 (as per CEA guidelines)
Indonesia	-	-	>5000
Japan	-	-	>10000
Malaysia	25	25-500	501-5000
Nepal	-	-	>10000
Newzeland	-	10000	10001-50000
Norway	-	-	>10000
Panama	100	101-1000	1001-10000
Peru	5-50	51-500	501-5000
Phillippines	-	-	>15000
Poland	100	101-1000	1001-15000
Romania	-	-	5-50000
Sweden	-	100	101-15000
Thailand	200	201-6000	6001-15000
Turkey	100	101-1000	1001-5000
USA	500	501-2000	2001-15000
Vietnam	50	51-500	501-500
Zimbabe	5-500	501-500	

Strong, former Head of the U.S. Bureau of Reclamation in the book by Fritz has enumerated the advantages of small hydro power which can also be followed for minimicro hydropower are as under:

1. Mini-micro hydro is 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. Mini-micro is essentially non-polluting and releases no heat. Adverse environmental impacts are negligible and for small installations, may be totally eliminated.

3. Mini-micro plants can be designed and built within one or two years time. Licensing requirements are minimal, equipment is readily available, and construction procedures are well known.

4. Mini-micro hydro plants require some type of water control upto and including full regulation of watershed discharge. They are thus an important element in the multipurpose utilization of water resources and can reduce potential flood damage. Where storage facilities are involved flood water is retained and can be better directed to agricultural production, river regulation, improved navigation, fish and wildlife protection, recreation, municipal use and better control of waste water.

5. Mini-micro hydro is a reliable resource within the hydrologic limitations of the site. The relative simplicity of hydraulic machinery makes energy instantly available as needed. Since

no heat is involved, equipment has a long life and malfunctions are rare.

6. Mini-micro hydro in remote areas using relatively simple technology can be a catalyst in mobilizing productive resources and creating enhanced economic opportunities for local residents.

7. Mini-micro hydro 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 overall economy, efficiency and reliability.

8. Mini-micro hydro has an excellent peak power capability. While approximately four units of energy input are required for three units of output, the input is low-cost hydraulic energy and the output is high value electrical energy. In a large electrical system, the alternative for handling peak loads may be utilization or costly expansion of old and relatively inefficient thermal units.

9. Mini-micro hydro technology is well developed and proven, with turbine efficiencies running as high as 90 percent. Small units ranging from a few kilowatts to a few megawatts have been in operation since the turn of the century. While the equipment must be adopted to the specific site for greatest efficiency, its performance will generally live up to the manufacturer's claims.

10. Mini-micro hydro facilities have a long life. As a rule, dams and control works will perform for a century or more with little maintenance.

11. Mini-micro hydro requires few operating personnel and some small scale installations are operated entirely by remote control. Freedom from fuel dependence together with the long life of equipment make hydroelectric power installations resistant to inflation.

12. Mini-micro hydro development can make maximum use of local materials and labour. When compared to thermal facilities, small hydro usually provides more local employment in the construction of civil works.

13. Mini-micro hydro resources remain untapped, especially in the developing countries where less than seven percent of potential has been developed. In some countries the figure is less than two percent.

14. Mini-micro hydro's economic feasibility is improving when compared to other energy sources that use finite 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, mini-micro hydropower has become increasingly desirable.

15. Potential in the industrialised 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.

Mini-micro hydro power stations also possess additional advantage that it does not involve deforestation, submergence or rehabilitation. Thus it can accelerate industrial and economic development of villages by bringing cheap source of energy at every door step and also provide employment to the rural people in various ways.

1.6 SCOPE OF STUDY

In this dissertation, water conductor system only for high head mini-micro hydel scheme has been discussed. The development of mini-micro hydel schemes includes the following main components in water conductor system.

- Diversion arrangement
- Desilting system
- Power canal and tail race
- Forebay/balancing reservoir and intake
- Surge tank
- Penstock.

In this dissertation an attempt is made at detailed design studies stage particularly for water conductor system covering from intake to tail race channel i.e,, how detailed designs of these civil structures should be done and what measures should be taken to make the project cost effective.

CHAPTER - 2

MINI-MICRO HYDRO POWER PROJECT PLANNING AND COST EFFECTIVE TECHNOLOGY

2.1 PLANNING A MINI-MICRO HYDROPOWER PROJECT

Implementing a hydropower project requires a series of sequential steps which can be broadly categorized as planning activities, procurement, construction and start-up. A typical project as described by US Army Corps of Engineers is shown in Fig. 2.1.

Planning studies include all types of investigations performed to determine the desirability of carrying out a project. These studies are initiated when a proposal for a site is deemed worthy of interest and should be completed when construction starts (on Fig. 2.1 Planning studies therefore could extend from time 0 to approximately the 24th month).

Planning studies vary in scope, detail, depth and lead to various decisions and commitments made during the preconstruction period. Following business and international practice, planning studies are generally grouped in three main categories (1) reconnaisance studies, also referred to as appraisal and prefeasibility studies (2) feasibility studies and (3) detailed design studies or definitive site studies.

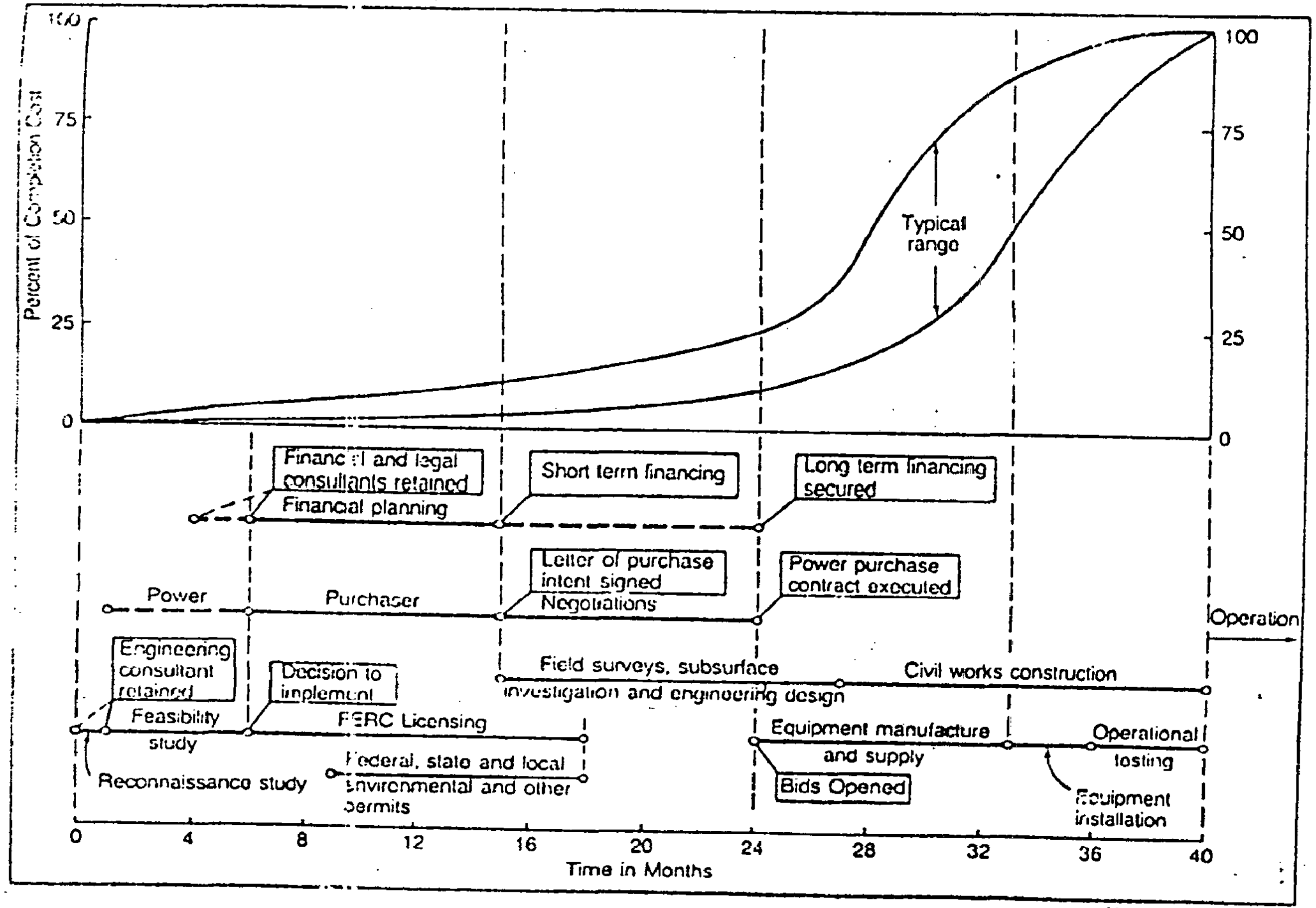


FIG. 2.1 : TYPICAL PROJECT IMPLEMENTATION SCHEDULE AND EXPENDITURE PATTERN

Briefly, the objective of reconnaissance studies is to determine if the project under investigation merits a full feasibility study (months 0 to 1 on Fig. 2.1). Feasibility studies will define the proposed project more concretely and assess its potential as well a determine whether an investment commitment should be made (month 1 to 6 in Fig. 2.1). Definite site studies or detailed design studies are final studies performed between the time of the implementation commitment and the beginning of construction (month 6 to 24 on fig. 2.1). These studies result in permit applications, licensing negotiations, financing arrangements, marketing agreements, engineering designs and specifications.

The budget required to perform all site studies could reach 25 percent of total project cost; reconnaissance and feasibility studies alone may require upto about 10 percent of total cost (as indicated in Fig. 2.1). The sufficient funding must be available to reach a substantiated "go/no go" decision must be kept in mind by planners and investors when initiating a proposed project.

2.2 COST EFFECTIVE MINI MICRO HYDROPOWER PROJECT TECHNOLOGY

Generally, Mini-micro Hydropower technology should be open, appropriate and cost-effective by simplification, standardization and automation which will reduce cost, increase benefits and thus, stimulate the fast development of mini-micro hydro power.

The civil work of Mini-micro hydropower is far less significant than that of large and medium hydros. However, the cost percentage of mini-micro civil works is generally greater than 50% of the project investment and its completion date is controlled by the progress schedule of civil works. As a result, reducing the civil works cost and expediting their construction are still two important objects in mini-micro hydro power construction. In particular, following features are quite significant for the design of mini-micro hydro project works.

- The investment of mini-micro is generally limited. However, this limited investment is still an overburden to developers in developing countries. Owing to the limited investment and the small size of structures, there is neither financial capability nor necessity to spend much money on the cost of some items, such as geologic exploration, access of the site, hydraulic modelling, structural testing etc.
- Utilization of local materials is especially important to the design of civil works.
- Attraction of the local people to participate in the mini-micro hydro project construction is very important and local skill and appropriate technology to utilize maximum amount of locally available construction material should be specially considered in the civil works design. Furthermore, mini-micro hydro power stations can neither be scaled down from large or medium hydro projects nor be said of as their simplification. On the contrary, a mini-micro hydro-power engineer should be multi-displined.

CHAPTER - 3

DIVERSION ARRANGEMENT

3.1 GENERAL

The diversion structure should be as simple and cost effective as possible. The diversion structures may be of following types.

(i) Trench type weir (ii) Solid boulder structure

(iii) Dams

In view of the fact that trench type weir is most popularly used for mini-micro hydropower, therefore, after a brief description of the said three types of diversion arrangements, detailed design of trench weir is covered in para 3.5 subsequently followed by separate emphasis on sediment problems in para 3.6.

3.2 TRENCH TYPE WEIR

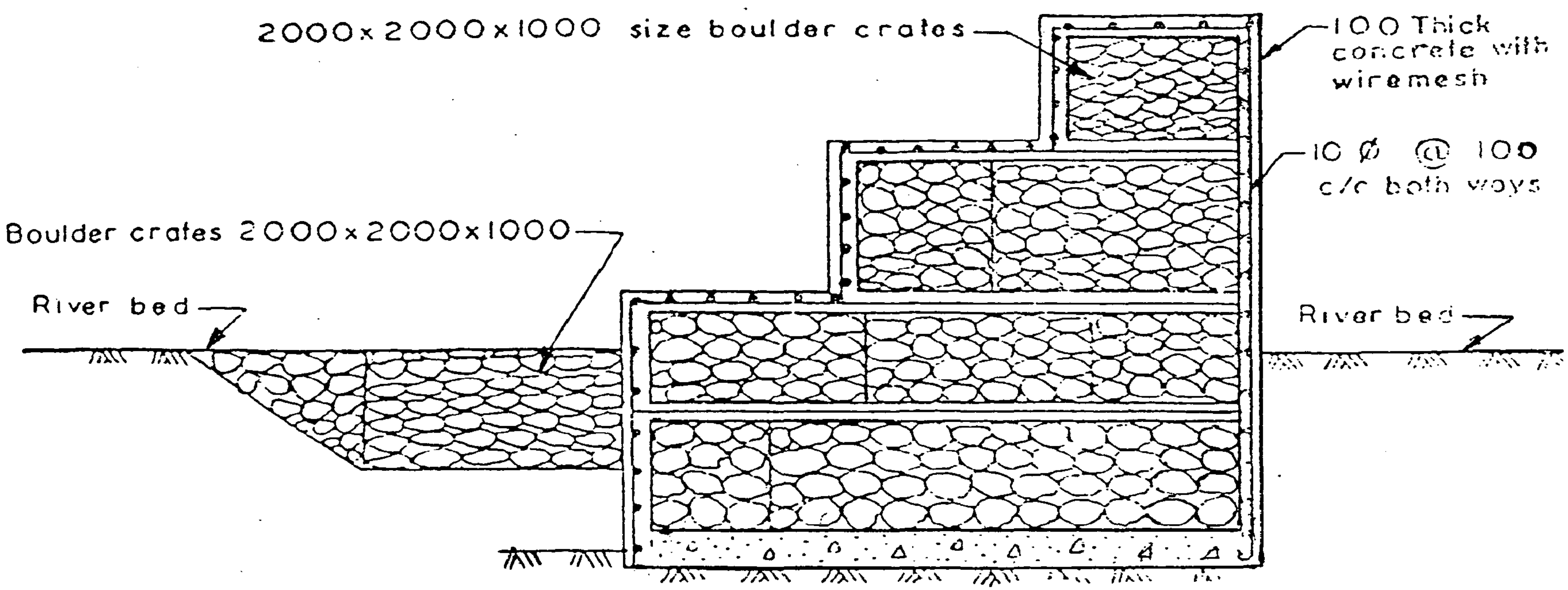
Most of the hill streams carry a heavy charge of sediments and boulders during high floods, since their bed slopes are steep and banks erodible. In addition, full grown trees and timber logs, 10-15 m long and about 1.0 m dia are some times seen floating in these streams during floods. It is not possible to create large storage within economical means in these streams in view of steep gradients and narrow gorges. Any raised structure constructed across these streams also runs the risk of both being hit by rolling boulders and floating logs, beside the storage getting soon filled up with sediments and boulders. Gated barrages on such stream (with catchment prone to cloud bursts) are also not economical due to high flood intensity necessitating provision of large water way and big size gates. Trench type weirs have, however, proved to be quite successful under such situations.

Drop type trench weir is a simple structure comprising a trash rack at stream's bed level and a trench laid across the stream entirely below its bed. The water drops into the trench through openings in the trash rack and is carried further to the main water conductor of the power generation scheme. The trash-rack is given a slope in the direction of flow and the clear opening between the bars is usually kept as 30 mm.

3.3 SOLID BOULDER STRUCTURE

Boulder type structure is suitable where boulders are available in the river and rock is encountered in the river bed within one metre depth. If solid boulder structure type weir is provided in non-rocky foundation, maintenance and repair of damages due to scour may create problems. The diversion weir should be capable of diverting all the lean season flows and structure should be reasonably safe during monsoon floods.

The solid boulder structures are constructed by encasing boulders in G.I. weirs, piling them up to form weir and coating the same with rich cement concrete reinforced with say 10 mm dia, mild steel bars, 100 mm centre to centre on the face. Fig. 3.0 shows the detail of boulder weir.



Note : All dimensions are in mm

FIGURE 3.0 : DETAILS OF BOULDER WEIR

3.4 DAMS

3.4.1 Earthen Dams

Among dams built of local materials, earth dam are the most widely used dam type by reason of loose requirement of foundation conditions, full use of local materials, large area of construction surface, and simple technique of construction. The modern development of soil mechanics leading to lowering the qualification requirement of materials for earth dams, also attributes to the large percentage of earth dams used as the water retaining structures for mini-micro hydropower stations.

In the initial stage of small hydropower development with only small size installed machine units, the earth dams were predominantly of homogeneous type. Later, the rapid progress in small hydro development necessiated the construction of larger and larger projects. Such projects have other than homogeneous earth dams, like central core, sloping core and also multiple-zoned dams.

3.4.2 Overflow Earth Dams

In china overflow earth dams are constructed to avoid the excavation of spillways on the steep abutments. Such dams can retain the water and also discharge the flood over the dams, having the benefit of reducing the cost of civil works.

The spillway surface of such dams are provided with protective facings against erosion. The protective facing can be soil-cement facing, stone masonry facing, concrete facing or asphalt facing. The first two types of facings are cheap in construction cost, and so they are widely used to economise the cost of civil works. For higher dams concrete or asphalt concrete facings are used exclusively to acquire a higher resistances agaist erosion.

In Taoyuan country of Hunan province (China), Hongzing sloping core earth dam is 20.2 m high overflow dam. A power house with an installed capacity of 150 kw is located below the dam. The whole crest length of 50 m permits overflowing. To resist erosion of water flow, the overflow surface is protected by concrete facing with a thickness of 30 cm. Beneth the concrete facing are cushioning layers consisting of dry laid stone masonry and sand-gravel. The masonry is 100 cm thick. The downstream end of the spillway is provided with flip bucket for energy dissipation. After the completion of the dam, overflowing took place many times with the maximum overflowing water depth at the crest being 0.72 m. The dam performed normally.

3.4.3 Stone Masonry Dams

Stone masonry dams are constructed in mountainous regions where rock materials are in greater quantity than soil. Dams of this type have less amount of volume as compare with earth dams. They are simple in construction and can facilitate overflowing to the advantage of handling flood during construction as well as during operation.

In china, stone masonry dams are quite commonly found in the mountanous regions, most of these are of gravity type, which posses less rigid requirement of topographical and geological conditions and are more convenient in the construction than such dams of other types.

3.4.4 Stone Masonry Arch Dams

Stone masonry arch dam is one of the dam types using advanced technique. Such dam is characterised by small dam volume, good stress condition and high safety, factor. Accordingly, stone masonry arch dam is one of the common types of dams adopted in the development of small hydropower. Stone masonry arch dams are provided in hilly area showing V-shaped valley.

3.4.5 Roller Compacted Concrete Dams

The first rollcrete gravity dam (Kengkou dam) built in 1986 in Datian country, Fujion province (China) is 56.8 m high and with a crest length of 122.5 m. The rollcrete construction of Kengkou dam is characterised by high flyash content in the heating zone of the dam. These dams are not provided with any joint in the dam body in coarse of construction. The layer thickness of loose mixture for the heating area is kept as 56 cms and for the interfacing area 37 cms and these compacted to a thicknesses of 50 cm and 34 cm respectively. Vibratory roller are used for compacting the rollcrete. Compared with conventional concrete, the quantity of cement per cubic metre for Kengkou rollcrete dam was reduced by 44%, the unit cost of rollcrete was reduced by about 12% and the period of construction was also shortened.

3.4.6 Rubber Dams

The rubber dam is composed of rubber dam bag, concrete base, side walls and water charge and discharge system. The dam bag is a composite of both canvas and rubber hot pressed in high temperature with chloroprene rubber and several layers of acrylic fibers canvas, and anchored on its base and side walls. The bag is charged with water and air to form the dam. It can be filled and defilled with the help of air pump and pump house will be located outside. The main advantages of rubber dam are as follows :

1. Compared with movable gate dam with equivalent scale the cost of rubber dam can be decreased by 30-70%
2. Its structure is simple and easy to instal on the top of a spillway without gates.
3. It is convenient and reliable in operation and has sound antivibration performance.
4. It can raise its regulation capability and increase the reservoir capacity which can promote benefit.

3.5 DETAILED DESIGN OF TRENCH WEIR

3.5.1 Hydraulic Design

Width of the trash track along the stream is determined from an equation connecting parameters like design discharge, specific energy, ratio of opening area to the total area of rack surface, coefficient of discharge through the openings and width of water way. The flow through the trench as water is gradually with-drawn from the stream is a case of spatially varied flow with increasing discharge. Size and slope of trench is determined as per the hydraulic criteria applicable for design of a side channel spillway which assumes that all the energy of the over falling water is dissipated in turbulence and that the slope in the trench must be sufficient to accelerate the over falling water in the direction of flow down the trench.

The analysis of flow profile for the case of decreasing discharge gives the relationship for determining width of trash rack required for complete withdrawal of proposed discharge through the rack. Since the trash rack for such structures in the bed of stream is composed of parallel bars, the direction of flow through the rack openings is nearly vertical and the following equation is used for determining the width of trash rack.

$$B = \frac{Q}{\varepsilon_1 \varepsilon_2 C L \sqrt{2gE}} \qquad (3.1)$$

where,

B = width of trash rack/trench at bed level of stream

Q = discharge proposed to be withdrawal from the stream

ε_1 = ratio of the opening area to the total area of rack surface

ε_2 = percentage of opening remaining in the trash rack clogged.

C = co-efficient of discharge through the openings (0.435 for a grade of 1 in 5 to 0.497 for a horizontal slope of the rack of parallel bars)

L = length of rack (water-way)

g = acceleration due to gravity (9.81 m/sec^2)

E = specific energy at any section of stream over the rack

$= \left[\frac{Q}{C_d L}\right]^{2/3}$, If the entrance to the reach of the rack is regarded as a broad crested weir and C_d = coefficient of discharge for broad crested weir.

The principle applicable for the design of lateral spillway channels can be used for determining the bed profile and size of the trench as both have spatially varied flow with increasing discharge. The simple new design developed by Jullian Hinds can be adopted for evolving the parameters of the trench as given below.

Assuming uniform inflow into the trench in the entire length of waterway, total discharge at a section distance 'x' from the upper end of the trench is given by

$$Q = qx \qquad (3.2)$$

where,

Q = total flow at the point

q = inflow per metre length of waterway

Velocity = discharge relationship can be expressed by the equation

$$V = ax^n \qquad (3.3)$$

where 'a' and 'n' are arbitrary constants and 'V' and 'x' denote average velocity in direction of channel and distance from the upper end of the trench respectively (value of n is such that if n = 0.5, the water surface curve is straight and the channel bottom will be concave starting at crest level. If n is greater than 0.5 the water surface curve will be convexed upwards. If n is exactly 1.0, the water surface curve will be parabola and bottom will be a parallel curve. if n is between 0.5 and 1.0 the bottom line will start at crest level from upper end of trench).

The theoretical ordinate 'Y' of the water surface curve measured downwards from the datum line (elevation of centre of trash track) is given by the equation.

$$Y = \frac{n+1}{n} h_v \qquad (3.4)$$

where,

h_v = Theoritical velocity head for a point at the end of the trench

$$h_v = \frac{n}{n+1} \cdot \frac{A}{2T} \qquad (3.5)$$

where,

A is the area and

This the top width of water prism at the end of the trench. Substituting value of h_v from equation (3.5) in equation (3.4)

$$Y = \frac{A}{2T} \qquad (3.6)$$

(equation 3.5 and 3.6 are applicable only for the end section of the trench) as an example the design of trench weir for Kush

Hydroelectric Project (Arunachal Pradesh) has been done which is as follows.

3.5.2 Design Example

As an illustration the following example covers the detailed hydraulic design of trench weir for Kush Mini HEP 2 MW, (Arunachal Pradesh).

3.5.2.1 Hydraulic design

Design discharge = 'Q' = 1.72 m^3/sec. This includes power draft, 10% discharge for evaporation and seepage in the channel and 20% discharge for continuous flushing of sediment.

Slope of racks along the flow in the river = 1 in 8.

Length of rack (waterway) 'L' = 20 m

Ratio ε_1 = 0.3375 (The racks consists of 50 mm dia bars spaced at 30 mm clear spacing. Ratio of openings to total area = 30/30+50 = 0.375, value adopted = 0.9 x 0.375 = 0.3375 assuming 10 percent reduction for frame work).

Ratio ε_2 = 0.5(50% of the openings in the trash rack have been assumed to be clogged with pebbles and trash carried by the stream).

C = 0.45825 (for a rack slope of 1 in 8).

Calculation of C :

For horizontal i.e. 'O' slope 'C' = 0.497

for 1 in 5 slope i.e. 0.2 slope 'C' = 0.435

i.e. for a difference of 0.2 decrease in value of C = 0.062

For a difference of $\frac{1}{8}$ i.e 0.125 decrease in

value of C = $\frac{0.062}{0.2}$ x 0.125 = 0.03875

So for a s;p[e pf 1 in 8 value 'C' = 0.497 - 0.03875 = 0.45825

C_d = Coefficient of discharge = 1.53

for broad crested weir

$$\text{Specific energy } E = \left[\frac{Q}{C_d L}\right]^{2/3}$$

$$= \left[\frac{1.72}{1.53 \times 20}\right]^{2/3} = 0.1467 \text{ m}$$

$$\text{Width of rack 'B'} = \frac{Q}{\varepsilon_1 \varepsilon_2 \, CL \sqrt{2gE}}$$

$$= \frac{1.72}{0.3375 \times 0.5 \times 0.45825 \times 20 \sqrt{2 \times 9.81 \times 0.1467}} = 0.6556$$

Say 0.66 m

In the above analysis a uniform intensity of discharge for the entire length of the water way has been assumed. For streams with less discharge and considerable width it is desirable to be liberal in providing the rack width to ensure desired withdrawal as there could be considerable variation in intensity of discharge along the length of the rack i.e. width of the stream. So provide at least 2.5 time the calculated width.

So take base width = 1.75 m.

The width of the trash rack as obtained above fixes one of the dimensions of the trench i.e. top width. The bed level and water surface profile in the trench can then be worked out as per the simple relationship given by Julian Hinds as applicable for the discharge carrier of a side channel spillway. The application of this method is used here as under for rectangular and trapezoidal sections.

(a) For Rectangular Section of Trench

Length of the rack (water way)	L	= 20 m
Design discharge		= 1.72 cumec
Width of trench	T	= 1.75 m
Section		= Rectangular
Let water depth at any section along the channel		= d
Then area	A	= Td = 1.75 d

For the section at the end of the trench.

velocity head $h_{v20} = \frac{n}{n+1} * \frac{A}{1.75T}$

Choosing value of n, an arbitrary constant as 0.5

$$h_{v20} = \frac{0.5}{1.5} \times \frac{Td}{1.75xT} = \frac{d}{5.25}$$

$$Q_{20} = A\sqrt{2gh_v} = 1.75d \sqrt{2 \times 9.81 \times d/5.25} = 3.383\ d^{3/2}$$

We can thus find out the depth of water in the trench at the exit end. Since discharge at this section is 1.72 cumec.

So putting value of Q_{20} in the above equation

$1.72 = 3.383\ d^{3/2}$

$d^{3/2} = 1.72/3.383 = 0.5084$

$d = 0.637$ m

Velocity at the exit end = $\frac{1.72}{1.75 \times 0.637}$ = 1.5429 m/sec

Velocity - distance relation along the trench is given by

$$V = ax^n$$

or

$$a = \frac{v}{x^n} = \frac{1.5429}{20^{0.5}} = 0.345$$

Therefore $V = 0.345*x^{0.5}$ (3.7)

The ordinate 'Y' of water surface curve measured downwards is given by

$$Y = \frac{n+1}{n} \; h_v$$

$$= 3 \text{ hv [for n = 0.5]}$$

$$= \frac{3V^2}{2g} \qquad (3.8)$$

The bed levels and water surface profile along the trench can then be found from equation (3.7) and (3.8) and these are tabulated in Table 3.1 and plotted in Fig. 3.1 by using Lotus 123R3 & HG packages on computer.

(b) For Trapezoidal Section of Trench (Ref. Fig. 3.4)

To take rough idea of depth following calculation will be done.

Length of rack	L = 20 m
Discharge	Q = 1.72 cumec
Section	= Trapezoidal
Taking base width (same as for rectangular section)	= 1.75 m

TABLE 3.1: BED LEVELS AND WATER SURFACE PROFILE FOR TRENCH WEIR (RECTANGULAR SECTION) OF MINI HYDRO ELECTRIC PROJECT (ARUNACHAL PRADESH)

S.NO	X (M)	Q=qX cumec	X**n n=0.5	V=aX^n a=0.345	A=Q/V	d=A/1.75	Y=n+1/n*V^2/2(	d+Y	BED LEVEL OF CHANNEL 799.828-(d+Y)	WATER SURFACE ELEVATION 799.828-Y
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	799.828	799.828
2	2.000	0.172	1.414	0.488	0.353	0.201	0.036	0.238	799.590	799.792
3	4.000	0.344	2.000	0.690	0.499	0.285	0.073	0.358	799.470	799.755
4	6.000	0.516	2.449	0.845	0.611	0.349	0.109	0.458	799.370	799.719
5	8.000	0.688	2.828	0.976	0.705	0.403	0.146	0.548	799.280	799.682
6	10.000	0.860	3.162	1.091	0.788	0.450	0.182	0.632	799.196	799.646
7	12.000	1.032	3.464	1.195	0.864	0.493	0.218	0.712	799.116	799.610
8	14.000	1.204	3.742	1.291	0.933	0.533	0.255	0.788	799.040	799.573
9	16.000	1.376	4.000	1.380	0.997	0.570	0.291	0.861	798.967	799.537
10	18.000	1.548	4.243	1.464	1.058	0.604	0.328	0.932	798.896	799.500
11	20.000	1.720	4.472	1.543	1.115	0.637	0.364	1.001	798.827	799.464

Discharge per unit length of trench 'q'=1.72/20=0.086 cumecs

KUSH TRENCH WEIR

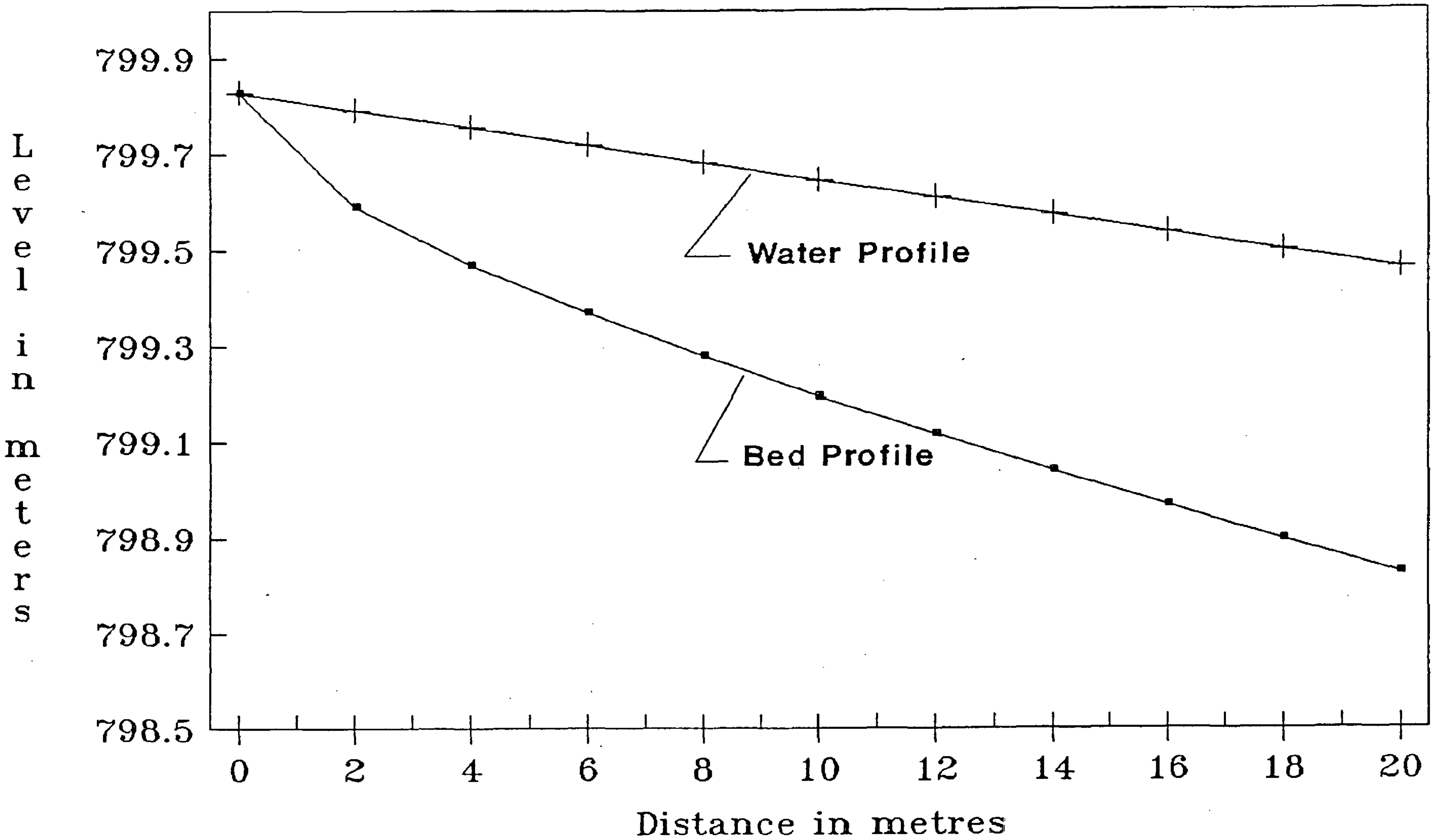


FIG.NO.3.1: WATER & BED SURFACE PROFILES
(RECTANGULAR SECTION)

Side slope = 2 horizontal : 1 vertical

$T = 1.75 + 2 \times 0.5Y$

$Q = AV$

$$Y = \frac{n+1}{n} h_v = \frac{0.5+1}{0.5} \times \frac{V^2}{2 \times 9.81}$$

$$V = \sqrt{\frac{Y}{0.1529}}$$

$$A = \frac{(1.75 + Y) + 1.75}{2} \times Y = \frac{3.5 + y^2}{2}$$

$$Q = AV = \left(\frac{3.5\ y+y^2}{2} \right) \sqrt{\frac{y}{0.1529}}$$

By trial and error i.e. $Y = 0.5$ m, $Q = 1.80$ m

So take depth of trench weir = 0.5 m

$\therefore$ To width = $1.75 + 2 \times 0.5 \times 0.5 = 2.25$ m

Average width = $T_{av} = \frac{1.75 + 2.25}{2} = 2.00$ m

Let the water depth at any section along the channel = d

Then area $A = Td = 2d$

For the section at the end of trench velocity head

$$h_{20} = \frac{n}{n+1} \cdot \frac{A}{2T}$$

Choosing value of n, an arbitrary constant as 0.5

$$h_v = \frac{0.5}{1+0.5} \times \frac{2d}{2T} = d/6$$

$$Q_{20} = A \sqrt{2gh_v} = 2d \sqrt{2\text{x}9.81 \text{ x } d/6} = 3.6166\ d^{3/2}$$

Now we an find out the depth of water in the trench at the exit end since discharge at this section is 1.72 cumec.

Substituting the value of Q_{20} in the above equation

$$1.72 = 3.6166\ d^{3/2}$$

$$d^{3/2} = 0.4756$$

$$d = 0.609\ m$$

Velocity at the exit end = $\dfrac{1.72}{2 \times 0.609} = 1.412$ m/sec

Velocity distance relation along the trench is given by

$$V = ax^n$$

$$a = \frac{V}{x^n} = \frac{1.412}{20^{0.5}} = 0.3157 \qquad (3.9)$$

$$\therefore V = 0.3157 * x^{0.5}$$

The ordinate Y of water surface curve measured downwards from the datum is given by

$$Y = \frac{n+1}{n} h_v$$

$$= \frac{0.5 + 1}{0.5} \quad \frac{V^2}{2g} = (n = 0.5)$$

$$= \frac{3V^2}{2g} \qquad (3.10)$$

The bed level and water surface profile along the trench can then be found from equation (3.9) and (3.10) and these are tabulated in Table 3.2 and plotted in Fig. 3.2 by using Lotus 123R3 & HG packages on computer.

TABLE 3.2 : BED LEVELS AND WATER SURFACE PROFILE FOR TRENCH WEIR (TRAPEZOIDAL SECTION) OF KUSH MINI HYDRO ELECTRIC PROJECT (ARUNACHAL PRADESH)

S.NO	X (M)	Q=qX CUMEC	X**n n=0.5	V=aX^n a=o.3157	A=Q/V	d=A/2	Y=n+1/n*v^2/2g	d+Y	BED LEVEL OF CHANNEL 799.797-(d+Y)	WATER SURFACE ELEVATION 799.797-Y
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	799.797	799.797
2	2.000	0.172	1.414	0.446	0.385	0.193	0.030	0.223	799.574	799.767
3	4.000	0.344	2.000	0.631	0.545	0.272	0.061	0.333	799.464	799.736
4	6.000	0.516	2.449	0.773	0.667	0.334	0.091	0.425	799.372	799.706
5	8.000	0.688	2.828	0.893	0.770	0.385	0.122	0.507	799.290	799.675
6	10.000	0.860	3.162	0.998	0.861	0.431	0.152	0.583	799.214	799.645
7	12.000	1.032	3.464	1.094	0.944	0.472	0.183	0.655	799.142	799.614
8	14.000	1.204	3.742	1.181	1.019	0.510	0.213	0.723	799.074	799.584
9	16.000	1.376	4.000	1.263	1.090	0.545	0.244	0.789	799.008	799.553
10	18.000	1.548	4.243	1.339	1.156	0.578	0.274	0.852	798.945	799.523
11	20.000	1.720	4.472	1.412	1.218	0.609	0.305	0.914	798.883	799.492

Discharge per unit length of trench 'q'=1.72/20=0.086 cumec

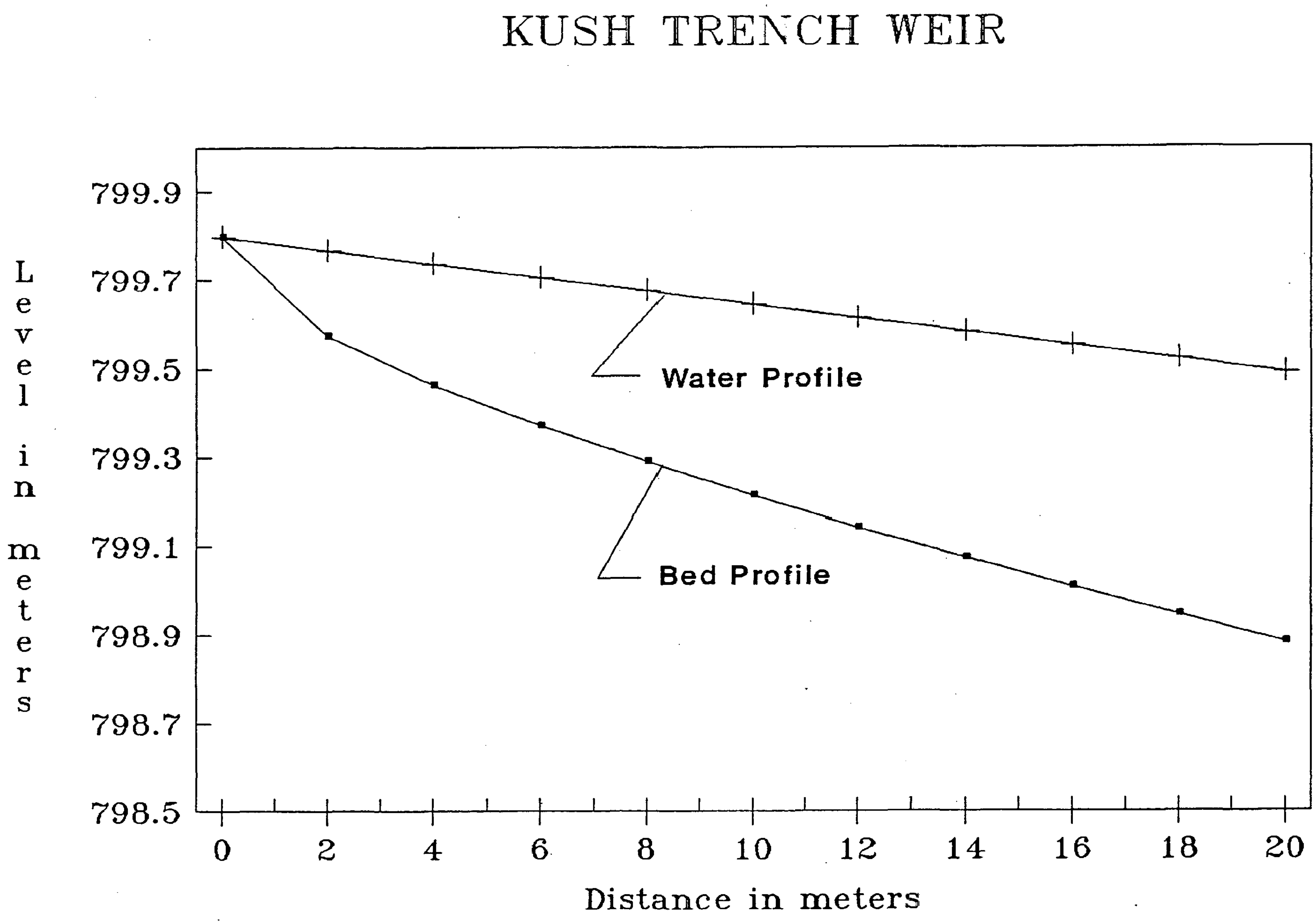


FIG.NO.3.2: WATER AND BED SURFACE PROFIL
(TRAPRZOIDAL SECTION)

3.5.2.2 Scour depth

Following formulae are used to determine the scour depth

Silt factor $= f = 1.76 \sqrt{m}$ (3.11)

m = mean diameter of particles

Scour depth 'R' in metres $= 1.35 \left(\frac{q^2}{f} \right)^{1/3}$ (3.12)

where, q = Intensity of flood in m^3/sec/mtr.

For design flood discharge of 226 cumec

$$q = \frac{226}{20} = 11.3 \text{ cumec/sec/mtr.}$$

Assume that average 3" (76.2 mm) dia particles are present

$f = 1.76 \sqrt{76.2} = 15.36$ say 15

Scour depth 'R' $= 1.35 \left(\frac{11.3^2}{15} \right)^{1/3} = 2.76$ m

Recommended scour depth $= 1.25$ R $= 1.25 \times 2.76 = 3.45$ m

3.5.2.3 Check for looseness factor

Maximum stable width of stream is given after refering IS. 6966-1973 by following equation.

$P = 4.83\ Q^{1/2}$ (3.13)

(where Q = design flood)

$P = 4.83\ (226)^{1/2} = 72.61$ m

Overall length of weir = 20 metre

$\therefore$ Looseness factor $= \frac{20}{72.61} < 1$

3.5.2.4 Depth on u/s of weir

Maximum carrying capacity of the trench weir

$$Q_{max} = B\varepsilon_1\ \varepsilon_2\ CL\ \sqrt{2gE}$$

$$= 1.75 \times 0.3375 \times 0.5 \times 0.4525 \times 20\ \sqrt{2 \times 9.81 \times 0.1467}$$

$$= 4.59\ m^3/sec$$

$$\text{Depth on u/s of the weir} = \sqrt[3]{\frac{(Q_{max}/L)^2}{g}} = \sqrt[3]{\frac{\frac{(4.59)^2}{20}}{9.81}} = 0.175\ m.$$

3.5.2.5 Stability analysis of trench-weir structure (rectangular section ref. fig. 3.3).

Critical Case :

When the entire discharge of the stream is conveyed through the trench i.e, "No flow condition in the downstream".

i) Submerged soil mass upto level of trench weir at upstream

ii) Water level is upto water surface profile for the above case

iii) No soil mass in the d/s of the trench.

Known parameters :

i) Saturated wt. of soil mass $\gamma_{sat} = 1.8\ t/m^3$

ii) Submerged wt. of soil $r_{sub} = \gamma_{sat} - \gamma_{w}$

$$= 1.8 - 1 = 0.8\ t/m^3$$

iii) Coefficient of friction between two surfaces $\mu = 0.7$.

Detailed calculations are shown in Table 3.3.

KUSH HYDRO-ELECTRIC PROJECT.(ARUNACHAL PRADESH)

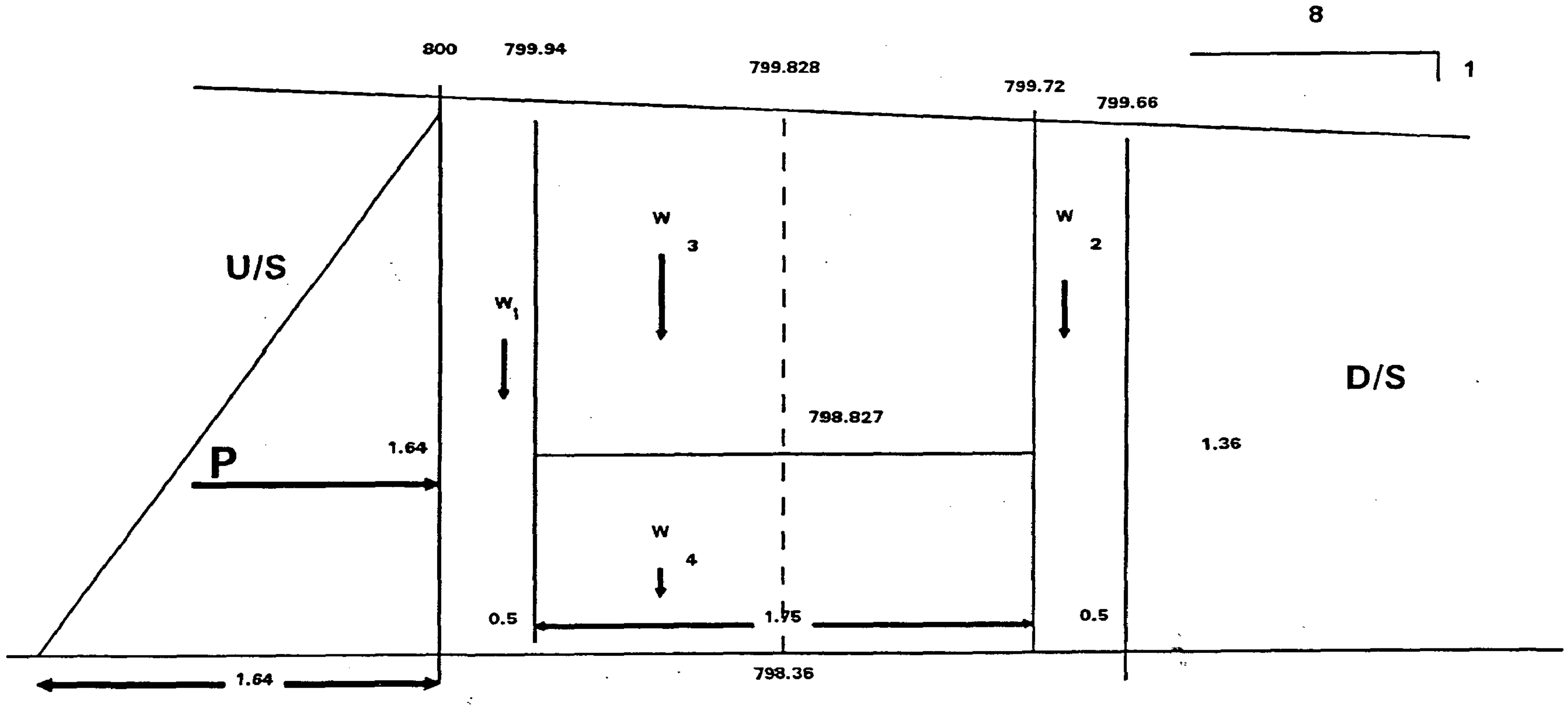


FIG. NO:3.3 STABILITY ANALYSIS OF TRENCH WEIR

RECTANGULAR SECTION

Table 3.3

Name of Forces	Designation	Magnitude in tonnes	Lever arm in m	Moments about toe in tm
Vertical Forces	W_1	$0.5 \times 1.61 \times 2.4 = 2.004$	2.50	5.01
	W_2	$0.5 \times 1.33 \times 2.4 = 1.668$	0.25	0.417
Weight of water	W_3	$1.003 \times 2 \times 1.0 = 2.006$	1.375	2.758
Weight of concrete	W_4	$1.75 \times 0.467 \times 2.4 = 1.961$	1.375	2.696
		$\Sigma V = 7.639$	$\Sigma M\ (+)=$	11.382
Horizontal water pressure	P	$1 \times \frac{1.64^2}{2}$ 1.345	0.547	0.736 (-)
Horizontal Submerged Soil Pressure	P'	$0.7 \times 0.8 \times \frac{1.64^2}{2} = 0.753$ $(\mu \times \gamma_{sub} \times h/2^2)$	0.547	0.412 tm (-)
Uplift pressure	U	$\frac{2.75 \times 1.64 \times 1}{2} = 2.225$	1.83	4.127 tm (-)
		$\Sigma H = 2.978$	$\Sigma M(-) =$	4.539

ΣM = Net (+) Moment = 11.382 - 4.539 = 6.843 tm

ΣV = 7.639 tonne

ΣH = 2.978 tonne

M_{+ve} = 11.382 tm

M_{-ve} = 4,539 tm

(i) $(\bar{x}) = \frac{\Sigma M}{\Sigma V} = \frac{6.843}{7.639} = 0.896$ m

Eccentricity $e = b/2 - \bar{x} = \frac{2.75}{2} - 0.896 = 0.479$ m. $< \frac{b}{6}$

As 'e' is less than b/6 so tension will be developed any where in the structure.

(ii) Factor of safety against sliding $= \frac{\mu \Sigma V}{\Sigma H} = \frac{0.7 \times 7.639}{2.978} = 1.8 > 1.$

Hence safe

NOTES

1. All dimensions are in millimetre and levels in metre.

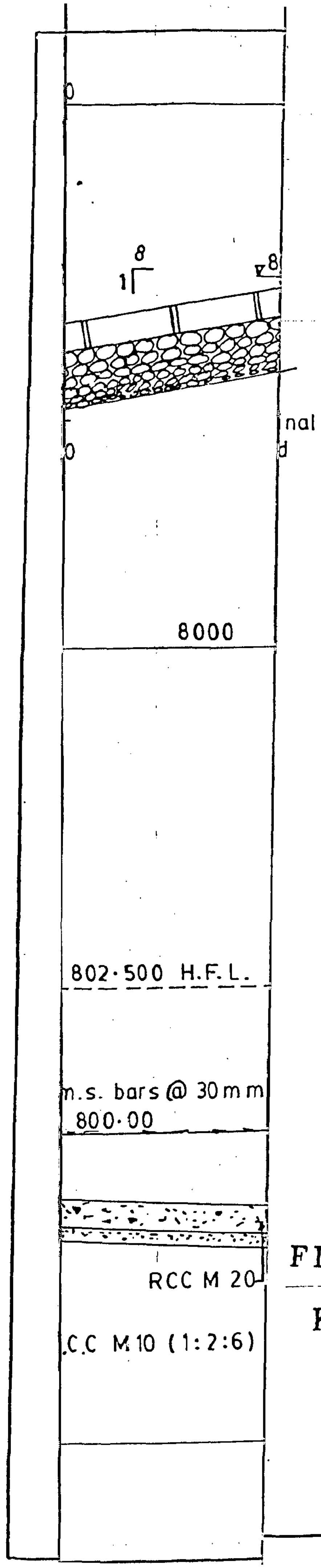
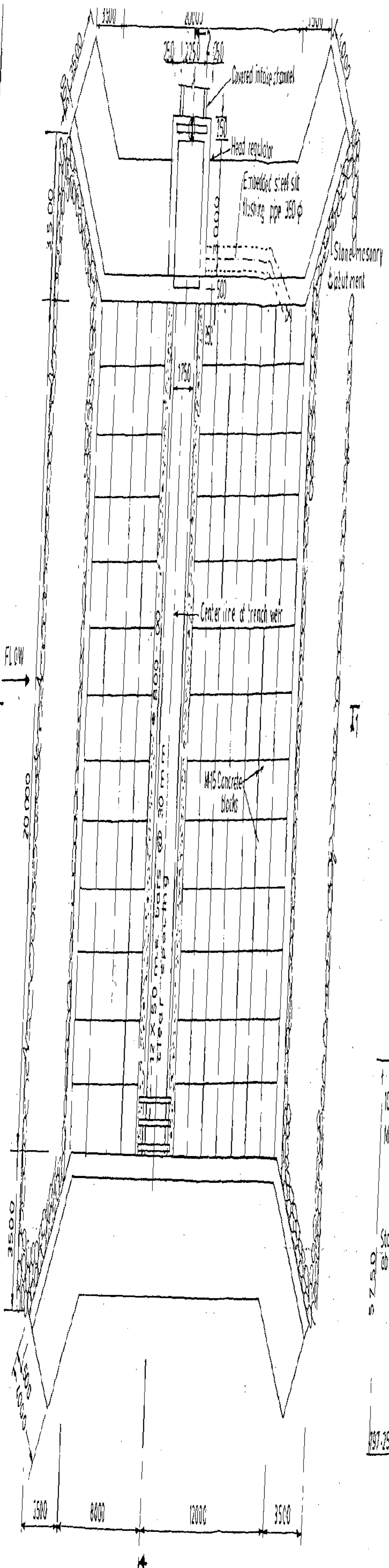


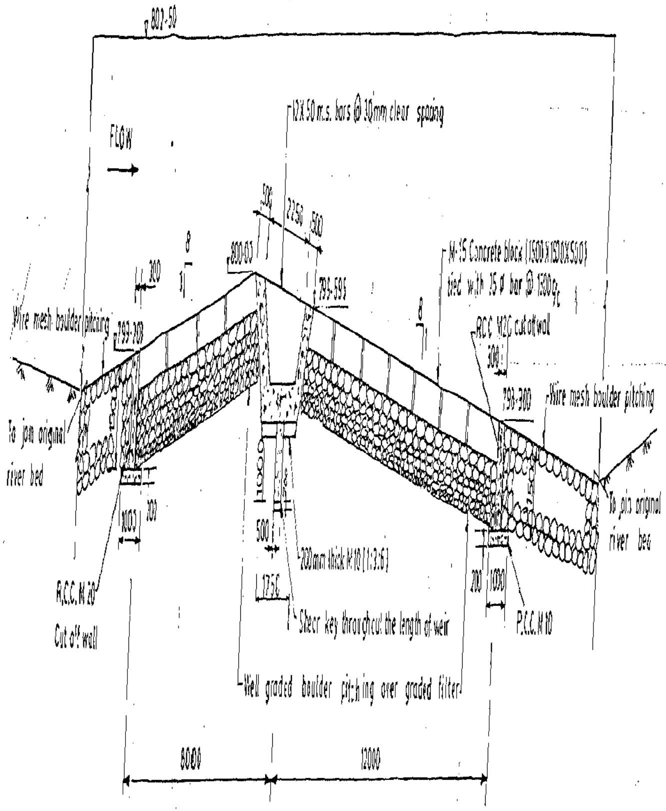
FIG. 3.4 : TRENCH WEIR

KUSH MINI HYDRO ELECTRIC PROJECT

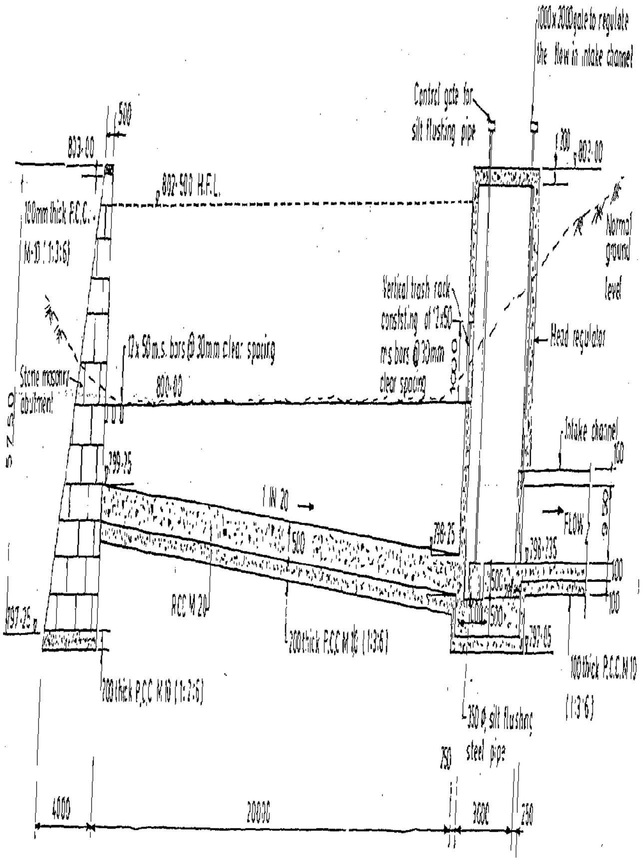
NOTES
1. All dimensions are in millimetre & levels in metre.



PLAN
(1:150)



SECTION 1-1
(1:100)



SECTION 2-2
(1:100)

FIG. 3.4 : TRENCH WEIR

KUSH MINI HYDRO ELECTRIC PROJEC

(iii) Factor of safety against overturning = $\frac{\Sigma M(+)}{\Sigma M(-)} = \frac{11.382}{4.539}$

$= 2.51 > 1.5$ (safe)

(v) Maximum and minimum vertical stress

Maximum vertical stress at toe $P_{max} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right]$

$$= \frac{7.639}{2.75}\left[1+ \frac{6x0.479}{2.75}\right]$$

$$= 5.68 \ t/m^2$$

Min vertical stress $= \frac{7.639}{2.75}\left[1- \frac{6x0.479}{2.75}\right] = (-)0.125 \ t/m^2$

It should be less than bearing capacity of soil.

Similarly stability analysis for trapezoidal section can be done.

3.6 SOME SEDIMENT PROBLEMS WITH TRENCH WEIRS

With special reference to Andhra HEP and Binwa HEP in Himachal Pradesh.

3.6.1 Andhra Hydroelectric Project

Andhra HEP is in Shimla district and is run off the river scheme with a trench weir on Andhra Khad, a surface desilting tank, a storage reservoir with 45450 m^3 respectively, a tunnel 641 km long, a surge shaft of 8.0 m dia and a single penstock 641 m long with variable dia of 1.5 m, 1.4 m and 1.2 m in different reaches. Gross head is 330 m and design discharge of 6.5 m^3/sec. The project was commissioned in Sept. 1987. Layout of intake area and trench weir details are shown in Fig. 3.5 and 3.6.

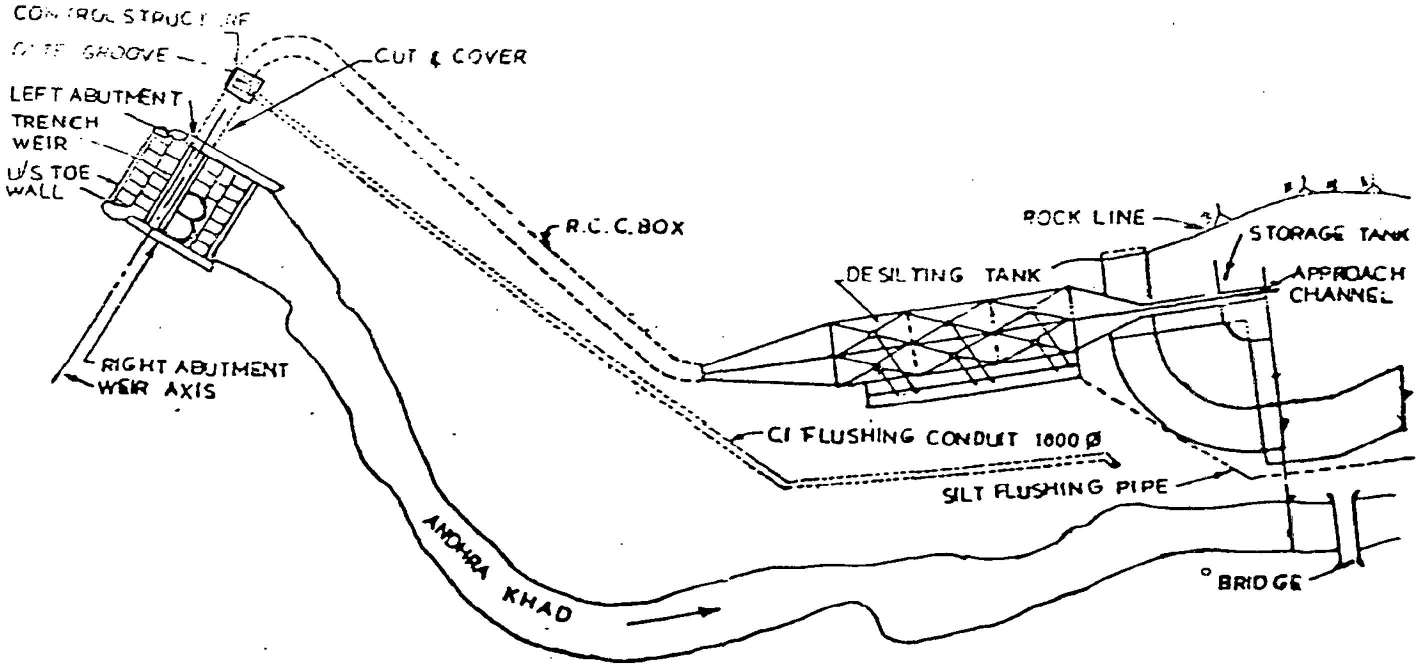


FIG. 3.5 LAYOUT PLAN OF WATER-CONDUCTOR SYSTEM
ANDHRA HYDEL PROJECT (H.P)

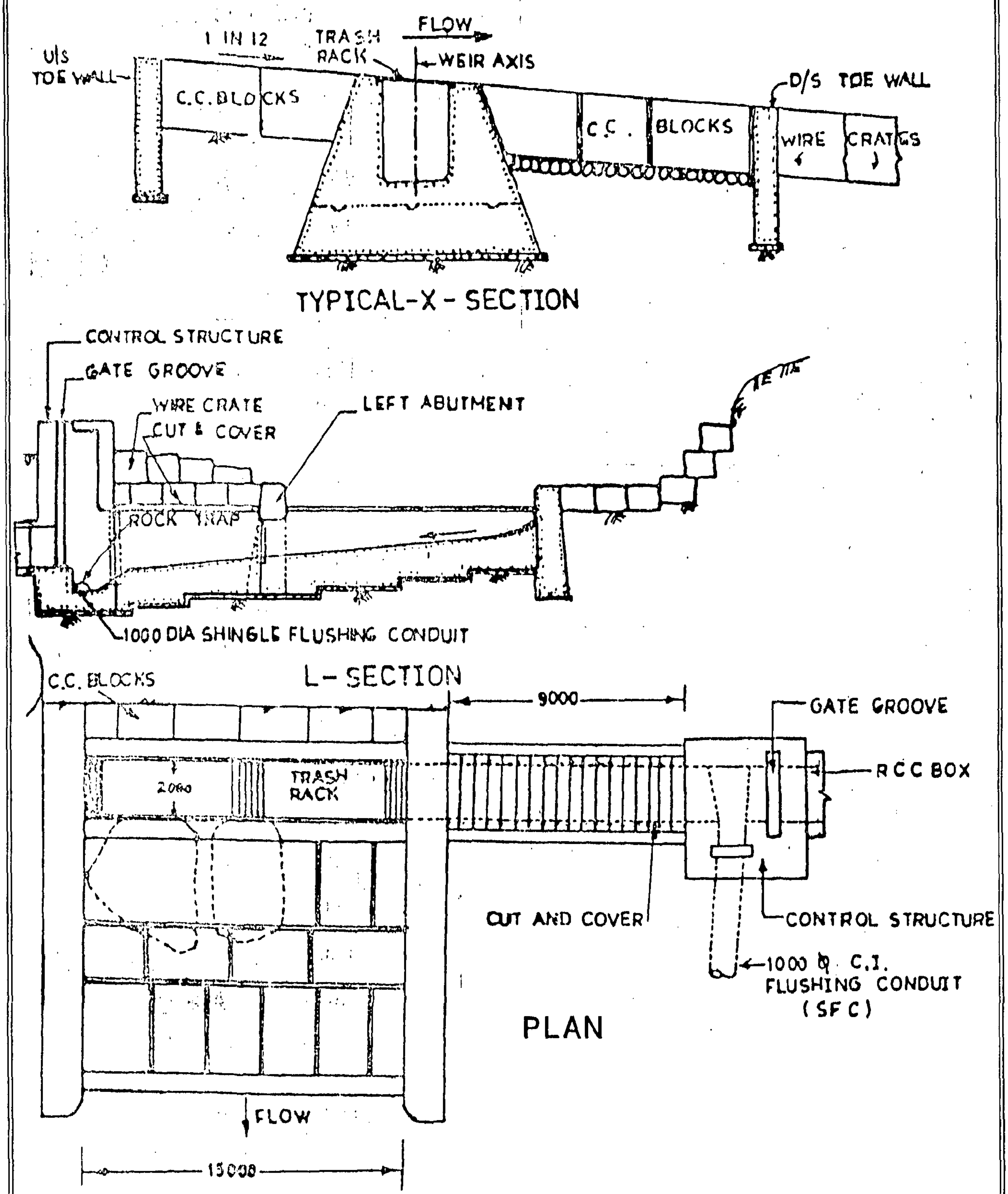


FIG. 3.6 ANDHRA TRENCH WEIR : PLAN AND SECTION

3.6.1.1 Problems faced

In Andhra HEP during July 1988 floods, 94 m length out of a total 172 m length of flushing conduit was reported to have choked resulting in closure of power house for 44 days on two occasions.

In July, 1990 some of the pipe nearly 15 m length of shingle flushing conduit again got chocked during flood, resulting in closure of power house for 12 days.

The operating instructions require that shingle flushing conduit must be kept in perfect operating condition even if it entails reduction in releases for power generation and when discharge in Andhra Khad is 15 cumec or more, the flushing gate must be in fully open position. For lesser discharge occasional but regular flushing is recommended through shingle flushing operation of SFC (shingle flushing conduit) is however manual.

3.6.1.2 Sediment removal

In case of Andhra HEP, however, considerable difficulty was experienced in sediment removal from the single flushing conduit (SFC) when it got chocked for the first time in a length of 94 m, during the flood of July 1988. The sediment removal from the 9 m long cut and cover section in between the trench weir gate and gate control structures (upstream of SFC) was carried out with the help of crane and concreting bucket. Manual cleaning had to be resorted to for SFC and because the diameter of the flushing conduit is only 1.0 m, it took considerable time besides being hazardous, first of all gate at the mouth of SFC that had got struck in silt was lowered to closed position with great difficulty, so that it is safe to enter the conduit, there being no other control to check the sudden flow of water from the stream into the flushing system, with its mouth 5.0 m below the bed of the stream. A slit about 2.0 m x 0.6 m was then cut in one of the pipes of burried conduit to serve both as a manhole and for supply of fresh air. Shingle deposited in the conduit was largely removed by loosening it with high pressure jet and then pushing the same to the exit end by deploying a chain of persons, one behind the other, spread all along and within one metre dia, 172 m long flushing conduit.

After entire cut and cover section (down-stream of trench weir) and single flushing conduit was clear of sediments, the trench weir 12 m long and 12 m wide with depth varying from 0.5 m to 3.4 m still continued to be full with sediments (permeability of the order of 1.38×10^{-3} cm/sec) with water flowing over trench weir and no water entering the water conductor (seepage through the sediments filled in the trench was negligible). After raising the SFC gate to its fully open position a high pressure water jet was forced over wall sediments (filling the trench) from cut and cover side and within seconds, the trench weir became operational with sediments (49 cum) flushing out through the single flushing conduit which started flowing full with water velocity of 4-5 m/sec.

Manual cleaning of SFC had not to be resorted to during its chocking in Sept. 1988, and July, 1990. Regarding Sept. 1988, silt cleaning operation, it was reported that after the cut and cover section had been cleared of sediments and water drawn from the trench weir, the erosion of silt automatically started from the SFC which got completely silt free within 25 to 30 minutes.

Regarding July, 1990 chocking of SFC, it was reported that downstream of the gate whereas the conduit was full with sediments in lower portion, a small open space existed on the top portion of the conduit and after resorting to high pressure water jetting at the mouth of SFC, upstream and downstream of the gate, the sediments in the flushing pipe got washed off automatically. The operation staff had by now acquired some on the job training thus reducing time required for cleaning operation.

Gradation of sediments that filled the water conductor just downstream of the trench weir during flood of July, 1988 at the time of chocking of the system is as given in following table.

Table 3.4 : Sediment Gradation

Sl.No.	I.S. Sieve	Percentage passing
1.	20 mm	100
2.	10 mm	95
3.	4.75 mm	80
4.	2.36 mm	76
5.	600 micron	44
6.	212 micron	14
7.	75 micron	2

As it can be seen from the table 3.2 that 80% of the particles were below 4.75 mm and 86% of the sediment i.e. above 0.2 mm needed elimination through single flushing arrangement or through desilting tank.

- One reason for the trench weir type intake being less efficient could be the tendency of the stream to damming up during flash floods followed by sudden release of water thus bringing with it considerable bed load of shingle, pebbles and sand etc.
- A long length of shingle flushing conduit i.e. 172 m with two bends may also be contributing to the system being not efficient.
- Diameter of SFC is less as compared to quantity of sedimentation.

3.6.2 Binwa HEP

Binwa HEP a run-off river scheme in Kangra district was commissioned in August, 1984. The project comprises two trench weirs, one on Banu that and one on Parai Khad, tributaries of Binwa Khad, an underground desilting tank 37.5 m long and 5.0 m wide, a free flow tunnel 1.8 m D-shaped 2 km long; a forebay reservoir with storage capacity of 18500 m^3, a single penstock 1.1 m dia for 209 m initial length followed by a 1.0 m diameter penstock of 289 m length feeding 2 units of 3 MW each, operating under a gross head of 233.0 m and utilizing design discharge of 3.23 m^3/sec. Layout of intake area and trench weir details are shown in Fig. 3.7 and 3.8.

3.6.2.1 Problems faced

Trench weir for Binwa Hydel Project was constructed after conducting model studies at Irrigation Research Institute Roorkee and ensuring satisfactory performance on the model. Still there were some sediment problems which are as under.

During July, 1985 it was noticed that due to flash flood in one of the Khads, the entire 32 m length of the open channel downstream of the trench weir and the three hoppers and desilting, tank got filled with sediments on one occasion and open channel got filled on the other occasion, resulting in closure of powerhouse for 9 days. During July, 1986 when a flood protection wall upstream of open channel has been constructed which, no doubt saved the open channel from direct entry of wooden logs and trash etc. yet one of the hopper got chocked and power house had to remain close for 66 hours i.e. about 3 days.

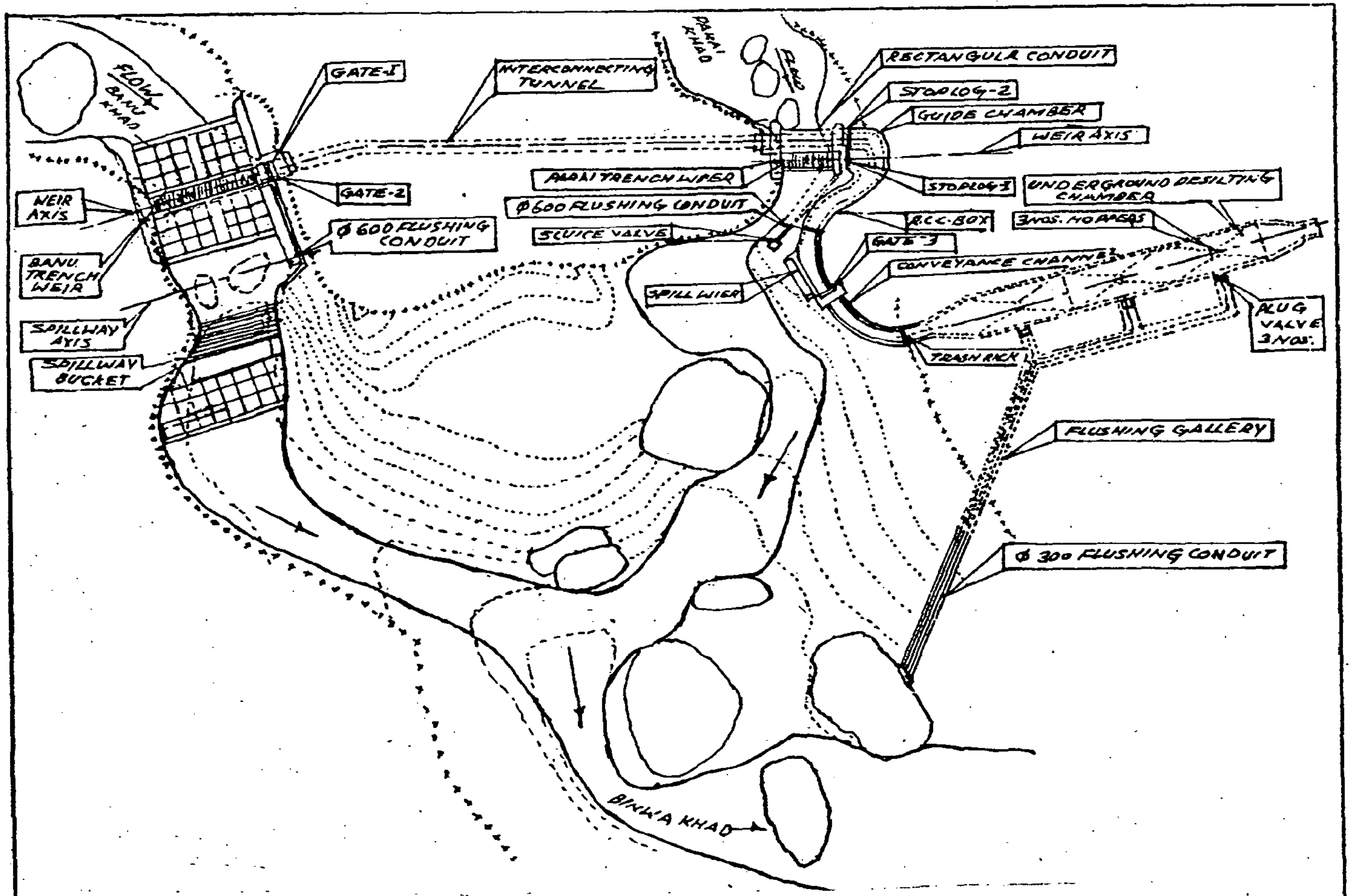


FIG. 3.7 : LAYOUT PLAN OF INTAKE AREA - BINWA HYDEL PROJECT

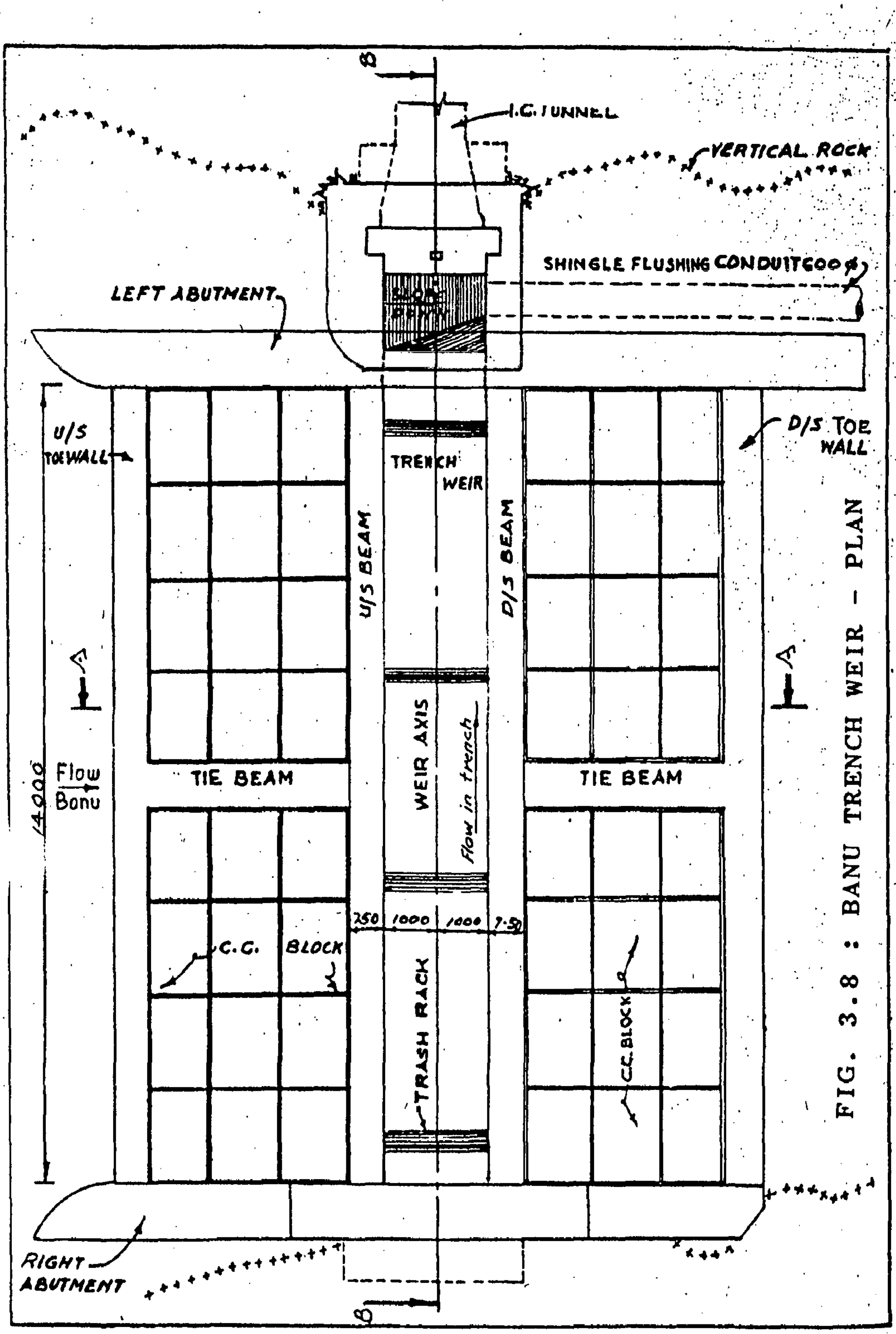
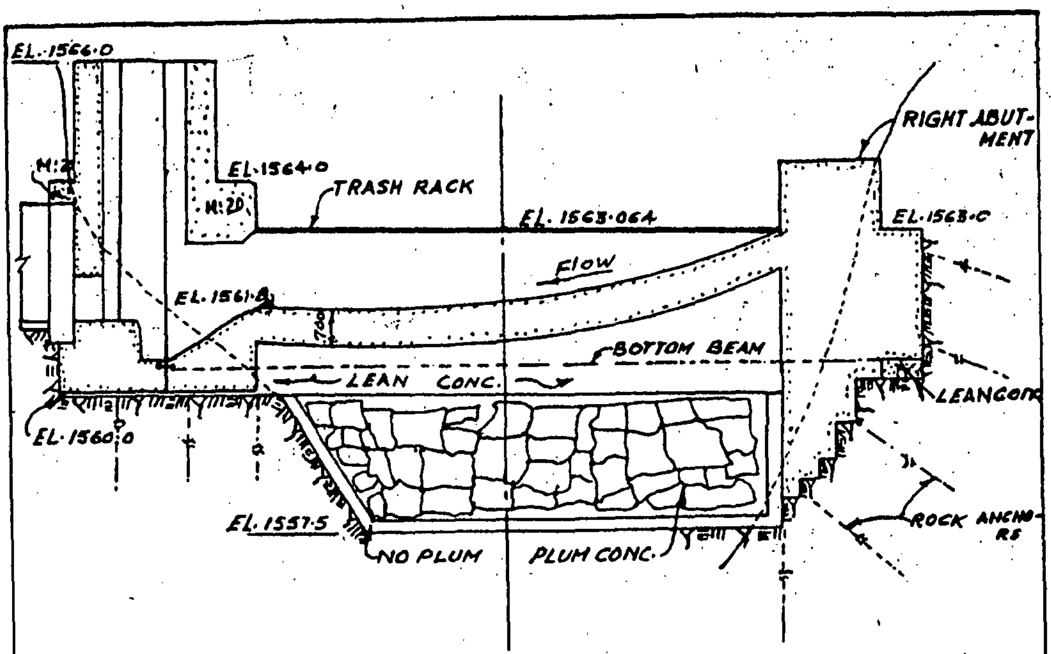
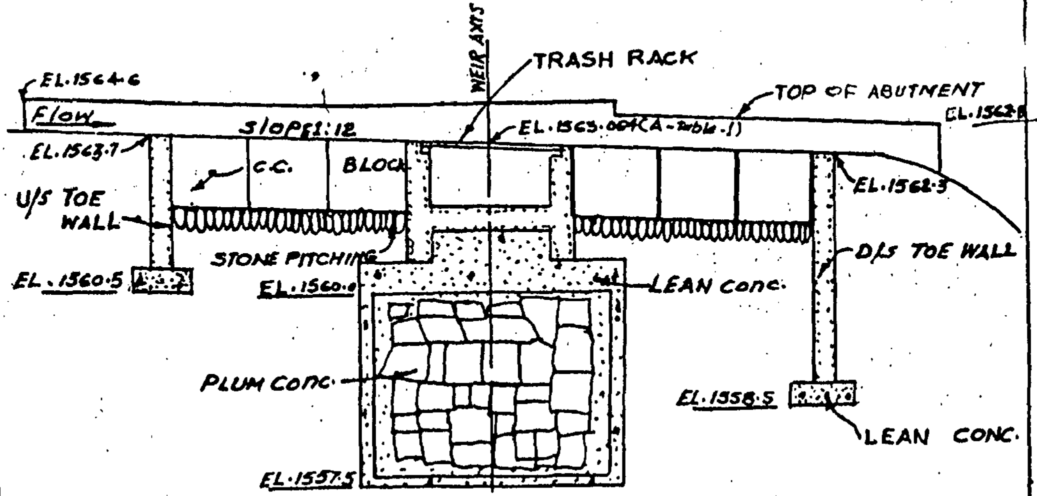


FIG. 3.8 : BANU TRENCH WEIR - PLAN



SECTION- B.B

FIG. 3.8B



SECTION - A·A

FIG. 3.8A - BANU TRENCH WEIR - SECTIONS

3.6.2.2 Sediment removal

200 to 225 cum of sediments got deposited in the open channel and desilting tank in Binwa HEP during the first flash flood. Removal of sediment from the open channel and underground desilting tank (located just at tunnel inlet) did not present much problem except that these had to be removed manually involving considerable lift. The deepest point of the hopper is about 7.0 m below the bed of open channel, 1.5 m x 2.0 m in size. As the operation staff gradually acquired the requisite skill in operation of gates and valves, so deposition of sediments in the water conductor reduced in the subsequent years and also the closure of power house.

CHAPTER - 4

DESILTING CHAMBER

4.1 GENERAL

Desilting chamber adds to the cost of mini-micro hydro-power stations but it is necessary where the water contains large quantities of coarse silt to minimise corosion damages to the turbine runner and other moving parts of the machine. Generally, hill streams carry appreciable quantity of silt and sand during rainy season. These are most harmful due to the fact that development of such streams is generally for high heads and abrasion effects become more pronounced with increasing head.

Different types of silt ejection devices have been developed and tested at Irrigation Research Institute, Roorkee and hopper type sediment ejectors with continuous hydraulic flushing arrangement have been observed with better sediment removal efficiency. But in mini-micro hydel projects as economy is the main factor so, a longitudinal slope accompained withsuitable transverse slope for smooth travel of settled sediment towards depressions can be provided. In these cases the slope in the bed being lesser than the angle of repose of silt particles, some sediment will get deposited in the tank itself and will need removal from time to time.

4.2 SIZE OF PARTICLES TO BE REMOVED

The desilting chamber may be designed to exclude the particles coarser than the size mentioned in Table No. 4.1 for various heads of water to achieve a power draft free of abrasion effects.

Table 4.1

S.No.	Head	Size of silt particles to be removed
1.	Medium head	0.2 to 0.5 mm
2.	High head	0.1 to 0.2 mm

4.3 SIZE OF DESILTING CHAMBER

Size of desilting chamber can be worked out by Camp's design approach as described here in below for 95% removal of the particle sizes mentioned in Table 4.1.

4.3.1 Flushing Discharge

If 'Q' cumec is the design discharge to be utilized for power generation then fluhing dischage of 26 to 25% of it should be taken and a total (1.2 Q to 1.25 Q) cumec of river discharge should be drawn through the intake.

4.3.2 Flow Through Velocity 'V'

The critical flow through velocity is defined as the maximum velociy at which the particle once settled would not be lifted again. As p er Camp's formula critical velecity $V = a\sqrt{ds}$ cm/sec. Where ds = limit particle size coarser to which are to be remove and 'a' is the co-efficient

$a = 36$, for $d > 1$ mm

$a = 44$ for $1 \text{ mm} > d > 0.1$ mm and

$a = 51$ for $0.1 \text{ mm} > d$

As per Emil Mosonyi care should be taken during the hydraulic designing of the settling basin to ensure the calculated velocity in the structure. In basin of older design the specified flow through velocity of 0.2 to 0.3 m/sec could only be realized by using large basin cross-sections and the transition from the canal were abrupt. In such basins instead of the low velocity full area flow expected according to the design,a high-velocity central current was observed between dead spaces and vortices. This phenomenon which is referred to as "hydraulic short circuit" has been observed in many operating setting basins. So now a days design is based upon higher mean flow-through velocities in the basin ranging from 0.4 to 0.6 m/sec.

4.3.3 Depth of Desilting Chamber

As per E.Mosonyi the next value that can be assumed, is one of the main dimension of the chamber. In view of the fact that long and/or wide basins can be constructed at lower costs than deep ones the minimum practical depth should be adopted for the design. The depth of horizontal flow settling chamber applied at water power projects is generally between 1.5 to 4.0 m.

4.3.4 Settling Velocity or Fall Velocity 'w'

The relative motion between sediment particles and the surrounding fluid under various conditions of entrainment, transportation and deposition appears to depend upon essentially the same factors as the velocity at which the particle would fall through the fluid under their own weight. As a result, the fall velocity of the indivisual particle has come to represent a

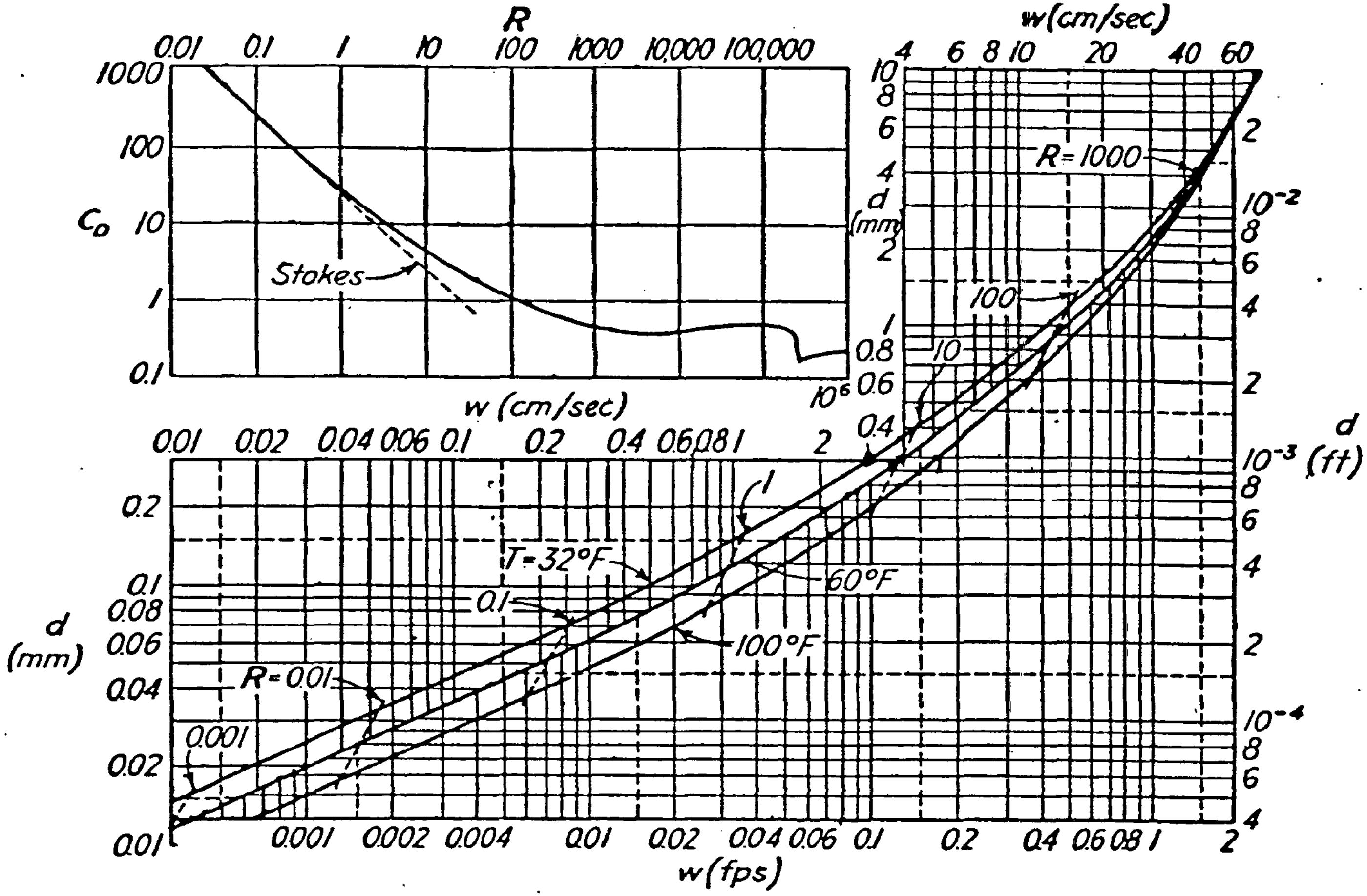


FIG. 4.1: Chart of sedimentation diameter versus fall velocity in water.

chacterstic of the particle considered as well as analytical value.

The the fall velocity can be read from the fall velocity v/s particle diameter, curve developed by Hunter Rouse, for different temperatures of water as shown in Fig 4.1.

4.3.5 Camp's Design Approach

Effective design of a desilting basin depends upon an analysis of the effect of turbulence upon the rate of deposition.Camp has evaluated the turbulent transport function for two dimensional flow by making use of two simplyfying assumptions.

(1) that the fluid velocity is same at every point in the channel and

(2) that the mixing co-efficient is also the same at every point.

On these assumptions he derived a retation for the ratio of the sediment leaving the basin to that entering.

$$1 - \frac{(q_s)_2}{(q_s)_i} = F \left[\frac{wy^{1/6}}{V_n \sqrt{g}}, \quad \frac{WL}{V_y} \right] \tag{4.1}$$

in which

$(q_s)_e$ = quantity of sediment of given particle size in effluent

$(q_s)_i$ = quantity of sediment of given particle size in iffluent

w = Fall velocity of the given particle size

y = basin depth

L = basin length

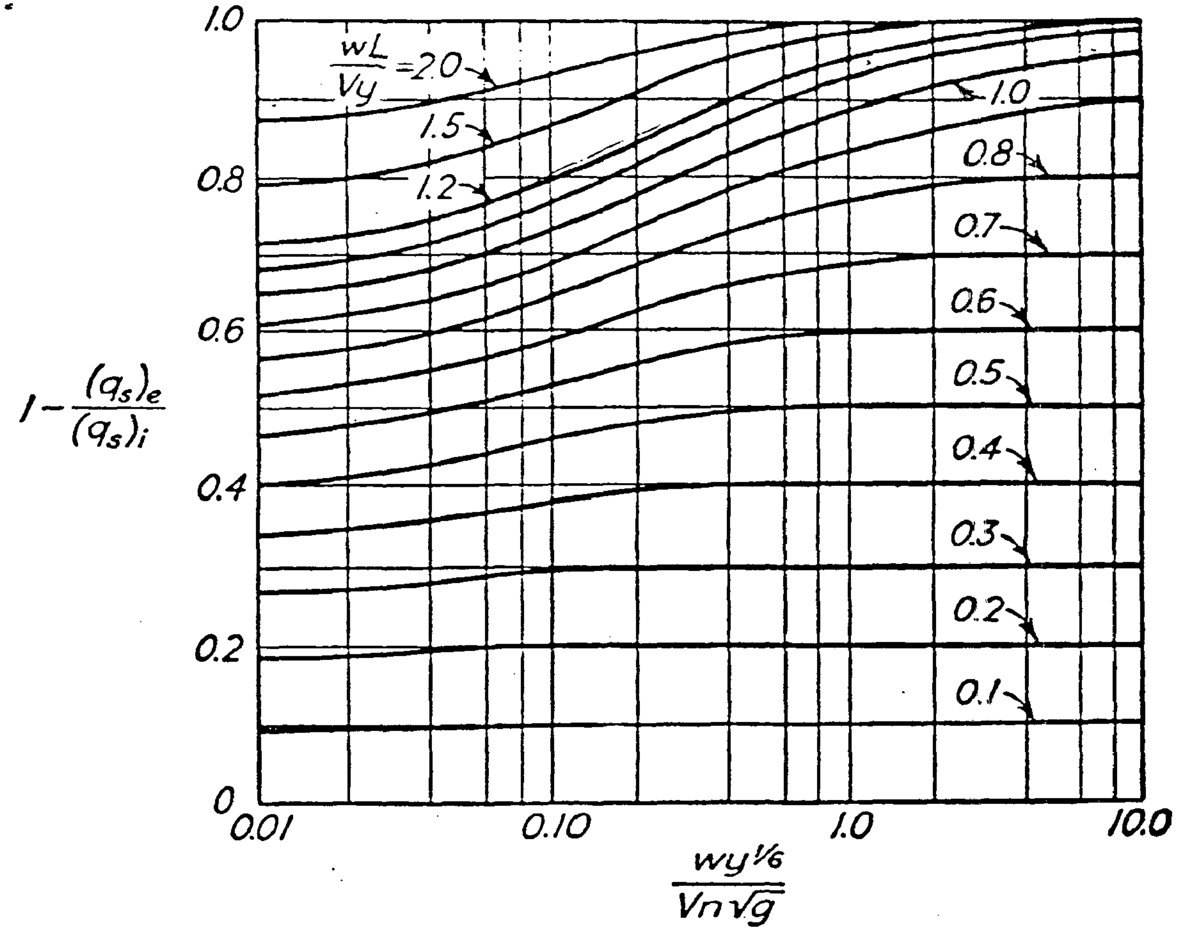


FIG. 4.2 : Sediment-removal function for settling basins.

V = mean velocity of flow in basin

g = acceleration due to gravity

n = Manning's coefficient

This function has been evaluated analyticalley as shown in Fig (4.2) and verfied experimentally by Dobbins. It provides a praçtical and relatively easy means of estimating the effectiveness of a settling basin in removing suspended sediment, and conversly, of designing a basin to remove sediment of original size.The first dimensionless number in the expression 4.1 is nothing but the desired removal ratio. The secound dimenionless number is worked out, since all the variables are known. The third dimensionless number $\frac{WL}{Vy}$ can now be read from the Camp's curves as given in Fig 4.2. In this number all variables other than 'L' being known, the value of 'L' can be worked out.

4.4 DESIGN EXAMPLE

Desilting chamber for Kush mini hydro electric project (Arunachal Pradesh) has been designed as per details in Figure 4.3 to serve as an example.

As the Kush river carries appreaciable quantities of course silt during rainy season, a desilting chamber is considered necessary to minimise the abrasion effects on the turbine parts.The desilting chamber has been designed in order to remove sediment perticles above 0.25 mm size. Removal ratio considered is 95%. The discharge outgoing from the desilting tank for power generation is 1.43 cumec (including 10% losses). To provide for continuous flushing of silt during monsoon periods, it is proposed to draw a discharge of 1.72 cumec from the intake so that 0.29 cumec (20 % of design discharge) may be flushed to ensure the flushing of the deposited silt.

Limit particle size ds = 0.25 mm

Removal ratio = 0.95

Design discharge = 1.43 cumece

Flushing discharge = 20 % of design discharge = $\frac{20}{100} \times 1.43$

= 0.286 say 0.29 cumece

Total discharge to be taken from intake = 1.43 + 0.29 = 1.72 cumec.

Flow through velocity $V = a\sqrt{ds}$

here a = 44

ds = 0.25

$V = 44 \times \sqrt{0.25}$ = 22 cm/sec =0.22 m/sec

But as per Masony's criteria it should be taken between 0.4 to 0.6 m/sec.

Depth of water in desilting chamber = 2.25 m (assumed)

Let 0.3 m be the board

So total depth y = 2.25 + 0.3 = 2.55 m. It should be between 1.5 to 4 m as per criteria.

Fall velocity :

Let the temperature of water in monsoon season be 32° F

Limit particle size ds = 0.25 mm

From sedimentation versus fall velocity curve (Fig. 4.1).

fall velocity = 3 cm/sec = 0.03 m/sec

Manning's co-efficient = n = 0.018

Table 4. 2 : **Dimensions of Sediment Chamber for Kush Mini-Hydroelectric Project by Camp's Design Approach for 95% Removal Ratio**

D_s (mm)	W (m/sec)	y (m)	n	$y^{1/6}$	V (m/sec)	$\frac{w\ y^{1/6}}{Vn\sqrt{g}}$	$\frac{wL}{Vy}$	$\frac{Vy}{w}$	L (m)	B (m)	L x B (m^2)
0.25	0.03	2.25	0.018	1.145	0.40	1.523	1.12	30.0	33.6	1.91	64.0
0.25	0.03	2.25	0.018	1.145	0.45	1.354	1.15	33.8	38.81	1.70	66.0
0.25	0.03	2.25	0.018	1.145	0.50	1.219	1.18	36.5	44.25	1.53	68.0

At it can be seen from the above table that the value of 'L' is minimum for V = 0.4 m/sec which comes out to be 33.6 m. Also total area of tank (64 m^2) is minimum with V = 0.4 m/sec. Soit will be economical one.

So adopt length L = 33.6 m

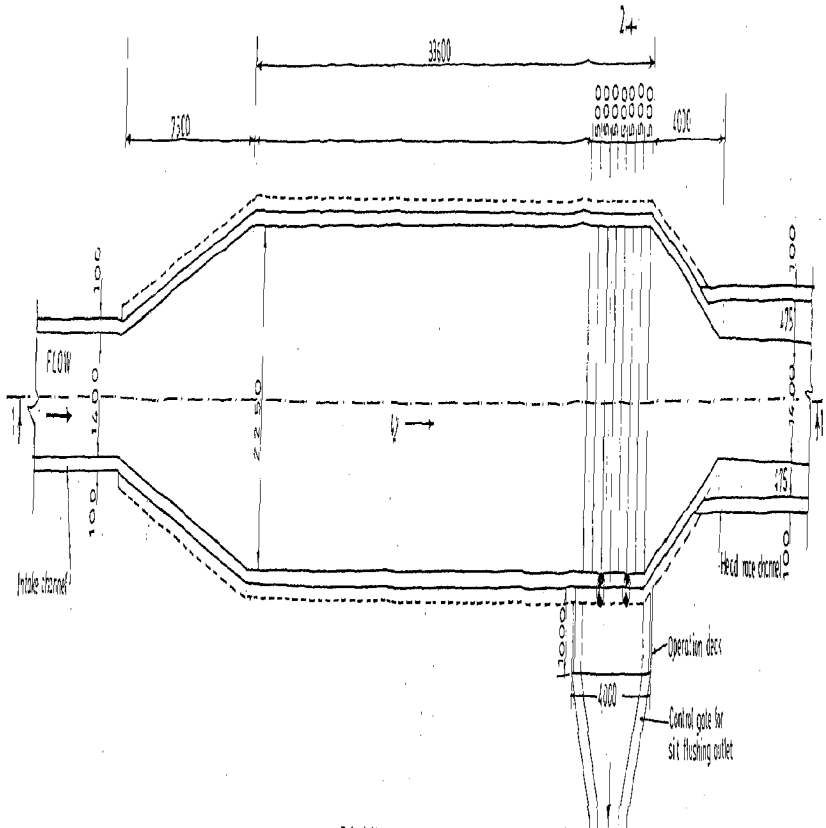
With B = 1.91 m.

But minimum width should be at least equivalent to or more than the depth. So adopt width of tank = 2.25 m.

Here a longitudinal slope of 4% is provided in the bed of the tank accompained with suitable transverse slope for smooth travel of settled sediment towards depressions. Suitable channel transitions of 7.5 m length in upstream and 4.0 m in down stream are provided.

NOTES:-

1. All dimensions are in millimetre and levels in metre.



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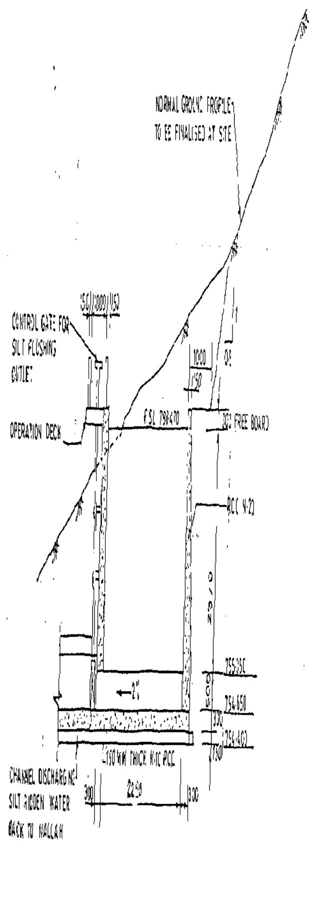

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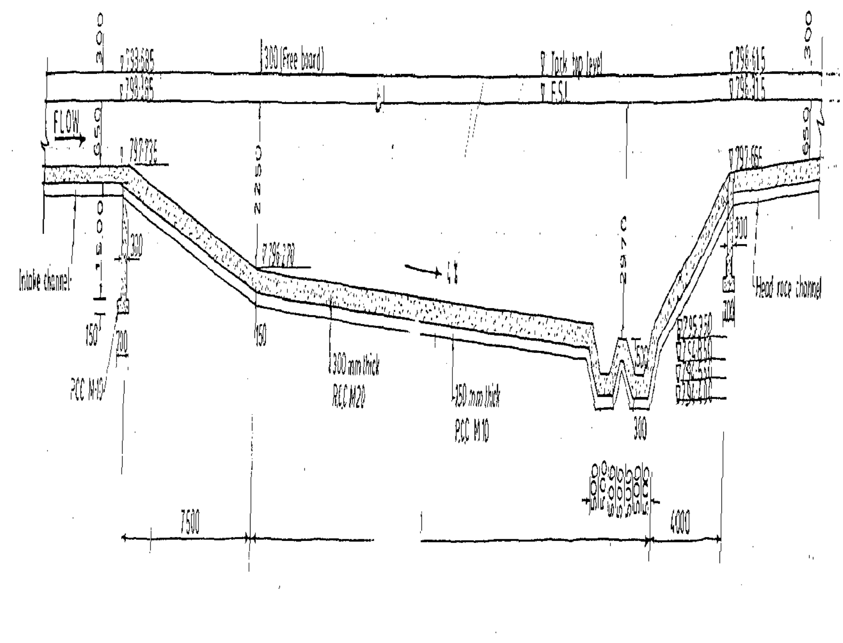


SECTIO

FIG. 4.3 : DESILTING CHAMBER

KUSH MINI HYDRO ELECTRIC PROJECT

5) In extremely severe and long winter periods the water freezes in the canal.

The following solutions can be applied in the critical sections having above problems separately or simultaneously.

a) The water may be conveyed in tunnels. In order to reduce head losses such tunnels should be designed for free-surface flow conditions. Free flow tunnels can be protected against over pressures caused by occasional surges in the power canal by installing spillways in the latter. Spillways may be most conveniently located at places where the canal intersects a rivulet or another canal. Tunnels provide protection against the formation of frazil ice, snow and stone avalaunches.

b) Open or closed flumes of steel, concrete as R.C.C. can be provided.

c) In case of freezing, protection can be affected by providing closed canal or covering the canal with removable cover. Canal sections crossing settlements are frequently constructed with a closed cross-section in order to occupy less ground area, to provide protection against pollution and to meet safety requirements.

Canals that are stable, with an open cross-section and are provided with a cover for the sole reason of protecting against pollution and excessive cooling may be referred to as covered canals. However, if the cover is designed as a monolithic structure with the side walls and the foundation, the canal is of closed type.

d) When there are chances of rock falls the channel should not only be covered by the covers but should also be protected by a soil cover covering of say 300 mm. So that the covers or the roof slab of the canal do not get damaged by the falling rock pieces/boulders.

Where the mountain side is too steep but good rock formations are available use of open surface tunnel sections may prove to be the most economical solutions.

5.2 DESIGN OF CANAL

Longitudinal slope of canal may vary from 1 in 500 to 1 in 1000 depending upon

i) available gross head

ii) Topography of the alignment

iii) Economics.

If available gross head is more, in that case steep slope can be given but simultaneously care should be taken for friction losses. If gross head is very small a flat slope is preferable.

5.2.1 Design of Canal with Steady Flow

Selection of the slope of the power canal and dimensions of the cross-section are closely related. The hydraulically most efficient cross-section will be one which minimizes the hydraulic mean depth R (ratio of cross-sectional area of the canal to wetted perimeter). However, practical considerations, such as a minimum width requirement to operate excavation equipment, may dictate the use of a wider section.

TABLE 5.1 :

VALUES OF n FOR USE IN MANNING OR KUTTER FORMULA

0.009 and 0.010	Very smooth and true surfaces, without projections. Clean new glass, pyralin, or brass, with straight alignment.
0.011 and 0.012	Smoothest clean wood, metal, or concrete surfaces, without projections, and with straight alignment.
0.013	Smooth wood, metal, or concrete surfaces without projections, free from algae or insect growth, and with reasonably straight alignment.
0.014	Good wood, metal, or concrete surfaces with very small projections, with some curvature, with slight insect or algae growth, or with slight gravel deposition. Shot concrete surfaced with troweled mortar.
0.015	Wood with algae and moss growth, concrete with smooth sides but roughly troweled or shot bottom, metal with shallow projections. Same with smoother surface but excessive curvature.
0.016	Metal flumes with large projections into the section. Wood or concrete with heavy algae or moss growth.
0.017	Shot concrete, not troweled, but fairly uniform.
0.018 to 0.025	Metal flumes with large projections into the section and excessive curvature, growths, or accumulated debris.
0.016 to 0.017	Smoothest natural earth channels, free from growths, with straight alignment.
0.020	Smooth natural earth, free from growths, little curvature. Very large canals in good condition.
0.022	Average, well-constructed, moderate-sized earth canal in good condition.
0.025	Very small earth canals or ditches in good condition, or larger canals with some growth on banks or scattered cobbles in bed.
0.030	Canals with considerable aquatic growth. Rock cuts, based on average actual section. Natural streams with good alignment, fairly constant section. Large floodway channels, well maintained.
0.035	Canals half choked with moss growth. Cleared but not continuously maintained floodways.
0.040 to 0.050	Mountain streams in clean loose cobbles. Rivers with variable section and some vegetation growing in banks. Canals with very heavy aquatic growths.
0.050 to 0.150	Natural streams of varying roughness and alignment. The highest values for extremely bad alignment, deep pools, and vegetation, or for floodways with heavy stand of timber and underbrush.

The flow rate 'Q' m^3/sec which will pass through a canal with cross-sectional area of A m^2, hydraulic mean depth R metre and slope of is 'S' is given by

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \qquad (5.1)$$

where 'n' is the Manning roughness co-efficient, the value of which varies from 0.010 for very smooth surfaces to 0.05 for natural streams with rocky beds. Table 5.1 gives accepted values of n for lined and unlined channels. Solution of equation 5.1 involves trial and error if water depth is required and channel slope, shape, and bed width of canal section roughness, and flow rate are known.

For a rectangular channel

Let, bottom width = b

Depth = y_o

Area 'A' = by_o

Wetted parameter P = $b + 2y_o$

then we know $Q = \frac{1}{n} AR^{2/3}S^{1/2}$

or $$\frac{Qn}{1/2} AR^{2/3}$$

or $$\frac{Qn}{S^{1/2}} = by_o \left[\frac{byo}{b+2y_o} \right]^{2/3} \qquad \because (R = A/P)$$

or $$\frac{Qn}{S^{1/2}} = bb^{2/3} y_o \left[\frac{y_o}{b+2y_o} \right]^{2/3}$$

$$\text{or} \qquad \frac{Qn}{S^{1/2}} = b^2 b^{2/3} \frac{y_o}{b}\left[\frac{y_{o/b}}{1+2y_o/b}\right]^{2/3}$$

$$\text{or} \qquad \frac{Qn}{S^{1/2} b^{8/3}} = \frac{y_o}{b}\left[\frac{y_{o/b}}{1+2y_o/b}\right]^{2/3} \qquad (5.2)$$

Since the right hand side of equation (5.2) are functions of only $\frac{y_o}{b}$, a table of values of $Qn/S^{1/2/}b^{8/3}$ may be formed using a range of values of y_o/b. If the channel width 'b' is assumed, the depth can be calculated using that table. This procedure can be followed for other cross-sectional shapes as well. Table 5.2 is such a tabulation for trapezoidal sections with side shapes ranging from Z = 0 (rectangular section) to Z = 4

5.2.2 Design of Canal with Unsteady Flow

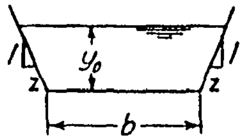
Flow in power canal will take time to respond to a change in turbine flow rate. Thus if canal is operating at a steady flow rate and the flow to the turbines is stopped suddenly, a positive surge will move upstream in the canal. Momentum and continuity considerations can be used to evaluate the height of surge Y and its velocity C in accordance with the following equations for the particular case of rectangular channel.

$$Y_2 = \frac{Y_1}{2}\left[\sqrt{1+8\ (V^2|gy)-1}\ \right] \qquad (5.3)$$

$$C = \sqrt{gy_1}\ \left[\left(\frac{y_2}{2y_1}\right)\ \left(1+\frac{y_2}{y_1}\right)\right]^{1/2} \qquad (5.4)$$

TABLE 5.2 :

Uniform Flow in Trapezoidal Channels by Manning Formula



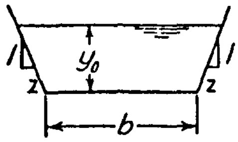
Values of $\frac{Qn}{b^{8/3} S_0^{1/2}}$

$\frac{y_0}{b}$	$z = 0$	$z = \frac{1}{4}$	$z = \frac{1}{2}$	$z = \frac{3}{4}$	$z = 1$	$z = 1\frac{1}{4}$	$z = 1\frac{1}{2}$	$z = 2$	$z = 2\frac{1}{2}$	$z = 3$	$z = 4$
0.02	0.00213	0.00215	0.00216	0.00217	0.00218	0.00219	0.00220	0.00221	0.00222	0.00223	0.00225
0.03	0.00414	0.00419	0.00423	0.00426	0.00429	0.00431	0.00433	0.00437	0.00440	0.00443	0.00449
0.04	0.00661	0.00670	0.00679	0.00685	0.00690	0.00696	0.00700	0.00707	0.00715	0.00722	0.00735
0.05	0.00947	0.00964	0.00980	0.00991	0.0100	0.0101	0.0102	0.0103	0.0104	0.0106	0.0109
0.06	0.0127	0.0130	0.0132	0.0134	0.0136	0.0137	0.0138	0.0141	0.0143	0.0145	0.0149
0.07	0.0162	0.0166	0.0170	0.0173	0.0176	0.0177	0.0180	0.0183	0.0186	0.0190	0.0196
0.08	0.0200	0.0206	0.0211	0.0215	0.0219	0.0222	0.0225	0.0231	0.0235	0.0240	0.0250
0.09	0.0240	0.0249	0.0256	0.0262	0.0267	0.0271	0.0275	0.0282	0.0289	0.0296	0.0310
0.10	0.0283	0.0294	0.0305	0.0311	0.0318	0.0324	0.0329	0.0339	0.0348	0.0358	0.0375
0.11	0.0329	0.0342	0.0354	0.0364	0.0373	0.0380	0.0387	0.0400	0.0413	0.0424	0.0448
0.12	0.0376	0.0393	0.0408	0.0420	0.0431	0.0441	0.0450	0.0466	0.0482	0.0497	0.0527
0.13	0.0425	0.0446	0.0464	0.0480	0.0493	0.0505	0.0516	0.0537	0.0556	0.0575	0.0613
0.14	0.0476	0.0501	0.0524	0.0542	0.0559	0.0573	0.0587	0.0612	0.0636	0.0659	0.0705
0.15	0.0528	0.0559	0.0585	0.0608	0.0628	0.0645	0.0662	0.0692	0.0721	0.0749	0.0805
0.16	0.0582	0.0619	0.0650	0.0676	0.0699	0.0720	0.0740	0.0776	0.0811	0.0845	0.0912
0.17	0.0638	0.0680	0.0717	0.0748	0.0775	0.0800	0.0823	0.0867	0.0907	0.0947	0.103
0.18	0.0695	0.0744	0.0786	0.0822	0.0854	0.0883	0.0910	0.0961	0.101	0.105	0.115
0.19	0.0753	0.0809	0.0857	0.0900	0.0936	0.0970	0.100	0.106	0.112	0.117	0.128
0.20	0.0813	0.0875	0.0932	0.0979	0.102	0.106	0.110	0.116	0.123	0.129	0.141
0.21	0.0873	0.0944	0.101	0.106	0.111	0.115	0.120	0.127	0.134	0.142	0.156
0.22	0.0935	0.101	0.109	0.115	0.120	0.125	0.130	0.139	0.147	0.155	0.171
0.23	0.0997	0.109	0.117	0.124	0.130	0.135	0.141	0.151	0.160	0.169	0.187
0.24	0.106	0.116	0.125	0.133	0.139	0.146	0.152	0.163	0.173	0.184	0.204
0.25	0.113	0.124	0.133	0.142	0.150	0.157	0.163	0.176	0.187	0.199	0.222
0.26	0.119	0.131	0.142	0.152	0.160	0.168	0.175	0.189	0.202	0.215	0.241
0.27	0.126	0.139	0.151	0.162	0.171	0.180	0.188	0.203	0.218	0.232	0.260
0.28	0.133	0.147	0.160	0.172	0.182	0.192	0.201	0.217	0.234	0.249	0.281
0.29	0.139	0.155	0.170	0.182	0.193	0.204	0.214	0.232	0.250	0.267	0.302
0.30	0.146	0.163	0.179	0.193	0.205	0.217	0.227	0.248	0.267	0.286	0.324
0.31	0.153	0.172	0.189	0.204	0.217	0.230	0.242	0.264	0.285	0.306	0.347
0.32	0.160	0.180	0.199	0.215	0.230	0.243	0.256	0.281	0.304	0.327	0.371
0.33	0.167	0.189	0.209	0.227	0.243	0.257	0.271	0.298	0.323	0.348	0.396
0.34	0.174	0.198	0.219	0.238	0.256	0.272	0.287	0.315	0.343	0.369	0.422
0.35	0.181	0.207	0.230	0.251	0.270	0.287	0.303	0.334	0.363	0.392	0.450
0.36	0.190	0.216	0.241	0.263	0.283	0.302	0.319	0.353	0.384	0.416	0.477
0.37	0.196	0.225	0.251	0.275	0.297	0.317	0.336	0.372	0.406	0.440	0.507
0.38	0.203	0.234	0.263	0.289	0.311	0.333	0.354	0.392	0.429	0.465	0.536
0.39	0.210	0.244	0.274	0.301	0.326	0.349	0.371	0.412	0.452	0.491	0.568
0.40	0.218	0.254	0.286	0.314	0.341	0.366	0.389	0.433	0.476	0.518	0.600
0.41	0.225	0.263	0.297	0.328	0.357	0.383	0.408	0.455	0.501	0.545	0.634
0.42	0.233	0.279	0.310	0.342	0.373	0.401	0.427	0.478	0.526	0.574	0.668
0.43	0.241	0.282	0.321	0.356	0.389	0.418	0.447	0.501	0.553	0.604	0.703
0.44	0.249	0.292	0.334	0.371	0.405	0.437	0.467	0.524	0.579	0.634	0.739
0.45	0.256	0.303	0.346	0.385	0.422	0.455	0.487	0.548	0.607	0.665	0.778
0.46	0.263	0.313	0.359	0.401	0.439	0.475	0.509	0.574	0.635	0.696	0.816
0.47	0.271	0.323	0.371	0.417	0.457	0.494	0.530	0.600	0.665	0.729	0.856
0.48	0.279	0.333	0.384	0.432	0.475	0.514	0.552	0.626	0.695	0.763	0.897
0.49	0.287	0.345	0.398	0.448	0.492	0.534	0.575	0.652	0.725	0.797	0.939
0.50	0.295	0.356	0.411	0.463	0.512	0.556	0.599	0.679	0.758	0.833	0.983
0.52	0.310	0.377	0.438	0.496	0.548	0.599	0.646	0.735	0.820	0.906	1.07
0.54	0.327	0.398	0.468	0.530	0.590	0.644	0.696	0.795	0.891	0.984	1.17
0.56	0.343	0.421	0.496	0.567	0.631	0.690	0.748	0.856	0.963	1.07	1.27
0.58	0.359	0.444	0.526	0.601	0.671	0.739	0.802	0.922	1.04	1.15	1.37
0.60	0.375	0.468	0.556	0.640	0.717	0.789	0.858	0.988	1.12	1.24	1.49
0.62	0.391	0.492	0.590	0.679	0.763	0.841	0.917	1.06	1.20	1.33	1.60
0.64	0.408	0.516	0.620	0.718	0.809	0.894	0.976	1.13	1.28	1.43	1.72

TABLE 5.2 (Contd/-)

UNIFORM FLOW IN TRAPEZOIDAL CHANNELS BY MANNING FORMULA

Values of $\frac{Qn}{b^{8/3}S_0^{1/2}}$



$\frac{y_0}{b}$	z = 0	z = ¼	z = ½	z = ¾	z = 1	z = 1¼	z = 1½	z = 2	z = 2½	z = 3	z = 4
0.66	0.424	0.541	0.653	0.759	0.858	0.951	1.04	1.21	1.37	1.53	1.85
0.68	0.441	0.566	0.687	0.801	0.908	1.01	1.10	1.29	1.47	1.64	1.98
0.70	0.457	0.591	0.722	0.842	0.958	1.07	1.17	1.37	1.56	1.75	2.12
0.72	0.474	0.617	0.757	0.887	1.01	1.13	1.24	1.45	1.66	1.87	2.27
0.74	0.491	0.644	0.793	0.932	1.07	1.19	1.31	1.55	1.77	1.98	2.41
0.76	0.508	0.670	0.830	0.981	1.12	1.26	1.39	1.64	1.88	2.11	2.57
0.78	0.525	0.698	0.868	1.03	1.18	1.32	1.46	1.73	1.98	2.24	2.73
0.80	0.542	0.725	0.906	1.08	1.24	1.40	1.54	1.83	2.10	2.37	2.90
0.82	0.559	0.753	0.945	1.13	1.30	1.47	1.63	1.93	2.22	2.51	3.07
0.84	0.576	0.782	0.985	1.18	1.36	1.54	1.71	2.03	2.34	2.65	3.25
0.86	0.593	0.810	1.03	1.23	1.43	1.61	1.79	2.14	2.47	2.80	3.44
0.88	0.610	0.839	1.07	1.29	1.49	1.69	1.88	2.25	2.60	2.95	3.63
0.90	0.627	0.871	1.11	1.34	1.56	1.77	1.98	2.36	2.74	3.11	3.83
0.92	0.645	0.898	1.15	1.40	1.63	1 86	2.07	2.48	2.88	3.27	4.04
0.94	0.662	0.928	1.20	1.46	1.70	1.94	2.16	2.60	3.03	3.43	4.25
0.96	0.680	0.960	1.25	1.52	1.78	2.03	2.27	2.73	3.17	3.61	4.48
0.98	0.697	0.991	1.29	1.58	1.85	2.11	2.37	2.85	3.33	3.79	4.70
1.00	0.714	1.02	1.33	1.64	1.93	2.21	2.47	2.99	3.48	3.97	4.93
1.05	0.759	1.10	1.46	1.80	2.13	2.44	2.75	3.33	3.90	4.45	5.55
1.10	0.802	1.19	1.58	1.97	2.34	2.69	3.04	3.70	4.34	4.96	6.21
1.15	0.846	1.27	1.71	2.14	2.56	2.96	3.34	4.09	4.82	5.52	6.91
1.20	0.891	1.36	1.85	2.33	2.79	3.24	3.68	4.50	5.32	6.11	7.68
1.25	0.936	1.45	1.99	2.52	3.04	3.54	4.03	4.95	5.86	6.73	8.48
1.30	0.980	1.54	2.14	2.73	3.30	3.85	4.39	5.42	6.42	7.39	9.34
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.76	5.90	7.01	8.10	10.2
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	6.43	7.65	8.83	11.2
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.98	8.30	9.62	12.2
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	7.55	9.02	10.4	13.3
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	8.14	9.74	11.3	14.4
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	8.79	10.5	12.2	15.6
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	9.42	11.3	13.2	16.8
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	10.1	12.2	14.2	18.1
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	10.9	13.0	15.2	19.5
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	11.6	14.0	16.3	20.9
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	12.4	15.0	17.4	22.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	13.2	15.9	18.7	24.0
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	14.0	17.0	19.9	25.6
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	14.9	18.0	21.1	27.2
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	16.8	20.3	23.9	30.8
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	18.7	22.8	26.8	34.7
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	20.9	25.4	30.0	38.8
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	23.1	28.3	33.4	43.3
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	25.6	31.3	37.0	48.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	28.2	34.5	40.8	53.0
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	31.0	37.9	44.8	58.4
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	33.8	41.6	49.1	64.0
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	36.9	45.3	53.7	70.1
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	40.1	49.4	58.4	76.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	47.1	58.0	68.9	90.3
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.5	54.6	67.7	80.2	105
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	63.0	78.2	92.8	122
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	72.4	89.6	107	141
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	82.2	102	122	160
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	111	136	164	217
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	145	181	216	287

where,

g = acceleration due to gravity

V_1 = velocity before surge

y_1 = depth before surge

y_2 = depth after surge.

For a trapezoidal channel the equation of momentum and continuity are somewhat more complicated, but table 5.3 gives values of depth y_2 in terms of the upstream depth Y_1, the channels side slopes Z, and the total upstream specific energy $H_1 = y_1 + V_1^2|2g$

Sufficient freeboard should be provided to allow for the formation of this surge at the downstream end of the canal. However, additional freeboard may be required to allow for wind generated waves, sedimentation or excess storm flow. This free board should wshall not be less than 0.20 m for a canal carrying 0.3 m^3/sec or 0.30 m for a canal carrying 2.8 m^3/sec (100 ft^3/sec).

Eventually as the surge progresses upstream in the canal, the growing downstream volume of water will overtop the embankment unless an uncontrolled spillway is provided. The capacity of spillway must be equal to the canal capacity. Equation 5.4 given above can be used to estimate the velocity at which a negatives surge will move upstream if the flow into the penstocks is started suddenly. An increase in flow can not be introduced in less than approximately twice the time a surge takes to travel from penstock entrance to the canal intake gates. A forebay at the penstock

TABLE 5.3 : DOWNSTREAM DEPTHS FOR A SURGE IN A TRAPEZOIDAL CHANNEL

y_1/H_1	Values of y_2/H_1				
	Triangular sections, $\frac{b}{zy_1} = 0$	$\frac{b}{zy_1} = 1$	$\frac{b}{zy_1} = 4$	$\frac{b}{zy_1} = 20$, $\frac{zy_1}{b} = 0.05$	Rectangular sections, $\frac{zy_1}{b} = 0$
0.10	.370	.418	.476	.528	.552
0.20	.562	.616	.664	.696	.706
0.30	.694	.738	.766	.778	.780
0.40	.790	.814	.820	.808	.800
0.50	.852	.852	.828	.796	.780
0.60	.877	.846	.792	.742	.724
0.70	.866	.796	.712		
0.80	.800	.750	.706	.677	.667

intake must, therefore, have volume large enough to provide flow to the penstock untill an increase in flow through the canal is achieved. A foreby to accomodate a canal discharge has of atleast 3 mimutes has been recommended for this pruose.

5.3 HEAD RACE TUNNEL

When the mountain slope is too treacherous, discharge may be conveyed through tunnels. In order to reduce head losses, such tunnels may be designed for free flow conditions. Free flow tunnels can be protected against over pressure caused by occasionalsurges in the power canal by installing spillway in the later. The economic diameter of the tunnel is governed by the following considerations.

i) The minimum diameter of the power tunnel is subject to the constraints of workability during construction.

ii) The diameter of the tunnel is based on relative economics.

Larger diameter would mean lesser head loss due to friction resulting greater capital cost and lesser revenue loss. Similarly smaller diameter would mean smaller capital cost but larger revenue loss.

5.3.1 Economic Diameter of Power Tunnel

The most Economical diameter is one which leads to minimum total value of the sum of the following -

i) Annually loss of revenue on account of power head lost due to friction in tunnel.

ii) Recurring annual expenditure.

5.3.2 Calculation of Economic Diameter

(a) By anaytical method :

Annual loss of revenue :

Let us consider circular tunnel,

According to Manning's formula

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

where,

V = Velocity (average)

R = Hydraulic mean depth = $\frac{A}{P} = \frac{\pi}{4} \times \frac{D^2}{\pi \times D} = D/4$

D = Diameter of tunnel

S = Longitudinal slope

N = Rugosity coefficient

So putting the value of R in Manning's equation

$$V = \frac{1}{N} \left(\frac{D}{4} \right)^{2/3} S^{1/2} = \frac{1}{2.53N} D^{2/3} S^{1/2}$$

$$\text{or} \quad S = \frac{6.4 \, V^2 N^2}{D^{1.33}}$$

S in fact is the slope of hydraulic gradient or the head lost in friction per metre length of tunnel.

Let η is the overall efficiency of generation

$\therefore$ Power lost = P_e = 9.8 x Q x S x η k.w

Here discharge will vary with time so if discharge is Q_1, Q_2, Q_3, - Q_n over time T_1, T_2, T_3 ----- T_n

Then equivalent discharge 'Q' is that discharge which has the same total head due to friction as the sun of head losses for running Q_1, Q_2, Q_3 over time T_1, T_2, T_3 etc i.e.

i.e. $Q^3t = \Sigma Q_x^3 T_x = Q_1^3 T_1 + Q_2^3 T_2 + Q_3^3 T_3 + \ldots\ldots Q_n^3 T_n$

$$Q = \left(\frac{\Sigma Q_x^3 T_x}{\Sigma Tx} \right)^{1/3} \qquad \text{where } x = 1,2,3\ldots.n$$

where,

x = 1,2,3... n

Q = Eqvilant discharge in cumec

$= V * \frac{\pi}{4} D^2$

or $= V = \dfrac{1.273\ Q}{D^2}$

Now,

$$P_e = 9.8\ Q \times 6.4\ \frac{V^2N^2}{D^{1.33}} \times n$$

$$P_e = 9.8Q \times 6.4 \times \left(\frac{1.273\ Q}{D^2} \right)^2 \times \frac{N^2}{d^{1.33}} \times n$$

$$= 101.64\ \frac{Q^3}{D^{5.33}} \times N^2 \times n \qquad \text{K.w.}$$

Revenue lost (R_e) per year will be given by

Let C_o = Selling rate per unit.

$$R_e = P_e \times 24 \times 365 \times C_o$$
$$= 101.64 \times \frac{Q^3}{D^{5.33}} \times N^2 \times n \times 24 \times 365 \times C_o$$

$$= 8.9 \times 10^5 \times \frac{Q^3}{D^{5.33}} \times N^2 \times n \times C_o \qquad (5.5)$$

Recurring annual expenditure (A_e) :

It is calculated by multiplying the capital cost of the tunnel by the percentage of annual cost. It should include interst on capital and the operation and maintenance charges of the tunnel of diameter, 'D'.

let D/12 be the thickness of lining

Area of excavation per metre length of tunnel =

$$\frac{\pi}{4} \times \left[D + \frac{D}{12} \times 2\right]^2 = 1.069\ D^2$$

Area of lining per metre length of tunnel

$$= \frac{\pi}{4}\left[\left(D + \frac{D}{12} \times 2\right)^2 - D^2\right]$$

$$= \frac{\pi}{4}\left[\frac{49D^2}{36} - D^2\right] = \frac{\pi}{4} \times \frac{13D^2}{36} = 0.284\ D^2$$

Let

C_e = Cost of excavation per cum.

C_L = Cost of lining per cum.

Assuming overbreak = 15%

$$\text{Total cost of tunnel} = (1.069\ D^2 \times 1.15 \times C_e) + (0.284\ D^2 + 1.069\ D^2 \times 0.15)C_L$$

$$= (1.23\ C_e + 0.444\ C_L)\ D^2$$

Let annual cost is 'a' percent of total cost, then

$$A_e = (1.23\ C_e + 0.444\ C_L)\ D^2 \times a \qquad (5.6)$$

$\therefore$ Total annual loss of revenue + recurring annual expenditure is given by

$$T = R_e + A_e$$

$$= 8.93 \times 10^5 \times \frac{Q^3}{D^{5.33}} \times N^2 \times n \times C_o + (1.23\ C_e + 0.444\ C_L)\ D^2 \times a \qquad (5.7)$$

For minimum value of 'T'

$$\frac{dT}{dD} = 0$$

or

$$- 8.9 \times 10^{5} \times 5.33 \ \frac{Q^{3}}{D^{6.33}} = \times N^{2} \times n \times C_{o} + 2\ (1.23\ C_{e} + 0.444\ C_{L})D \times a = 0$$

$$\text{or} \quad 2a(1.23\ C_{e} + 0.444\ C_{L})D^{7.33} = 8.9 \times 10^{5} \times 5.33 \times N^{2} \times n \times C_{o} \tag{5.8}$$

$$\text{or } D^{7.33} = \frac{(19.35\ Q^{3}N^{2}\ \eta\ C_{o})10^{5}}{(C_{e} + 0.36\ C_{L})} \tag{5.9}$$

The most economical diameter of the tunnel can therefore, be determined by equation (5.8) by trial and error.

b) By Graphical Method

Step :

1) Plot differnt values of diameter on x - axis

2) Plot cost on y-axis

3) Putting different values of diameter D in equation (5.5) and find out the revenue loss per year.

4) Plot different values of revenue loss w.r.t. diameter

5) Again put different values of D in equation (5.6) and find out the corresponding recurring annual expenditure

6) Plot these values of annual expenditures w.r.t. different values of diameter 'D'. On the plot explained under step 4.

7) Draw a third plot against 'D' by adding up the curves drawn under steps 4 & 6.

Now the diameter corresponding, the loest point on this curve will be most economical diameter.

5.4 TAIL RACE

For discharging water after the generation of electricity in the power house, either a tail race channel or a tail race tunnel will be required.

The design of tail race channel will follow the same principles as those adopted for head race channel mentioned previously.

CHAPTER - 6

FOREBAY/BALANCING RESERVOIR AND INTAKE

6.1 GENERAL

The purpose of the forebay or balancing reservoir is to distribute evenly, over a proper transition, the water conveyed by the power canal among the penstocks and at the same time to regulate the power flow into the penstock, through intake as well as to ensure the disposal of excess water through spillway. It is advantageous to combine forebay and balaning reservoir wherever feasible and provide for a storage of 1,2 or 3 hours as may be suitable to the system justifying the economics and requirements for efficient operation of the power plant. Generally in mini-micro hydel projects a storage of few minutes is provided so that structure should be a small one and should not hamper the economics of the project.

Main function of the forebay is to provide immediate waterdemand at the time of starting of the generating units. It also provides enough water cushion over the penstock to prevent air entry and vortex formation.

Main Parts of Forebay

1) Basin

2) Spillway with chute and stilling basin

3) A bottom outlet for flushing out of sediment and ice.

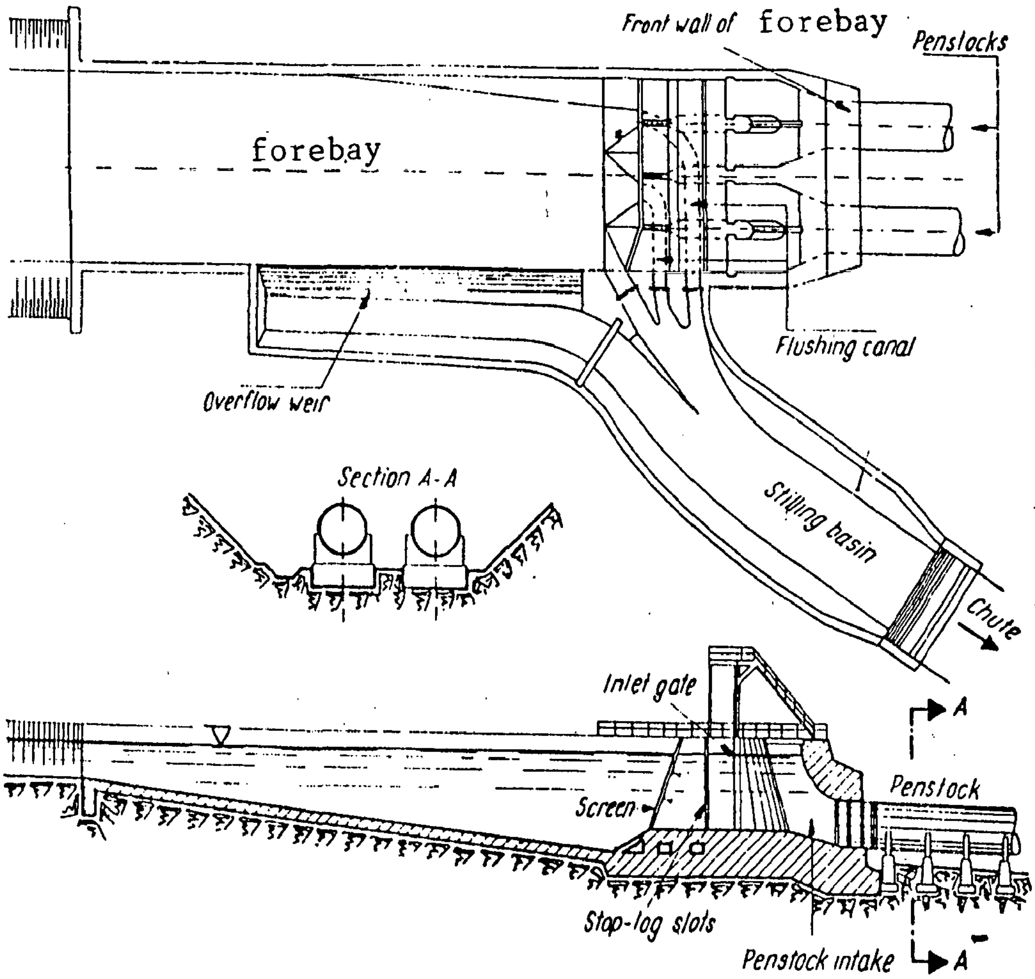


FIG. 6.1 : GENERAL ARRANGEMENT OF THE FOREBAY

4) The intake sill equipped with a screen

5) The gate chamber for controlling entry into the penstock and for into the forebay it self.

6) The penstock inlet.

All these parts are shown in Fig. 6.1.

Site selection is very important item in the design of forebay. Site for forebay and powerhouse should be selected simultaneously with a view to ensure shortest possible penstocks. Parts of the map where the contour lines are close to one another and closely follow the banks of the river course should be considered. This will ensure good quality rocks. The penstock should, as for as possible, be aligned along the ridge. The entire basin of the forebay may be excavated in rock, although the use of side walls for part of its perimeter is a more economical solution. The entire forebay may be arranged above the terrian, enclosed by embankments and dam like retaining walls. The front wall on the side of the penstock usually rises above the terrain. Forebays on steep mountain slopes should be constructed with vertical sides because inclined side walls would necessitate the movement of more quantities of earth or rock for both, the hillside excavation and valley side embankment. If there is no retaining embankment, or rockfill on the valley side the enclosure should be done by side walls of gravity dam type.

6.2 LAYOUT ARRANGEMENT OF FOREBAY

The power canal should join the forebay with a gradual transition and bottom of the basin should have a slope towards the sill. By providing this slope sediments accumulated if any can be flushed with water under pressure. Exposed retaining walls, enclosing the forebay should be provided with water tight control joints. If rock is of good quality without any fractures and faults in mountain side walls then water tighness for joints may not berequired. Bottom lining of basin is done only where seepage is expected. To avoid the concrete lining which is very costly the following solution should be adopted. The smoothened bottom of the basin may be covered with plastic clay in a thickness from 20 to 50 cm. The covering is compacted in several layers and is protected against disturbance due to soaking and wave action by a layer of gravel or crushed stone. Near the sill a pavement should be applied instead of the protecting stone layer.

- The spillway is usually an ogee type located in the valley side retaining wall of the basin. The spillway capacity should be equal to the design discharge of the channel. If the water supply is relatively small, the spillway may be located instead the side wall in the front wall at the end of the basin by applying a suitable structural arrangement can be combined with the bottom outlet.
- The forebay and the bottom outlet canal should be unitimmediately at the foot of the basin. Water spilling over the spillway crest and through the bottom outlet can be

  a) Diverted into a suitable natural drain

  b) Conveyed by a suitable chute.

If diverted into a suitable natural drain then it is the most advantageous and practicable solution.

In second case, an inclined trough chute may may discharge into a drain over a stilling pool constructed at the toe of the chute to prevent erosion of the bed.

The entrance of accumulated bed load into the penstock is prvented by a sill, which is followed by the screen. Flow velocities related to the gross area of the screen may vary between 0.8 m/sec and 0.25 m/sec at high head power plants. Flow to the pressure conduits is controlled either by vertical lift or tainter gates located in a gate chamber. Provision of stoplogs causes also be made in order to ensure proper repairs of the gates. At small power plants the screen and the stop log closure are accessible from the gate house.

6.3 HYDRAULIC DESIGN OF FOREBAY

6.3.1 Forebay Tank

First of all storage time is fixed as per guidelines issued by CEA-1982

Suppose storage time = T minutes

Design discharge = Q cumec

Capacity of tank required = Q x T x 60 cum

Assume width of the tank taking topography and geology of the location.

Let Depth of tank = D metres

Length of tank = L metres

$$\text{Area required} = \frac{Q \times T \times 60}{D} \text{ sq.m.}$$

$$\text{Width of tank 'D'} = \frac{Q \times T \times 60}{D \times L} \text{ metres}$$

Assumed depth of tank is also to be checked as per design criteria of intake which is disscussed in next subhead.

6.4 HYDRAULIC DESIGN OF COMPONENTS OF INTAKE

6.4.1 Bell Mouth Opening and Transition

Shape and size of opening - The penstock and conduit entrance should be designed to produce an acceleration similar to that found in a jet issuing from a sharp edged orifice. The surface should be formed to natural contraction curve and the penstock is assumed to the size of orifice jet at its maximum contraction.

The normal contraction of 40 percent (C_c=0.6) should be used in high head installations and 30 percent (c_c=0.6) for low head installation in order to reduce the height of opening.

6.4.2 Opening Area

$$\text{Opening area} = \frac{\text{Penstock Area}}{C_c \text{ Cos } \phi}$$

where ϕ = angle of inclination of penstock centre line to horizontal

C_c = Co-efficient of contraction

6.4.3. Height and Width of Opening

The height calculated from the distance above and below the intersect of the penstock centre line with the face of the entrance (Fig. 6.2).

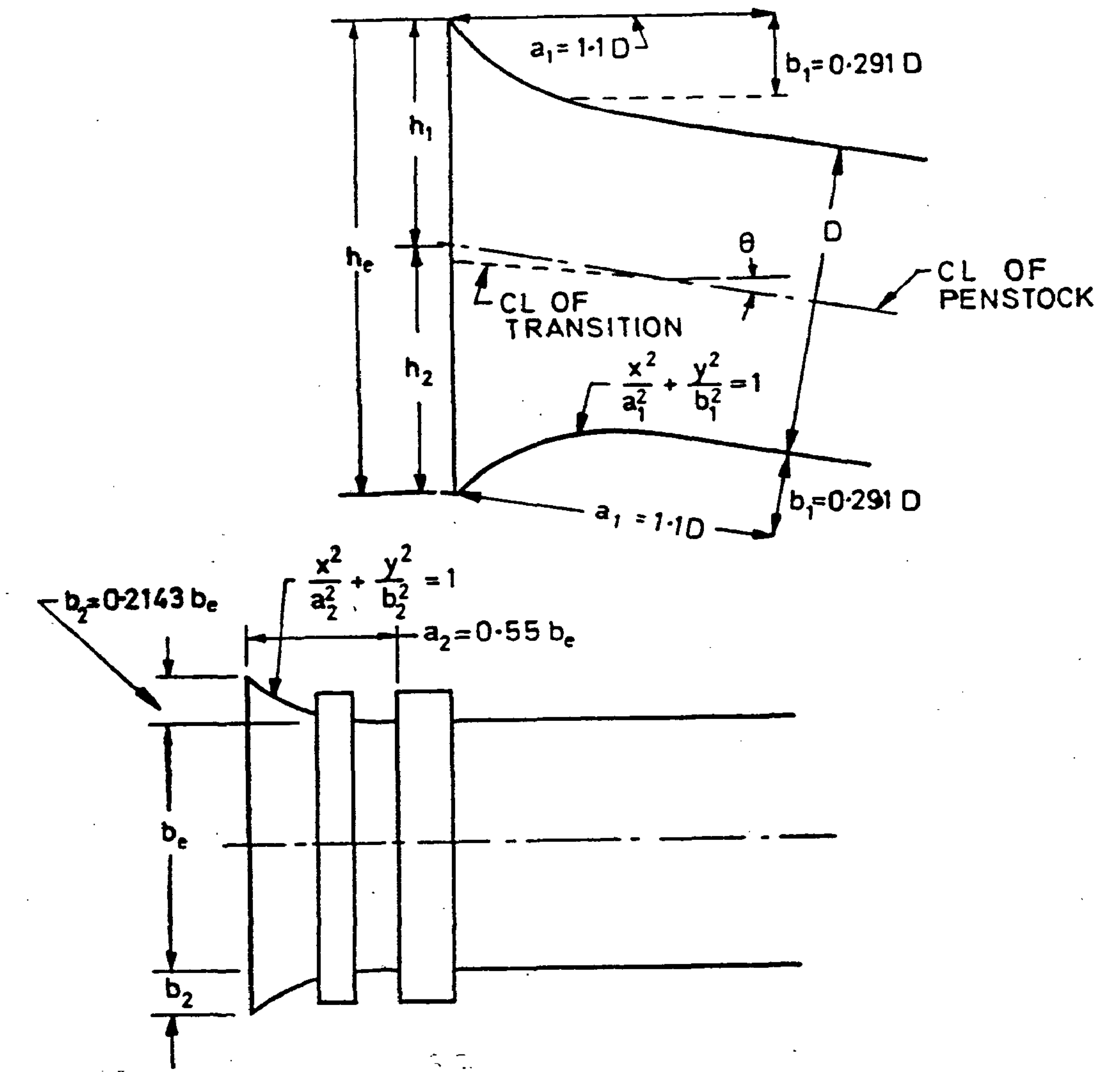


FIG. 6.2 BELLMOUTH DETAILS

Center line upper edge :

$$h_1 = D\ [(1.21\ \tan^2\phi + 0.847)^{1/2} + \frac{1}{2\cos\phi} - 1.1\ \tan\ \phi]$$

Centre line to lowere edge :

$$h_2 = D\ [(\frac{0.791}{\cos\phi} + 0.077\ \tan\ \phi)]$$

Total heighto of $h_e = h_1 + h_2$

This should be les than or equal to total height of the tank, as calculated in 6.5.1.

Width of opening be $= \frac{\text{Area of opening}}{h_e}$

6.4.4 Shape of Opening

The inlet should be streamlined to minimize the losses, the profile of the roof and follor shuld approximate to that of a jet from the horizontal slot. the profile is generally an ellipse given by the following equation

$$\frac{X^2}{(1.1D)^2} + \frac{Y^2}{(0.291D)^2} = 1$$

6.4.5 Transitions

In order to obtain hydraulically efficient design of intake transitions from rectangular, section ot a circular section conduit, the transition shall be designed in accordance with the following requirements :

a) Transition or turns shall be made about the centre line of flow and shall be gradual

b) Side walls shall not expand at a rate greater than 5^{o} from the centre line of the mass flow.

c) All slots or other necessary departures from the neat out line shal normally be outside the transition zone.

6.4.6 Central Line of Intake

Formation of vortices at the intake depends on a number of factors, such as approach geometry, velocity at the intake, geometrical features of trash rack structure etc. To prevent vortices, the centre line of intake shall be located below the minimum drawdown level such that the minimum cover of water over roof of the intake should be 0.3 be when the flow is normal to the intake face. In the case of oblique flows, this cover may be raised upto h_e, where h_e is the entrance height. The requirement of water cover may be reduced with the provision of anti-vortex devices.

The gate chamber continues over the penstock intake and the penstocks themselves. The gradually converging passage in deeply founded front wall of the forebay forms a transition to the penstock.

Provide some free board also it can be taken 1.15 to 0.3 m in mini-micro hydel projects. After fixing these dimensions stability of forebay should be checked for the following conditions.

Case - I :

No water in the forebay, dry earth pressure acting from outside, live load on top slab and bottom reaction due to self wt. of concrete.

Case II :

Forebay full, no earth pressure from outside, live load on top slab and bottom reaction due to self weight of concrete and that of water inside.

Case III :

No water in the forebay, 50% saturated earth pressure acting from outside, live load on top slab, bottom reaction and uplift pressure due to 50% saturation.

The design for various members is carried out for the worst moments obtained as a result of the above three conditions. As the moments for different cases are of reverse nature, equal rainforcement is provided on both faces and moment of resistance checked considering members as doubly reinforced. Analysis should also be done for earthquake condition and sections should be checked for the moments so determined. In the design the allowable stresses in concrete are increased by $33\frac{1}{3}\%$ as per provision of code :

6.5 SPILLWAY DESIGN

As mostly ogee spillway is provided in forebay its hydraulic design is discussed as follows in detail. We have to fix the length of spillway crest for which following equations is used.

$$L_{eH} = L_c - 2\,[N.K_p + K_a]\,H_d \qquad (6.1)$$

where,

L_{eFF} = Effective length

L_e = Clear waterway of the spillway crest

N = Number of piers

K_p = Pier contraction co-efficient

K_a = Abutment contraction co-efficient

H_e = Total design head on the crest including velocity head.

Now assume clear water way, no of piers etc. then find out effective length, and check the discharge to be passed by the following formula

$$Q = C.L_e\ H_e^{3/2} \tag{6.2}$$

where,

C = Co-efficient of discharge

Q = Discharge to be passed.

If this discharge is equal to the design discharge of power channel then assumed clear length of waterway is O.K. otherwise change this and find out the correct length.

After this D/S and U/S profiles of ogee spillway are designed as follows :

The D/S profile of ogee spillway is represented by the following equation

$$x^n = K.H_d^{n-1}.\ Y \tag{6.3}$$

where,

x,y are the co-ordinates of the points on the crest profile with its origin at the highest point C of the crest.

H_d = Design head including velocity head

K and n = constants depending upon the slope of the u/s face.

Values of K & n are shown below in Table 6.1.

Table 6.1

Slope of the U/S face of spillway	K	n
Vertical	2.0	1.85
1:3 (1H:3V)	1.936	1.836
$1:1\frac{1}{2}$ $(1H:1\frac{1}{2}V)$	1.939	1.810)

For U/S profile having vertical u/s face following euation will be used :

$$Y = \frac{0.724(x + 0.27\ H_d)^{1.85}}{H_d^{0.85}} + 0.126\ H_d - 0.4315\ H_d^{0.375} * (X + 0.27\ H_d)^{0.625} \quad (6.4)$$

The U/S profile extends upto $x = -0.27\ H_d$

6.6 DESIGN EXAMPLE

As an ilustration the following example covers the detailed hydraulic design of forebay, intake and spillway for Kush Mini-micro hydel project (2 MW) as per details in Figure 6.3.

6.6.1 Design of Forebay

Design discharge	= 1.3 m^3/sec
Storage required	= 2 minutes
Capacity of tank	= 1.3 x 2 x 60 = 156.0 m^3
Assume water depth	= 2.25 m
Area required	= 156/2.25 = 69.33 m^3
Providing width	= 6.0 m
∴ Required length	= 69.33/6 = 11.55 m say 12.0 m
Size of tank	= 12.0 x 6.0 x 2.25 m

6.6.2 Design of Intake

For high head normal contraction = 40%

So take coefficient of contraction C_c = 0.6

$Q = \tan^{-1} = 15.38^o$

Penstock diameters 0.762 metre

$Q = \tan^{-1} \frac{1.1}{4} = 15.38^o$

$$\text{Opening area} = \frac{\text{Penstock area}}{C_c \text{ Cos } \phi} = \frac{\frac{\pi}{4} \times 1.0^2}{0.6 \times \text{Cos } 15.38} = 1.36 \text{ sq.m.}$$

h_1 = Height of centre line to upper edge

$$= D \left[(1.21 \tan^2\phi + 0.0847)^{1/2} + \frac{1}{2 \cos\phi} - 1.1 \tan \phi\right]$$

$$= 1.0 \left[(1.21 \tan^2 15.38 + 847\ )^{1/2} + \frac{1}{2 \cos 15.38} - 1.1 \tan 15.38\right]$$

$$= 1.0 \left[(1.21 \tan^2 0.076 + 0.0847)^{1/2} + \frac{1}{2 \times 0.964} - 1.1 \times 0.275\right].$$

$$= 1.0 \left[0.42 + 0.216\right] = 0.636 \text{ m}$$

h_2 = Height of centre line to lower edge

$$= D \left[\left(\frac{0.791}{\text{Cos}\phi} + 0.077 \tan \phi\right)\right]$$

$$= 1.0 \quad \left[\frac{0.791}{\text{Cos } 15.38} + 0.077 \tan 15.38\right] = 0.841 \text{ m}$$

Total height of opening 'h_e' = $h_1 + h_2$ = 0.636 + 0.841 = 1.477 m

say 1.48 m

Width of opening 'b_e' = $\frac{1.36}{1.48}$ = 0.92 Adopt 0.92 m x 1.48 m opening

Minimum water cover over roof of the intake = 0.3 h_e = 0.3 x 1.48

= 0.44 m

Actually provided = 0.75 m (safe)

Provide free board = 0.3 m

Provide storage for silt below penstock = 0.25 m.

Total minimum height of forebay tank required = 1.48 + 0.44 + 0.30 (Free board) + 0.25 = 2.47 m < 3.0 m. Hence safe

- A water cover of 0.75 m is provided over the crown of penstock intake to prevent any air entry into the penstock
- A localy depressed pocket of 2 m x 2 m x 2.15 m is provided at the penstock intake in the forebay to reduce the structure height as well as to reduce the cost.
- A storage of 0.25 (depth) is provided below the penstock invert for possible silt deposition which would be removed manually through silt flushing pipe of 150 mm dia.
- A mild slope for the silt movement is provided in the floor towards the silt flushing outlet.

6.6.3 Design of Spillway

Design discharge 'Q' = 1.3 m^3/sec

Taking length of spillway = 6.0 m (equal to forebay tank width).

Finding out approximate value of H_e

Taking $L_e \approx L_c$ = clear waterway = 6.0 m

We know

$$Q = C.L_e\ H_e^{3/2} \quad \text{(Taking C = 2.0)}$$

$$1.3 = 2.0 \times 6 \times H_e^{3/2}$$

$$H_e^{3/2} = \frac{1.3}{2x6} = 0.108$$

H_e = 0.23 m say 0.25 m

$Q = 2.0 \times 6.0 \times 0.25^{3/2} = 1.5\ m^3/sec$ which is more than design discharge so it is O.K.

Downstream profile :

The d/s profile for a vertical u/s face is given by

$$x^{1.85} = 2\, H_d^{0.85} \,.\, y$$

$$Y = \frac{x^{1.85}}{2(H_d)^{0.85}} = \frac{x^{1.85}}{2 \times (0.25)^{0.85}}$$ ($H_e = H_d$ because velocity head is negligible)

$$Y = \frac{x^{1.85}}{0.616}$$

Let first find out the tangent point :

Let the d/s slope of the spillway 0.7 H:1V

Hence, $\frac{dy}{dx} = \frac{1}{0.7}$

Differentiating equation of the d/s profile w.r.t. 'x' we get

$$\frac{dy}{dx} = \frac{1.85\, x^{0.85}}{0.616} = \frac{1}{0.7}$$

$$\therefore\ x^{0.85} = \frac{0.616}{1.85 \times 0.7} = 0.476$$

$x = 0.418$ m

$$y = \frac{0.418^{1.85}}{0.616} = 0.328 \text{ m}$$

The co-ordinates from $x = 0$ to $x = 0.418$ m are worked out in the following table

Table 6.2

x metres	$y = \frac{x^{1.85}}{0.616}$ metres
0.1	0.023
0.2	0.083
0.3	0.175
0.418	0.323

U/S Profile :

As value of H_d is very less so upstream profile can be taken vertical.

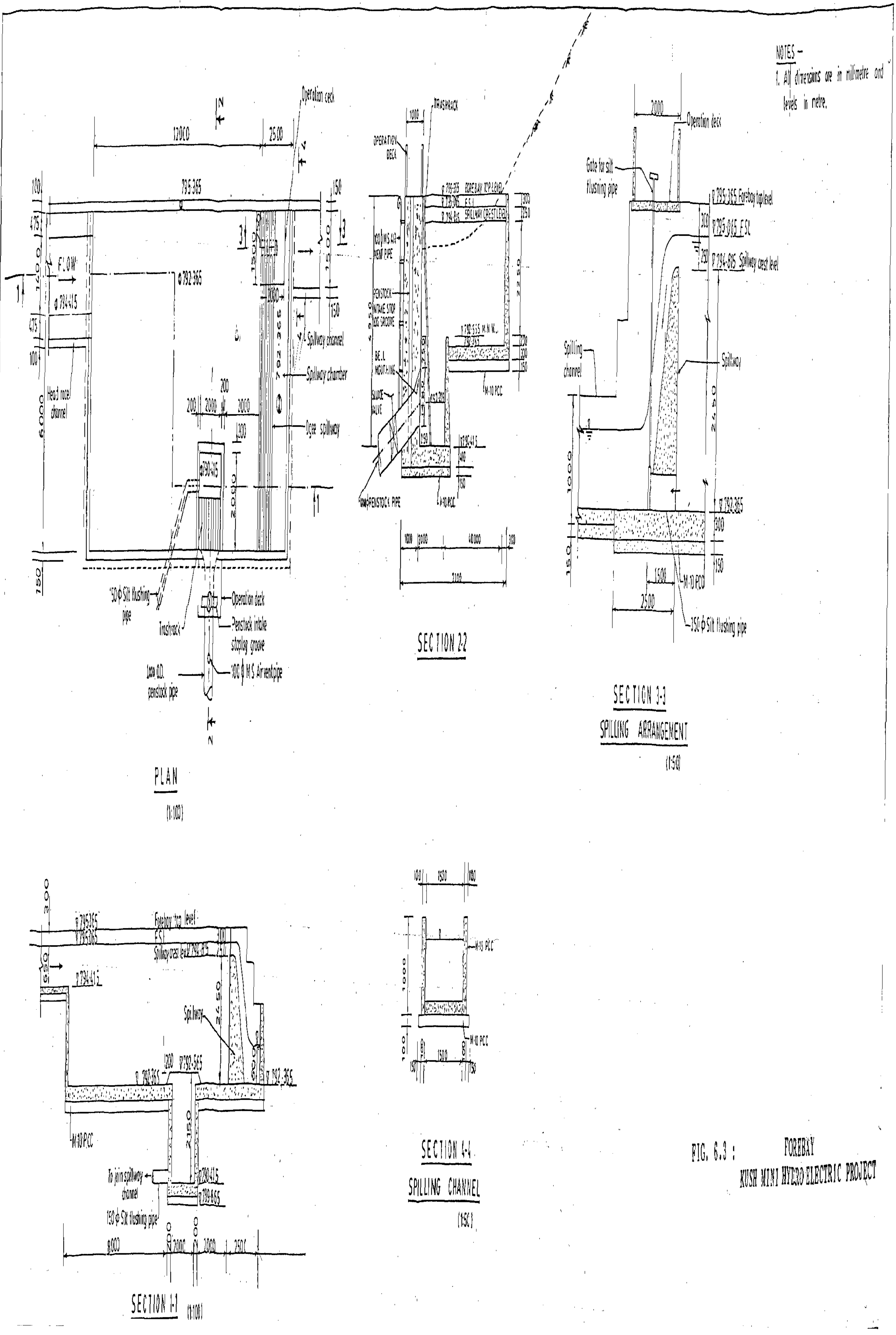


FIG. 6.3 : FOREBAY
KUSH MINI HYDRO ELECTRIC PROJECT

CHAPTER - 7

SURGE TANK

7.1 GENERAL

In mini-micro hydel projects pressurised water conductor system should be avoided as much as possible. Still if it is not possible to avoid pressurised water conductor system then it may be necessary to provide a surge tank at the meeting point of nearly horizontal head race conduit and steeply sloping penstock.

On abrupt, reduction of the electrical load turbine governor, in its eforts to maintain steady synchronus speed, will rapidly cause the turbine guide vanes to close, so that there will be abrupt reduction of flow. On the other hand, when an additional load is put on the machine, the governor causes the wicket gates to open and additional water is required to maintain additional supply. The surge tank is a reservoir which furnishes space, immediately available for the acceptance or delivery of water to meet the requirement of load changes. Surge tank also serves to reduce the water hammer pressure within the penstock under condition of load rejection and acceptance.

The surge chamber works in three ways -

1. Its introduction shortens the distance between the turbine inlet and the nearest free water surface and thereby greatly reduces the intensity of water hammer waves. Moreover, the water hammer effects in the tunnel upstream of the surge chamber are reduced to such a degree that they can in many

cases be neglected in practical design, and only the relatively short length of conduit below the surge chamber must be designed to withstand them. This permits substantial economy in pressure conduits.

2. With a reduction or rejection of load the surge chamber acts as a relief opening into which the main conduit flow is partly or wholly diverted. The water level in the chamber therefore rises until it exceeds the level in the main reservoir, thus retarding the main conduit flow and absorbing the surplus kinetic energy.

3. Finally when starting up or when increasing load, the chamber acts as a reservoir which will provide sufficient water to enable the turbines to pick up their new load safely and quickly. In doing so the water level in the surge chamber falls below the steady state level. Sufficient head is thereby created to accelerate the flow of water in the conduit until it is sufficient to meet the new demand.

Different types of surge tanks are as follows :

i) The simple surge tank

ii) Restricted orifice surge tank

iii) The differential surge chamber

iv) Special type of surge tanks

  a) shaft with lower expansion chamber

  b) shaft with upper expansion chamber

  c) shaft with upper and lower expansion chamber

  d) shaft with expansion chambers and spilling arrangements.

7.2 ALTERNATIVES FOR SURGE TANK

In mini-micro projects to make them economical there should be an approach to find out cheaper alternative of surge tank. Following are some alternatives.

i) **Jet deflector** : In mini-micro hydel schemes where impulse turbines are used, the pressure surges due to sudden load reduction canbe reduced/eliminated with the provision of jet deflector. This would eliminate the need of surge tank, though this would involve some wastage of water.

ii) **Water wasting relief valves** : The pressure rise resulting from water hammer can be limited by the use of pressure relief valves put at the downstream end of penstock or in the scroll-case itself.

The action of relief valve is to open rapidly and to release some water but to close slowly. It does not function during the small variations of the load and also it is not effective if the load increases.

China is using relief valves since 1965. Initially its control system used to be mechanical and now it have been changed to hydraulic type. This results in a big reduction of time lag (0.1 second, or close to zero second) and an enhancement in the reliability. Langyuan hydropower station (China) has 1,950 m long diversion tunnel and steel penstock, working at a head of 83 m. There are 3 units, totalling 5,600 kw, with the use of hydraulic-control relief valves in place of surge shaft. Since 1976, the relief valves have operated more than 1000 times. All had functioned accurately, ensuring the safety of the system and hydroelectric units. Owing to the substitution of surge chamber by the relief valves, there was a saving of 360,000 yuan, and shortening of construction period by one year. This economises on the investment of mini-micro hydropower stations and also enables them to be commissioned ahead of time. Figure 7.1 shows layout of wasting water relief valve.

iii) **Inclined pipe laid on the hill side slopes** : Inclined pipe laid on the hill side slope with or without a suitable tank at a suitable height extending above the full supply level of water at intake could serve as a surge chamber. Figure 7.2 shows layout of inclined pipe laid on the hil side slopes.

iv) **Air vessel** : Air vessel also called closed surge tanks reduces the change of water level due to confined air pocket above. These can surve as surge controllers & will be economical then conventional surge tanks because hight of tank will be reduced. Air pressure is maintained above the top level of water in the tank which will be absorb the increase in presure. Figure 7.3 shows the layout of air vessel.

7.3 LOCATION OF SURGE TANK

A surge tank is always located as close as possible to the powerhouse in order to reduce the length of penstock to a minimum and preferably on high ground to reduce the height of tower. Hence roughly

$$L < 5H \qquad (7.1)$$

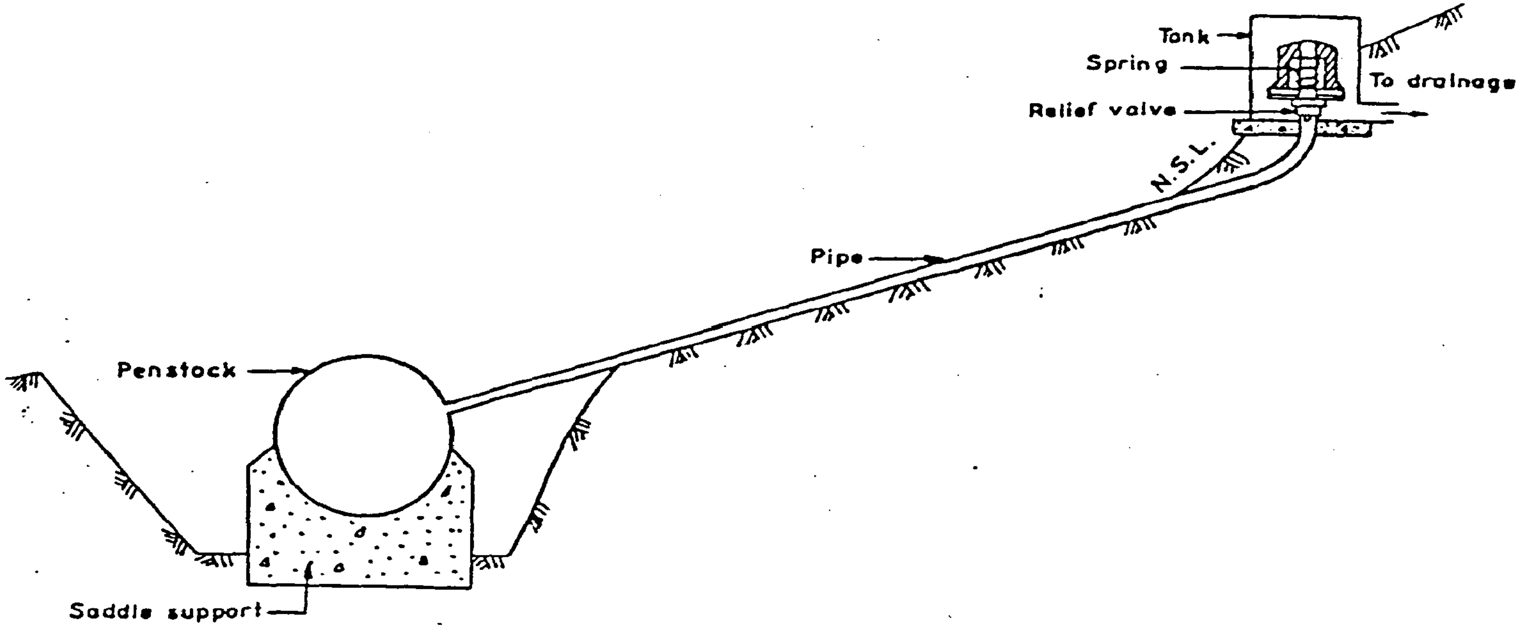


FIG. 7.1 : TYPICAL LAYOUT OF WATER WASTING RELIEF VALVE

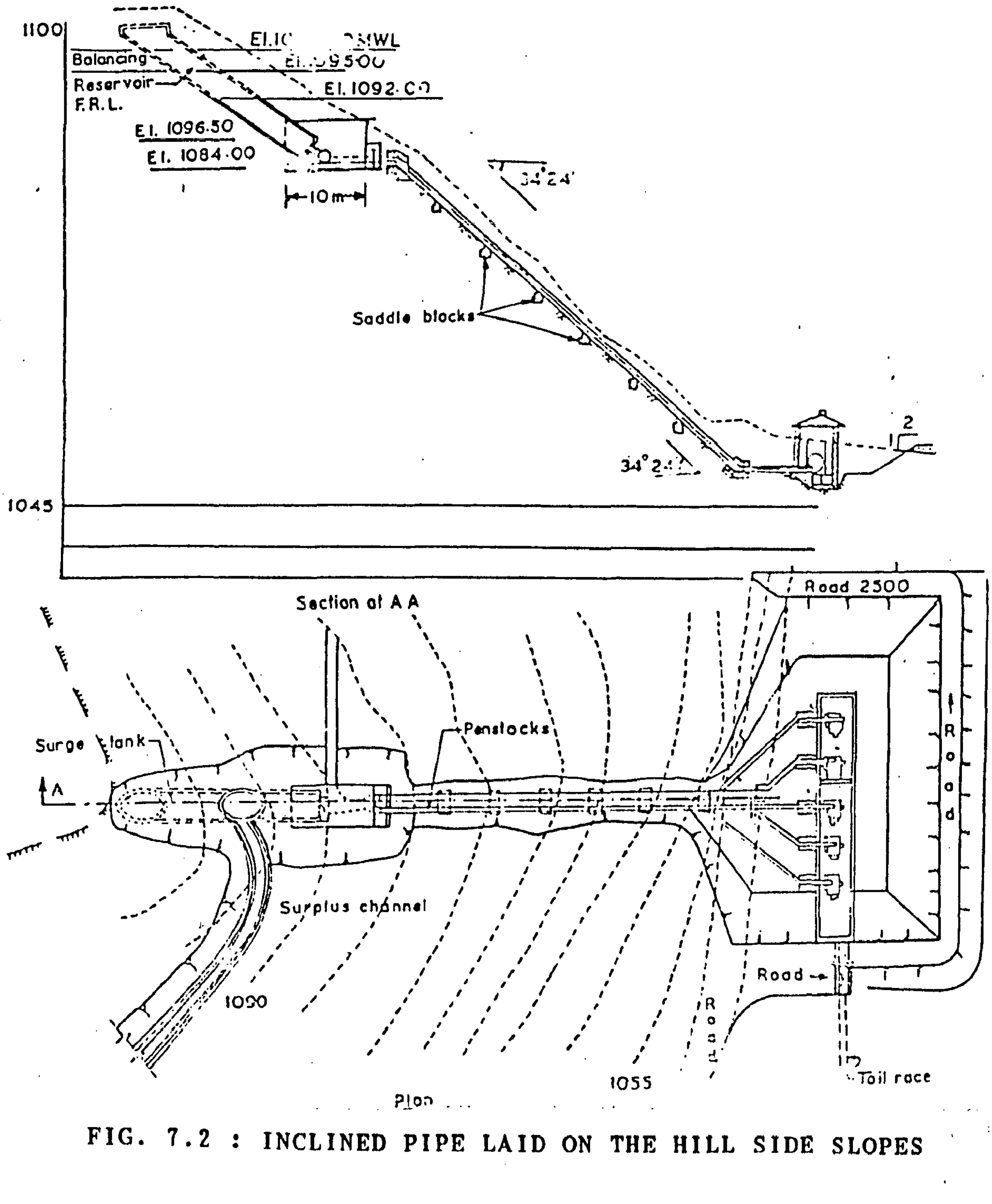


FIG. 7.2 : INCLINED PIPE LAID ON THE HILL SIDE SLOPES

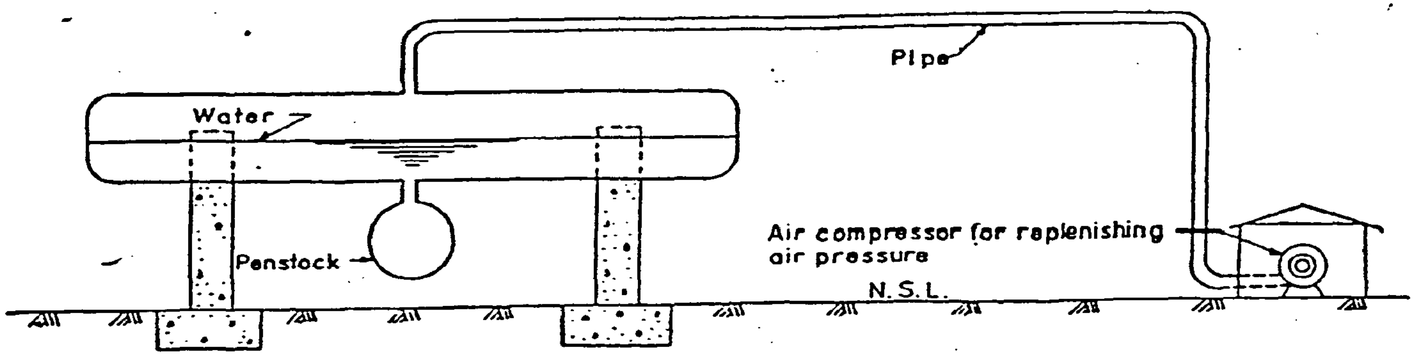


FIG. 7.3 : TYPICAL LAYOUT OF AIR VESSAL

where,

L = unregulated length i.e. the slope length of the conduit measured from the machines to the nearest free water surface.

H = gross head on machines.

For any given set of operating conditions the pressure conduit has therefore a maximum length, within which it must either terminate in the open forebay of a canal or reservoir or must be provided with a surge chamber. The rule given in equation 7.1 is only approximate and the unregulated length should always be made as short as practicable. When the conduit is very long a surge chamber is often only practical method of obtaining satisfactory operating conditions.

7.4 BASIC DESIGN CRITERIA

Surge tank design must comply with the following conditions.

1) The surge chamber must be so located that pressure variations caused by water hammer are kept within acceptable limits.
2) The chamber must be stable i.e. the surges resulting from load changes must be damped naturally and must not under any condition be sustained or amplified.
3) The chamber must be of such size and so proportioned that :
   a) It will contain the maximum possible upsurge
   b) The lowest downs surge should not allow air to be drawn into the penstock.
   c) The range of surges must not be great enough to cause undesirably heavy governor movements

7.5 LOADING CONDITIONS TO BE ADOPTED

In the event of certain electrical or mechanical failures the entire load would be rejected instantly; this might occur with the turbines at full load and with the reservoir at any level. Full load rejection must therefore be considered in every case. It is usual to consider full load rejection under two conditions :

i) With the reservoir at its maximum level, in which case the maximum upsurge level will govern the top level of the chamber.

ii) With the reservoir at its lowest drawdown level in which case the first downsurge level may control the bottom level of the chamber if air drawing is to be avoided.

The condition when load is "thrown on" are not quite so clear cut as those for load rejection. When the turbine is started from rest, it is first run up to synchronous speed on no load. Under this condition it will take 10 percent of its full load flow. The generator is then synchronised with the system and brought steadily to the desired load. Although it is desirable that the turbine should pick up its load without undue delay, the combined operation of running-up and loading is not instantaneous. If the station contains a number of manually controlled turbines, the operation of starting them all up would necessarily take an appreciable time. If, however, the turbines are automatically controlled, or if they were all running at part load, an increase to full load could take place rapidly.

An isolated station is not likely to be subjected to instantaneous large increases of load, particularly if the system load is of diverse nature. On the other hand, if one of a number of interconnected stations is suddenly cut out as result of the fault, the load would instantaneously be spread over the remaining stations in definite proportions dependent upon their governor settings. Each of these stations would then be subjected to sudden increase of load, and the amount of increase would depend upon the total number of stations involved and the amount of load rejected by one of them.

7.6 DESIGN ASSUMPTIONS

1) Friction and other losses must be assumed. Low friction value should be assumed when upsurge is being studied and a high one for determining down surge levels.

2) Losses due to friction, and those at intakes, trashracks and bends etc. are usually taken as proportional to the square of the conduit velocity. Losses due to change of direction of flow at the surge chamber, and part losses, must be taken as proportional to the square of the chamber velocity except in the particular case of full load rejection when they may be expressed as proportional to the square of the tunnel velocity. Friction losses in the chamber itself are always ignored.

3) It is usual to assume that the surges result from instantaneous changes of the flow through the turbine. This is a safe assumption, though the governor or relief valve, as

the case may be take an appreciable time to function.

4) When the guide-vane movements are assumed to be gradual it is usual to assume that variation of discharge is a linear function of time, whatever may be the corresponding variation of guide vane opening.

The assumption of instantaneous guide vane movements errs on the side of safety, i.e. the calculated surges will exceed those which will occur in practice. When computing surges for full-load rejection, it is not unreasonable therefore to assume that, though the electrical load is entirely lost, the flow is only reduced by 90 percent. A similar assumption can be made when full load is thrown on.

Turbine efficiency is usually taken as constant over the range of heads and loads involved.

7.7 DESIGN OF SIMPLE SURGE TANK (Ref. Fig. 7.4)

For mini-micro hydroelectric projects only simple surge tank is provided. So design of simple surge tank has been covered here with an example in para 7.8.

Different notations used in the design of different type of surge tanks are as under.

A_S = net crosssectional area of surge tanks

A_t = Cross-sectional area of head race tunnel

A_{th} = Thoma area of surge tanks

D = Diameter of head race tunnel

F_s = Factor of safety over A_{th}

g = Acceleration due to gravity

H = Gross head on turbines

H_o = Net head on turbines

h_{fp} = Total head loss in penstock system

K = Ratio of total power generated by the station to that of grid

K_m = A constant depending upon the efficienes of turbine and generator

L = Length of head race tunnel

P = Power gneragted

Q = Discharge through turbines

t = time, reckoned from the start of transient

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

V_1^* = Velocity of flow in tunnel at any instant, upstream of surge tank

v_1^* = Velocity of flow in tunnel corresponding to initial steady flow (upstream of surge tank)

Z = Water level in surge tank measured positive above reservoir level

Z_m = Maximum surge level above maximum reservoir level

β = Co-efficient of hydraulic losses such that $h_f = \beta V_1^2$

ϕ = Ratio of area of surge tank to that of conduit = A_s/A_t

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* To be taken as positive when the flow is downstream and negative when the flow is upstream.

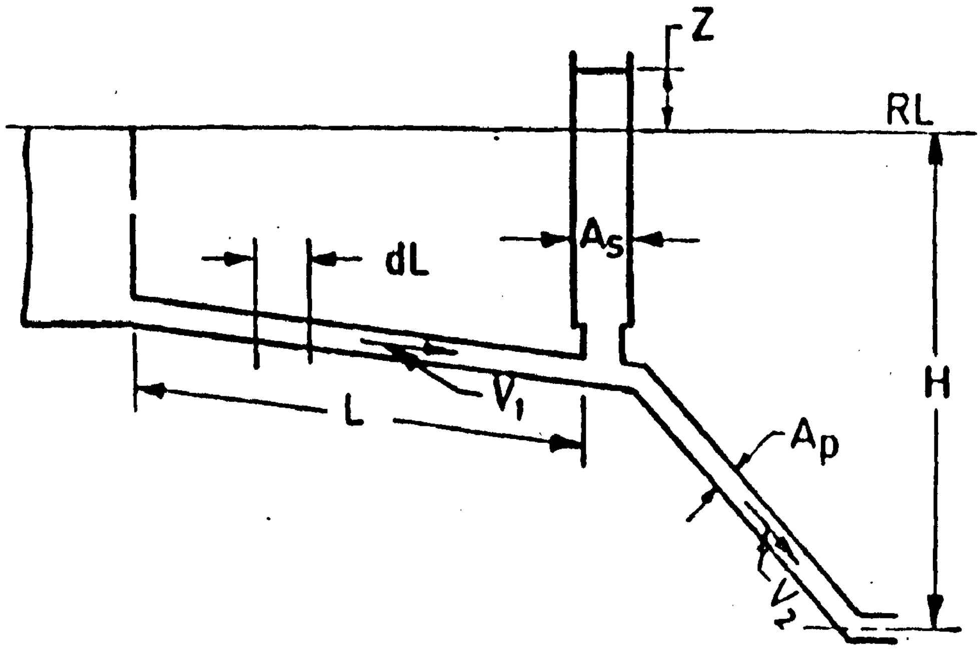

FIG 7.4: SIMPLE SURGE TANK

7.7.1 Area of Surge Tank

To ensure the hydraulic stability of surge tanks, its area is governed by Thoma criteria. Considering a station to be working in isolation, Thoma criteria is :

$$A_{th} = \frac{LA_t}{\beta V_1^2 H_o} \cdot \frac{V_1^2}{2g} \quad \text{(minimum valve of } \beta \text{ should be used)} \tag{7.2}$$

$$\text{and } \beta V_1^2 = \frac{FL_1^2}{2gD} + \text{other losses in tunnel system} \tag{7.3}$$

If the reservoir level varies significantly, A_{th} may vary appreciably between minimum and maximum reservoir levels and in such cases the maximum value of A_{th} is to be adopted.

If it can be ensured that the power station is to operate in a grid, the stabilizing effect of grid may be taken into account and the area of surge tank in that case may be calculated from the following formula.

$$A_s = A_{th} \left(1-1.5\ (1-K)\ \right) \tag{7.4}$$

7.7.2 Factor of Safety

Area of surge tank calculated by equation (7.2) or (7.4) gives the minimum area of surge tank. The following factor of safety on the area should be used to ensure rapid rate of damping in practical operation. In case of simple surge tank a factor of safety of two should be used.

7.7.3 Extreme Surge Levels

The extreme water level in the surge tank for the conditions enumerated in subhead 7.5 are determined by integrating the equations of mass oscillations for the respective tanks. These equations are non linear. They can be integrated mathematically as well as graphically.

In case of simple surge tank the following equations of flow should be satisfied

$$\frac{L}{g}\ \frac{dv_1}{dt} + Z \pm \beta V_1^2 + \frac{V_1^2}{2g} = 0 \quad \text{(Dynamic equation)} \qquad (7.5)$$

NOTE : Positive valve of βV_1^2 shall be used when flow is downstream and negative when it is upstream.

$$A_t V_1 = A_s \frac{dZ}{dt} + Q \quad \text{(Continuity equation)} \qquad (7.6)$$

$$P = K_m Q \left(H + Z + \frac{V_1^2}{2g} - h_{fp} \right)$$

(Power equation (7.7)

The maximum surge level for the case of complete load rejection may be determined from the following formula.

$$\frac{L}{2g\ \phi\beta^2 V_1^2} - \frac{Z_m}{\beta V_o^2} - \frac{L}{2g\ \phi\beta^2 V_o^2} \left[\bar{e}\ \frac{2g\phi}{L} + \beta V_o^2 \right] \qquad (7.8)$$

7. 7.4 Height of Surge Tank

The top of the surge tank may be kept 1.5 m above the heighest water level while the bottom of the surge tank may be kept 2.0 m below the lowest surge level determined by calculation or by model experiments for the worst conditions. The extra provision on the top may be in the form of a solid retaining wall. Wherever the surge tank is brought to the surface it is covered all around by such a wall. It should be ensured that there is a sufficient water cover over the tunnel to avoid vortex formation even at the lowest water level in the surge tank. If the topography near the surge shaft area permits, suitable arrangement for spillover may be considered so as to restrict surge height, which in some cases may prove to be more economical.

7.8 DESIGN EXAMPLE

As an illustration the following example covers the detailed calculations for maximum surge by different methods in simple surge tank.

Length of pressure tunnel $L = 3500$ m

Diameter of pressure tunnel $D = 3.0$ m

Net cross-sectional area of tunnel $A_t = \frac{\pi}{4} \times 3^2 = 7.07$ sq.m.

Discharge 'Q' = 20 cumec

Cross-sectional area of surge tank $A_{st} = 300$ sq.m.

Damping factor $m = \frac{2gA_{st}}{LxA_t}\,\beta = \frac{2x9.81x300}{3500x7.07}\,\beta = 0.238\,\beta$

The coefficient of roughness K is for concrete linings with sand cement mortor rendering.

$K = 90.9$ (from table)

Hydraulic radius = $R = \frac{D}{4} = \frac{3}{4} = 0.75$ m

Resistance factor of the tunnel (By Manning - Strickler Formula)

$$\beta = \frac{L}{K^2 R^{4/3}} = \frac{3500}{90.9^2 x 0.75^{4/3}} = 0.62$$

$\therefore$ Damping factor m = 0.238 x 0.62 = 0.147/m

For computing steady state loss y_o (distance of hydrodynamical equlibrium level below reservoir level) the flow velocity must be determined first.

$$V_o = \frac{Q}{A_t} = \frac{20}{7.06} = 2.83 \text{ m/sec}$$

Accordingly $y_o = \beta v_o^2 = 0.62 \times 2.83^2 = 4.97$ m

Surges can be obtained with the following initial constant

$A_o = my_o = 0.147 \times 4.97 = 0.73$

Values obtained from the curve (refer Fig. 7.5) are as follows,

$(mY_1) = 0.78$ corresponding to $Y_1 = -5.31$ m

$(mY_2) = 0.51$ corresponding to $Y_2 = +3.47$ m

$(mY_3) = 0.38$ corresponding to $Y_3 = -2.58$ m

As a check let us determine the surge limit (By Pattantyus method)

$$Y = \frac{LxA_t}{2g\beta . A_{st}} = \frac{3500 \times 7.07}{2x9.81x0.62x300} = 6.77 \text{ m}$$

Saying that the first negative surge (upsurge) Y_1 may not exceed 6.77 m computing finaly the period of later heimicycle.

$$\frac{T}{2} = \pi \sqrt{\frac{A_{st} xL}{A_t xg}} = 3.14 \sqrt{\frac{300x3500}{7.07x9.81}} = 386 \text{ sec.}$$

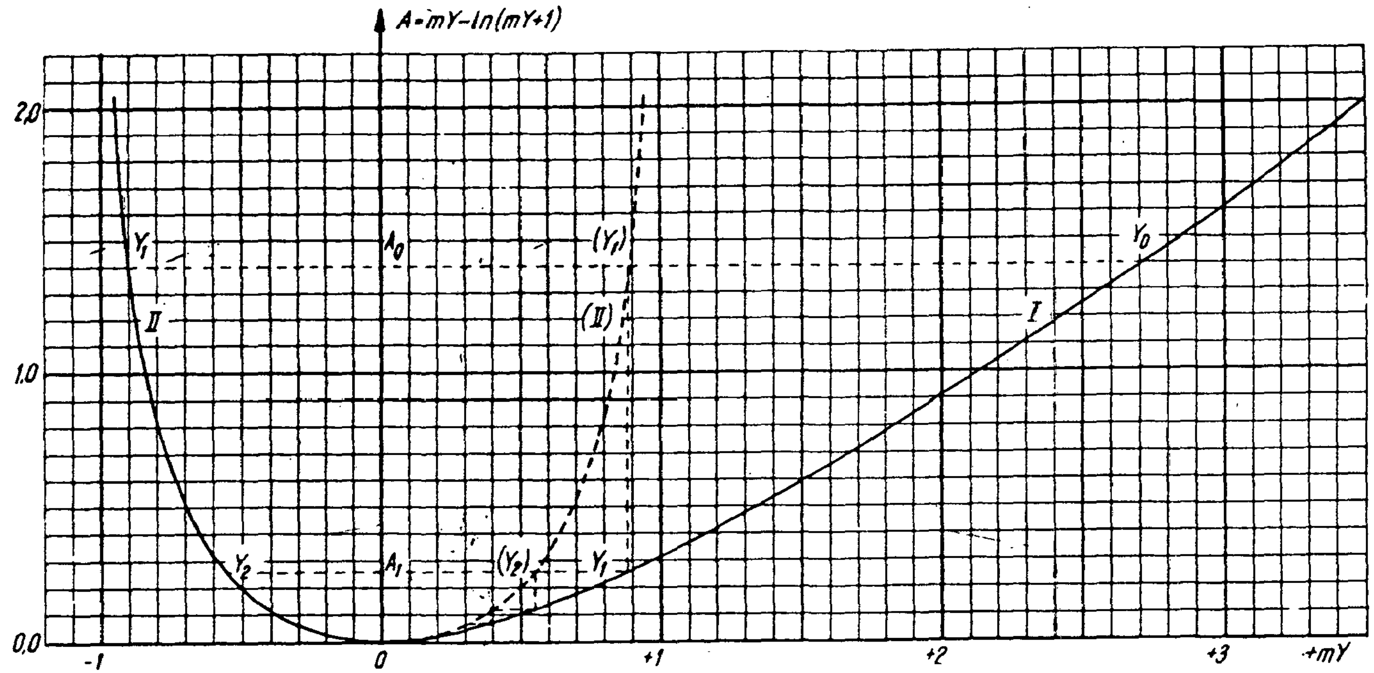

FIG. 7.5 : CONSTRUCTION OF SURGE PERFORMANCE CURVES FOR A SIMPLE SURGE TANK (CURVE OF MISES)

CHAPTER - 8

PENSTOCK

8.1 GENERAL

Conduits carrying water from the surge tank or directly from a reservoir/forebay to the generating machine are known as Penstocks. These are pressure conduits since they are subjected to appreciable internal pressure. The internal preasure varies being minimum at the upstream and to maximum at the junction with the scroll case.

The penstocks can be constructed of the following materials depending upon the head to which the penstock is subjected, the topography of the terrain, the discharge to be handled, availablity of material and economics etc.

(1) Steel (2) Cast Steel (3) Banded Steel

(4) Asbestos Cement (5) Wood Staves (6) R.C.C

As R.C.C. penstocks are most economical for mini-micro hydropower schemes, therefore after a brief description of the above said types of penstocks detailed structural design of R.C.C. penstock is covered in para 8.8.

8.2 STEEL PENSTOCKS

Penstocks for heads from o to 800 m are fabricated from steel. Small diameter penstocks say upto 0.5 m size are manufactured as a seamless pipe from mild steel of tensile strength varying from 5500 kg/cm^2 to 6500 kg/km^2. Higher size subjected to high pressure penstock are manufactured from plate steel.

8.3 CAST STEEL PENSTOCKS

Cast steel is usually used for special components only such as bends, wye pieces, expansion joints and flenges etc. Cast iron being of limited tensile strength, brittle and rigid is rarely used now a days.

8.4 BANDED STEEL PENSTOCKS

The penstocks subjected to very high pressure are generally fabricated as banded steel pipes. Banded pipe are of thin wall strengthened with the help of hoops of high strength steel slipped over them.

8.5 ASBESTOS CEMENT PENSTOCKS

For low capacity power stations prefabricated asbestos cement penstocks ay be used successfully. Essentially asbestos fibre serves the same purpose in concrete as reinforcing bars: it carries tensile forces. As per practical experience, asbestos cement pipes are capable of withstanding tensile stress upto 200-300 kg/cm^2

8.6 WOOD STAVE PIPES

Wood stave pipes are made of 4 to 5 m long prefabricated sections upto 60 cm diameter. But the type built continuosly at site have been applied for head upto 100 metres and upto 5 metres in diameter. Carefully machine-shaped staves are held together with steel bands. Though wood stave pipes have normally low cost, less thermal conductivity and have ease in both construction and errection, their use has been rare because for the following.

(i) Being subjected to decay

(ii) Defects in the wood

(iii) Insufficient water tightness at joints

(iv) Low resistance against buckling

8.7 R.C.C PENSTOCKS

R.C.C. can be used for installations of relatively low head and low capacity. Reinforced concrete penstocks may be either exposed or buried. Those having diameter smaller than 1.0 m are always prefabricated, while larger ones are cast in place. R.C.C penstocks offer considerable saving in steel. Because of the considerably lower heat conductivity of concrete reinforced-concrete pressure pipes are superior to steel pipes as for as freezing is concerned. So to make mini-micro hydel project more econimical R.C.C. penstocks are best suited. These are discussed in some detail in the following paragraphs.

Following are the two main types of prefabricated reinforced concrete pipes.

(i) Penstock with conventional reinforcement

(ii) Prestresed pipes

i) The reinforcing cage of conventinel concrete pipe is built up of longitudinal and internal as well as external hoop reinforcement, in which the longitudinal bars play no structural role unless the pipe is designed to act as a beam. Considerable longitudinal moments may arise if the pipe is supported at two ends only. Tensile stress due to longitudinal bending may be particularly high thus a significant longitudinal reinforcement may be required for relatively long pipes havng small diameters. Longitudinal bending moments may arise also in continuously supported pipes. Such moments may be higher, if the backfill cover is deep. This is due to the reason that distribution of the soil reaction may not be uniform.

ii) Prestressed reinforced concrete pipes are manuactured simply by tension wrapping a steel wire of high tensile strength on the outside of a conventional reinforced concrete pipe. Prestressing wire is, in general wrapped under tension on the pipe. The prestressing force depends upon the ultimate strength of the wire. End rings are used to anchor the ends of the wire. The diameter of tensioning wires varies from 2 to 6 mm. After wrapping, the helically wound prestressing wire is given a 1 to 2 cm thick mortar coating applied by spraying or guniting for protection against corrosion. Concrete of at least 400 to 500 kg/cm^2 cube strength should be used for pressure pipes.

Prefabricated R.C.C. pipes are generally used for operating pressures upto 5 to 15 kg/cm^2. These are manufactured with diameters 'between 0.3 m to 3 m at common lengths between 3 to 7 m. Wall thickness from 8 to 25 cm are common for prefabricated pressure pipes.

8.8 STRUCTURAL DESIGN OF REINFORCED CONCRETE PENSTOCKS :

Stress in R.C.C. penstocks should be investigated in two direction i.e. longitudinally and transversally. The method of support i.e. whether resting on cradles or placed continuously upon the ground, is decisive as regards the magnitude of longitudinal bending moments developing in the penstock. Longitudinal stresses will be significantly lower in the case of latter arrangement and may even be neglected entirly.

Pipes concreted at site are reinforced to take care of hoop tensile stresses, with the main reinforcement located either around the invert or around the soffit. The longitudinal bars serve to take temperature and shrinkage stresses and to hold the circular reinforcement in place.

In precast pipes, reinforcing bars are spaced at equal distances around the perimeter. This results in a certain wastage of steel, as part of the reinforcement is not utilized to its full load bearing capacity.

Hoop (circumferential) stress in a pipe having an internal radius r_i and a shell thickness δ is

$$\sigma = \frac{P}{\rho^2 - 1} \left(1 + \rho^2 \frac{r_i^2}{r^2}\right) \text{ kg/cm}^2 \qquad (8.1)$$

where P = internal pressure in kg/cm^2

r = the distance of the fibre under consideration from the centre line of the pipe and

$$\rho = \frac{r_i + \delta}{r_i} \qquad (8.2)$$

Stresses in the extreme fibres are obtained by substituting $r = r_i$, and $r = r_i + \delta$ respectively. Thus stress at the internal face

$$\sigma_1 = \frac{P}{\rho 2-1} (1 + \rho^2) = P \frac{(\rho^2+1)}{(\rho^2-1)} \tag{8.3}$$

whereas the stress at the external face

$$\sigma_2 = \frac{P}{\rho^2-1} (1+1) = P \frac{2}{\rho^2-1} \tag{8.4}$$

Stress at the internal fibre exceeds that in the external one exactly by the internal pressure P

$$\sigma_1 - \sigma_2 = P \frac{\rho^2+1}{\rho^2-1} - P \frac{2}{\rho^2-1} = P$$

In principle, the aim to be followed in designing is to create a pipe free of cracks.

8.8.1 Design Steps

Consider a total reinforcement of a_{st} cm^2 for unit (1 cm) length of pipe, i.e. the total cross-sectional area of the inner and outer hoop reinforcement per cm length.

Step - 1

Determining the tensile stress at both inner and outer fibres and checking that the higher stress i.e. at the inner face does not exceed the tensile strength of the concrete. As such the condition

$$\sigma_1 \leq \sigma_{ct}$$

must therefore be statisfied. The tensile strength of good quality concrete may be from 1/8 to 1/10 of the crushing strength. The more exact analysis can be done by taking into account the contribution of the reinforcement in resisting internal loads. Consequently the resulting tensile stresses in the concrete will be smaller than computed by equation 8.3. In order to allow for the effect of reinforcement, tensile stresses may be calculated by using the reducing factor

$$\lambda = \frac{A_c}{A_c + 8\ A_{st}} \qquad (8.5)$$

where, A_c = area of concrete cross-section.

A_{st} = Area of the reinforcing steel. Considering a pipe section of 1 cm length, this reduces to

$$\lambda = \frac{\delta}{\delta + 8a_{st}} \qquad (8.6)$$

where, δ = wall thickness in cm

a_{st} = cross-sectional area of hoop reinforcement per one centimetre pipe length. So the tensile stress in concrete may be obtained by reducing the values of equation 8.3 and 8.4 as

$\lambda\sigma_1$ and $\lambda\sigma_2$ respectively.

Then safety against cracking will be expressed as

$$\lambda\sigma_1 \le \sigma_{ct} \qquad (8.7)$$

In case of burried pipes if depth of cover is h and the unit weight γ_1 acts as a uniformly distributed load

$q = \gamma_1\ h \quad kg/m^2$

will act on the pipe section of length b cm induces a bending moment

$$M = \pm \frac{qr^2 b}{4} \text{ kg cm} \tag{8.8}$$

at the invert at the soffit and at the springing.

The B.M. acting upon 1 cm length of pipe is thus

$$M_o = \pm \frac{qr^2}{4} \text{ kg cm} \tag{8.9}$$

where r should be taken as mean radius $= r_i + \frac{\delta}{2}$ (m) (8.10)

Let us consider a hoop reinforcement per cm of pipe length a_{st} arranged symmetrically in the pipe wall of thickness δ, i.e. one half of the reinforcing steel $a_{st/2}$ located at the inner face, While other half located at the outer face of the wall, both at a distance of δ′/2 from the centre. (The concrete cover at both faces is δ/2 - δ′/2). With this arrangement the sectional modulus of the 1 cm long pipe wall is

$$S_o = \frac{\delta^2}{6} \quad 8a_{st} \quad \left(\frac{\delta'}{2} \right)^2 \quad \frac{2}{\delta} = \frac{\delta^2}{6} + 4_{ast} + \frac{\delta'^2}{\delta} \text{ cm}^2 \tag{8.11}$$

The stress due to vertical earth load is

$$G_m = \pm \frac{M_o}{S_o} \text{ kg/cm}^2 \tag{8.12}$$

Burried pipes should therefore satisfy the following condition

$$\lambda \sigma_1 + \sigma_m \le \sigma_{ct} \tag{8.13}$$

Step - 2 :

Check the cross-sectional area of the reinforcement. The hoop reinforcement a_{st} should alone be capable to carry the total hoop force due to internal water load on 1 cm length of pipe.

$$P_o = pr_i \quad kg/cm$$

Therefore the criterion as regards the stress in the reinforcement is

$$\sigma_{st} = \frac{Pr_i}{a_{st}} \leq \sigma_{sto} \qquad (8.14)$$

Conventional reinforced concrete pipes should be used upto loads not exceeding a certain limit. The internal load is expressed as the product of head and clear diameter. According to M. Mainardis the limit is generally accepted as

$$H \times d = 200 \text{ sq.m}$$

For internal loads higher than this limit presetressed concrete pipes should preferably be used.

8.9 ANCHORS AND SUPPORTS

Penstocks of fixed and semi-rigid type are secured in place at points by anchor blocks. Anchors are located either at vertical or horizontal bends in the line. There will be three types of anchor blocks possible as discussed below.

1. Forces transmitted by the adjacent penstock sections to an anchor block located at a point where the change in the slope of the terrain, and thus the bend in the penstock, is concave when viewed from above, tend to displace the anchor block over the terrain. These forces, however, have a component normal to the terrain, the magnitude of which

depends upon the angle of bend and this favourably influences the stability conditions of the anchor block.

2. At bends, where the penstock deflects downwards, i.e.which are convex when viewed from above, the resultant of forces acting upon the anchor has a component directed away from the terrain and at such places large concrete volumes are usually required to ensure the stability of the rock.

3. Intermediate anchor blocks spaced at least at 100 to 150 m are necessary at long straight penstock sections. These blocks are subjected to a force caused by pipe thrust and approximately parallel to the terrain. This is an intermediate case between cases 1 & 2, as far as stability is concerned.

Loads and Forces Acting on Anchor Blocks

f = coefficient of friction of penstock on intermediate supports,

f′ = Friction of expansion joint/m of circomference = 1.5 μW.H.e

u = coefficient of friction between expansion joint packing and liner

w = until weight of water in kg/m^3

H = Maximum head at the point including water hammer in metres,

e = Packing length in metres

A = Cross-sectional area of penstock at anchor in m^2

A′ = Cross-sectional area of penstock above upper reducer in m^2

A" = Cross-sectional area of penstock below lower reducer in m^2

t = Thickness of penstock shell in mm

Q = Discharge in m^3/sec

V = Velocity in m/sec

g = acceleration due to gravity in m/sec^2

P = dead weight of pipe from anchor uphill to expansion joint in kg.

W = weight of water in pipe in kg

P' = dead weight of pipe downhill from the anchor to expansion joint in kg

W = Weight of water in pipe in p' in kg

α_u = angle with the horizontal of pipe above anchor

α_d = angle with the horizontal of pipe below anchor

p = weight of pipe and contained water from anchor to adjacent uphill intermediate support in kg.

p' = weight of pipe and contained water from anchor to adjacent downhill intermediate support in kg

d = inside diameter of penstock in mm

a = cross-sectional area of penstock at uphill expansion joint in m^2

a' = cros-sectional area of penstock at downhill expansion joint in m^2

K = self weight of anchor in kg.

The anchor blocks shall be designed for the following loads and forces.

Load/Force	Number by which indicated in the Figure 8.1
1. Hydrostatic force acting along axis of penstock on each side of bend, $F_s = WAH$	1
2. Dynamic force acting against outside of bend, $F_d = \frac{QWV}{g}$	2
3. Force due to dead weight of pipe from anchor uphill to expansion joint, tending the pipe to slide downhill over piers, $d_u = P \sin \alpha_u$	3
4. Force due to dead weight of pipe from anchor downhill to expansion joint tending the pipe to slide downhill over piers, $D_d = P' \sin \alpha_d$	4
5. Sliding friction of pipe on intermediate supports due to expansion or contraction uphill from anchor, $S_{pu} = f \cos \alpha_u (P+W- \frac{p}{2})$	5
6. Sliding friction of pipe on intermediate supports due to expansion or contraction downhill from anchor, $S_{pd} = f \cos \alpha_d (P'-W'- \frac{p'}{2})$	6
7. Sliding friction of uphill expansion joint, $S_{eu} = \frac{f' \pi (d+2t)}{1000}$.	7
8. Sliding friction of downhill expansion joint $S_{ed} = \frac{f' \pi (d+2t)}{1000}$	8
9. Hydrostatic pressure on exposed end of pipe in uphill expansion joint, $F_u = \frac{WH\pi t\ (d+t)}{10^6} = WaH$	9
10. Hydrostatic pressure on exposed end of pipe in downhill expansion joint, $F_d = W.a'H$	10
11. Longitudinal force due to reducer above anchor $L_u = WH\ (A'-A)$	11
12 Longitudinal force due to reducer below anchor $L_d = WH\ (A-A'')$	12

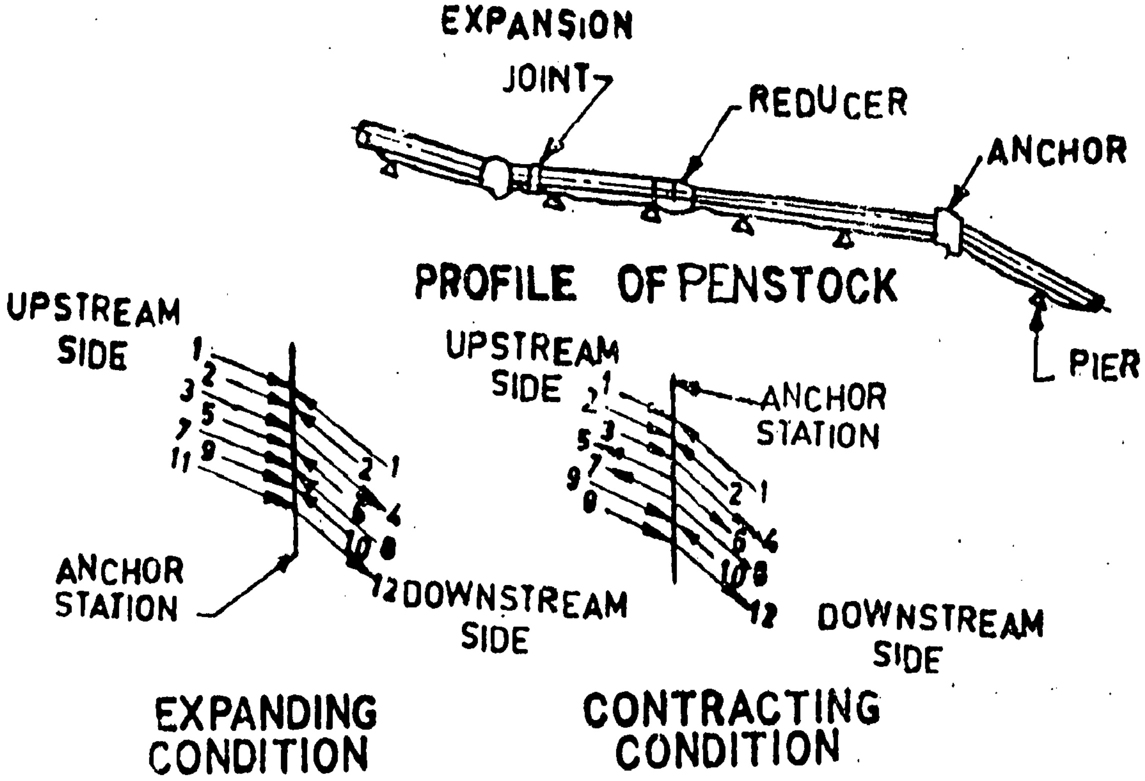

FIG. 8.1 : ACTION OF LOADS AND FORCES ON ANCHOR BLOCK

In addition to the above forces the anchor blocks shall be designed for the self weight of anchor + seismic force in accordance with the intensity of earthquake forces. (Refer IS:1893).

In loads and forces specified above shall be assumed to act in the directions shown in the the figure above. The anchor block shall be designed for the following conditions for both without seismic forces and with seismic forces.

i) Penstock empty + expansion
ii) Penstock empty + contraction
iii) Penstock full + expansion
iv) Penstock full + contraction

For seismic condition, the allowable stresses in the concrete and steel can be increased by $33\frac{1}{3}\%$.

Value of 'f' i.e. coefficinet of friction of penstock and piers may be taken as below :

i) Steel on concrete	0.60
ii) Steel on concrete with asphalt roofing paper in between	0.50
iii) Steel on steel, rusty plates	0.50
iv) Steel on steel, greased plates	0.25
v) Steel on steel with two layers of graphite service sheets in between	0.25
vi) Roller supports deteriorated	0.10
vii) Roller bearings, well lubricated	0.01
viii) Steel on phospher - bronze, well lubricated	0.15
ix) Steel on phosphor-bronze, unlubricated	0.25

The coefficient of friction between the packing and the liner may be taken as 0.26.

8.9.1 Design Criteria for Anchor Block

i) Anchor blocks should preferably be founded on firm rock.

ii) The foundation of anchor blocks shall be designed so that the maximum pressure on foundation shall not exceed the allowable bearing pressure. The permissible bearing capacity may be increased by $33\frac{1}{3}\%$ in case of seismic condition.

iii) When the ground profile is sloping, the safe bearing capacity shall be suitably reduced.

iv) The angle set up by resultant with ground shall not be less than 30° for stability of soil below anchor.

v) Anchor blocks shall be designed safe against sliding on foundation. The sliding friction factor computed by dividing the total horizontal forces by total vertical forces shall be loss than that given below :

Type of Surfaces	Sliding factor
Concrete on rock	0.50
Concreteon gravel	0.40
Concrete on sand	0.33
Concrete on clayey soil	0.25

vi) If tension occurs in some portion of the anchors foundation, the bearing pressure in the compression area should be suitably adjusted.

vii) Even if no reinforcement requirement is indicated by the design calculations, nominal reinforcement must be provided.

8.10 ECONOMICAL DIAMETER OF PENSTOCK

The economical dameter of a penstock required to carry a discharge 'Q' is the one at which annual costs due to the greater investment do not exceed the annual value of resulting increment energy output.

These are different methods to find out the economical diameter. Though there are some emprical relations also for determination of economical diameter they are not suitable for all cases because they have been arrived at after taking a number of assumptions. Trial and error method has been discussed here as under.

A) Trial and Error Method

i) Graphical method

ii) Computer aided method

1) Graphical Method

In this method cost of penstock per metre length is calulated for a trial value of diameter 'D'. Then annual cost is calculated taking into account the intrest on capital cost, supervision cost, of O & M cost and contengeney. Also the cost of loss of power terrif per year due to frictional loss in ponstock is added to get the total annual cost. For number of values of diameter 'D', the total annual cost/m length is calculated then a graph of diameter versus total annual cost is plotted. Corresponding to the towest point on the curve will give the economical diameter of the penstock.

ii) Computer Aided Method

In this method same procedure is adopted as in case of graphical one except plotting of graph. Here all the calculations are done with the help of computer only as follows.

Thickness of penstock wall 't' = $\frac{0.1\ HD}{2\sigma}$ m

Where H = Design head

D = Diameter of penstock

σ = allowable stress in penstock material

Taking efficiency of joints as 0.95

$$t = \frac{0.1\ HD}{2\sigma \times 0.95}\ \text{m or}\ \frac{10\ HD}{2 \times \sigma \times 0.95}\ \text{cm (Add 0.15 cm for rusting etc.)}$$

Cost of penstock/m length = π (D+t) t x ρ x cp.

Where ρ = specific gravity of penstock, matertal (used steel in this computer program)

Cp = Cost of penstock per ton of steel for taking extra steel in joint and stiffeners in consideration, increase the weight of steel by 10%.

Finished cost A = 1.1 x CP

Total annual cost of penstock/m length

$$TAC = A\ (1+s)\ (1+c) \times \left(\frac{1}{y} + AL+0 \right)$$

Where

S = Supervision cost in percent of finished cost A

C = Contengency in percent of finisted cost

Y = Life of penstock in years

AL = Rate of intrest in percent

O = Operation and maintenance cost as percent of finished cost

Now head loss through penstock

$$H_F = (f/D) \times \frac{v^2}{2g}$$

Cost of loss of energy per year TAL = 9.81 x Q x H(365xOPxPLF)

Where

Q = Design discharge

OP = Operation period of machine per day

PLF = Power load factor

F = Overall efficiency of power scheme

U = Power terrif per KWH.

Therefore total cost = TAC+TAL

The above calculation for total cost for different diameters can be done with the help of computer using FORTRAN 4 program. For this tentative initial value of diameter is entered in the program and then for each increase of diameter, cost is calculated and compared and finally diameter for minimum cost is selected.

8.11 DESIGN EXAMPLE

As an illustration the following example covers the economical diameter and water hammer pressure calculations, for Kush Mini HEP (2 MW). Computer program and results of economical diameter has been attached in the subsequent pages.

(From computer program)

Economical diameter = 1.00 metres

and velociety V = 1.66 m/sec

Thickness of penstock 't'= 0.55 cm

Water Hammer Pressure

Net head = 216.5 metres

Let us calculate water hammer pressure head

Dia D = 1.00 m, $\quad$ t = 0.55 cm

$$\therefore D/t = \frac{1.0 \times 100}{0.55} = 181.818$$

Velocity of travel of pressure wave 'a' 2784.22 ft/sec or 848.85 m/sec (From velocity of travel curve) (on Page 134a Fig 8.2)

$$\text{Pipe line constant } K = \frac{aV_o}{2gH_o} = \frac{848.85 \times 1.66}{2 \times 9.81 \times 216.5} = 0.332$$

Let closure time = 10 second

Length of penstock = L = 580 m

$$\text{Time constant } N = \frac{aT}{2L} = \frac{848.85 \times 10}{2 \times 580} = 7.318$$

For K = 0.332, $\quad$ N = 7.318

From chart based on elastic - water column theory (ref. Fig 8.2)

Value of P = 0.123

$$P = \frac{gh}{aV_o} = \quad \therefore h = \frac{P \times aV_o}{g} = \frac{0.123 \times 848.85 \times 1.66}{9.81} = 17.67 \text{ m}$$

From Allievi Chart Method :

$$\text{Pipe line constant } \rho = \frac{aV_o}{2gH_o} = \frac{848.85 \times 1.66}{2 \times 9.81 \times 216.5} = 0.332$$

$$\text{Operation constant } Q = \frac{aT}{2L} = \frac{848.85 \times 10}{2 \times 580} = 7.318$$

$Z^2 = 1.08$ $\quad$ (From Allievi chart)
Fig 8.3 P-134-a

$$\text{where } Z^2 = \frac{H_o + h_{max}}{H_o}$$

$$h_{max} = (Z^2 \times H_o) - H_o$$

$$\therefore h_{max} = (1.08 \times 216.5) - 2 = 17.32 \text{ m}$$

So max height of water hammer pressure = 17.67 m.

```
$DEBUG
C       ************************************************************
C               PROGRAM FOR CALCULATION OF ECONOMIC DIAMETER
C                              OF PENSTOCK
C*      ************************************************************
C*
c       CP = Finished cost of Penstock per ton of steel (in Rs.)
C       U  = Power tarrif per kWh
C       Y  = Life of Penstock (years)
C       AI = Rate of interest in percentage
C       f  = Friction factor
C       E  = Overall efficiency of power scheme
C       C  = Contigency in percentage of finished cost
C       S  = Supervision cost in percentage of finished cost
C       O  = O & M cost in percentage of finished cost
C       Q  = Discharge in cumec
C       H  = Design Head in metres
C       CST= Allowable Tensile/Hoop stress of steel in Kg/sq.cm
C       PLF= Power load factor
C       OP = Duration of operation of machine per day in hours
C*      ************************************************************
c*
        COMMON TSTMIN,TST,Q,CP,U,Y,AI,F,E,C,S,O,PLF
        COMMON H,CST,OP
        OPEN(UNIT=1,FILE='PTCK.DAT')
        OPEN(UNIT=2,FILE='PTCK.RES',STATUS='NEW')
        READ(1,*) CP,U,Y,  AI,F,E,  C,S,O  ,Q,H,CST,  OP,PLF,EPS
        WRITE(2,2)
C*
        DD=0.5
        IT=0
        WRITE (*,32)
32      FORMAT (10X,'PLEASE ENTER THE STARTING DIAMETER TO TRIED (M)')
        READ (*,*) D2
        WRITE (2,7)
7       FORMAT (10X,'BASED ON THE FOLLOWING DATA :'/10X,27('-'))
        Z = 100
        AII = AI*Z
        EI = E*Z
        CI = C*Z
        SI = S*Z
        OI = O*Z
        PLF1 = PLF*Z
        SUM = 0
        WRITE (2,8) CP,U,Y,AII,F,EI,CI,SI,OI,Q,H,CST,PLF1,OP
8       FORMAT (10X,
     1  'Finished cost of Penstock /ton of steel ='F10.2,' Rupees',
     1  /10x,'Power tarrif per kWh                   ='f10.2,' Rupees',
     1  /10x,'Life of Penstock                       ='f10.2,' years',
     1  /10x,'Rate of interest in percentage         ='f10.2,' %'
     1  /10x,'Friction factor                        ='f10.2,
     1  /10x,'Efficiency of machine                  ='f10.2,' %',
     1  /10x,'Contigency in % of finished cost       ='f10.2,' %',
     1  /10x,'Supervision cost in % of finished cost ='f10.2,' %',
     1  /10x,'O & M cost in % of finished cost       ='f10.2,' %',
     1  /10x,'Discharge through Penstock             ='f10.2,' cumec',
     1  /10x,'Design Head of Penstock                ='f10.2,' metres',
     1  /10x,'Allowable Tensile/Hoop stress of steel ='f10.2,' Kg/cm2',
     1  /10x,'Power load factor                      ='f10.2,' %',
     1  /10x,'Duration of operation of machine per day='f10.2,' hours')
```

```
      WRITE(2,5)
      WRITE(2,6)
      CALL TCOST(D2,TCOST2,CTUNEL,TUNCOS,PLOSS,V)
C*
500   IT=IT+1
      WRITE(2,10)IT,D2,TST,TUNCOS,CTUNEL,PLOSS,TCOST2
      D1=D2
      TCOST0=TCOST1
      TCOST1=TCOST2
510   WRITE (*,*) IT, D1, sum
      D2=D1+DD
      CALL TCOST(D2,TCOST2,CTUNEL,TUNCOS,PLOSS,V)
      IF (TCOST2 .LT. TCOST1) GOTO 500
      SUM = SUM + 1
        IF (DD .NE. EPS) THEN
          IF (SUM.EQ.1) THEN
             D1=D2-2*DD
             TCOST1=TCOST0
          ELSE
             D1 = D2 - DD
          ENDIF
      LDD = IFIX((DD+0.05)*10)
      DD = FLOAT(LDD)/20
          IF (EPS.GE.0.05) THEN
           IF(DD.LT.EPS) DD=EPS
          ELSE
           IF(DD.EQ.0.05) DD=EPS
          ENDIF

             GOTO 510
        ENDIF
      WRITE(2,40)D1
      CALL TCOST(D1,TCOST2,CTUNEL,TUNCOS,PLOSS,V)
      TST=TST+0.05
      TSTMIN=TSTMIN+0.05
      WRITE(2,45)TST
      write(2,50)TSTMIN
      WRITE (2,55) V
      STOP

2     FORMAT(20X,37('*')/20X,'DETERMINATION OF ECONOMIC DIAMETER OF',
     1 /34X' PENSTOCK'/20X,37('*')/)
5     FORMAT(//5X,76('-')/5X,'ITER',2X,'DIAMETER',2X,'THICKNESS',1X,
     1 'PENSTOCK COST',2X,'ANNUAL COST ',1X,'POWER LOSS',2X,
     1 'TOTAL COST'/)
6     FORMAT(11X,'  (m.)  ',2X,'   (CM.) ',5X,' (Rs.) ',5X,'(Rs.) '
     1 ,5X,'(Rs.)  ',5X,'(Rs.)  '/5X,76('-')/)
10    FORMAT(5X,I3,F10.2,F10.2,E13.3,F15.2,E13.5,E12.5)
40    FORMAT(4X,77('-')//10X,'Economic diameter of Penstock      =',
     1 F9.2,' metres')
45    FORMAT(10X,'Thickness of Penstock                =',F9.2,' Cm.')
50    FORMAT(10X,'Minimum Thickness as per USBR code =',F9.2,' Cm.')
55    FORMAT(10X,'Velocity                             =',F9.2,' m/sc')
      END

C*    ************************************************************
C*    This subroutine calculates the cost of tunnel,loss of revenue
c*    due to power loss and total annual cost
c*    *************************************************************
```

```
c*
      SUBROUTINE TCOST(D,TOTAL,TAC,A,TAL,V)
      COMMON TSTMIN,TST,Q,CP,U,Y,AI,F,E,C,S,O,PLF
      COMMON H,CST,OP
      PI=3.1415927

      TSTMIN=(D*100.0+50.0)/400.0
c*           (Taking efficiency of longitudinal joint= 0.95 )
      TST=10*H*D/2/(0.95*CST)
         TST=TST+0.15
         IF (TST .LT. TSTMIN) TST=TSTMIN
      CP1=PI*(D+TST*0.01)*(TST*0.01)*7.85*CP
C*    (Weight of steel liner is increased by 10% for all butt joint
c*                     welds,stiffening and anchor ring)
      A=1.1*CP1
      OAC=A*(1+S)*(1+C)
      TAC=OAC*(1/Y+AI+O)
      V=Q/(PI/4.0*D*D)
      HF=F/D*(V*V/(2*9.81))
      TAL=9.81*Q*HF*(365.0*OP*PLF)*E*U
      TOTAL=TAC+TAL
      RETURN
      END
```

****
DETERMINATION OF ECONOMIC DIAMETER OF
PENSTOCK
****

BASED ON THE FOLLOWING DATA :

Finished cost of Penstock /ton of steel	=	30000.00 Rupees
Power tarrif per kWh	=	1.60 Rupees
Life of Penstock	=	100.00 years
Rate of interest in percentage	=	12.00 %
Friction factor	=	.01
Efficiency of machine	=	80.00 %
Contigency in % of finished cost	=	3.00 %
Supervision cost in % of finished cost	=	2.00 %
O & M cost in % of finished cost	=	5.00 %
Discharge through Penstock	=	1.30 cumec
Design Head of Penstock	=	223.00 metres
Allowable Tensile/Hoop stress of steel	=	3360.00 Kg/cm2
Power load factor	=	80.00 %
Duration of operation of machine per day	=	24.00 hours

TER	DIAMETER (m.)	THICKNESS (CM.)	PENSTOCK COST (Rs.)	ANNUAL COST (Rs.)	POWER LOSS (Rs.)	TOTAL COST (Rs.)
1	.25	.24	.487E+03	92.18	.16358E+06	.16367E+06
2	.75	.41	.253E+04	478.15	.67316E+03	.11513E+04
3	.50	.32	.133E+04	251.45	.51118E+04	.53633E+04
4	.75	.41	.253E+04	478.15	.67316E+03	.11513E+04
5	1.00	.50	.408E+04	772.28	.15974E+03	.93203E+03

Economic diameter of Penstock	=	1.00 metres
Thickness of Penstock	=	.55 Cm.
Minimum Thickness as per USBR code	=	.43 Cm.
Velocity	=	1.66 m/sc

FIG. 8.2 :

EXPLANATION.

P - Pressure Rise as a proportion of h_{max}
h - Pressure Rise or excess Head above normal, Feet.
h_{max} - Pressure Rise of instantaneous closure = aV_0/g. Feet
g - Acceleration of Gravity, Feet per second, per second
a - Velocity of Pressure Wave along Pipe. Feet per second - See graph.
V_0 - Velocity in Pipe near gate, corresponding to H_0 and Q_0. Feet per second
H_0 - Initial Steady Head near gate, corresponding to V_0. Feet.
Q_0 - Initial Steady Flow in pipe prior to start of gate closure corresponding to H_0. Cu. Ft. per sec.
T - Time of gate closure travel. Seconds
L - Length of pipe from gate to forebay or other point of relief. Feet

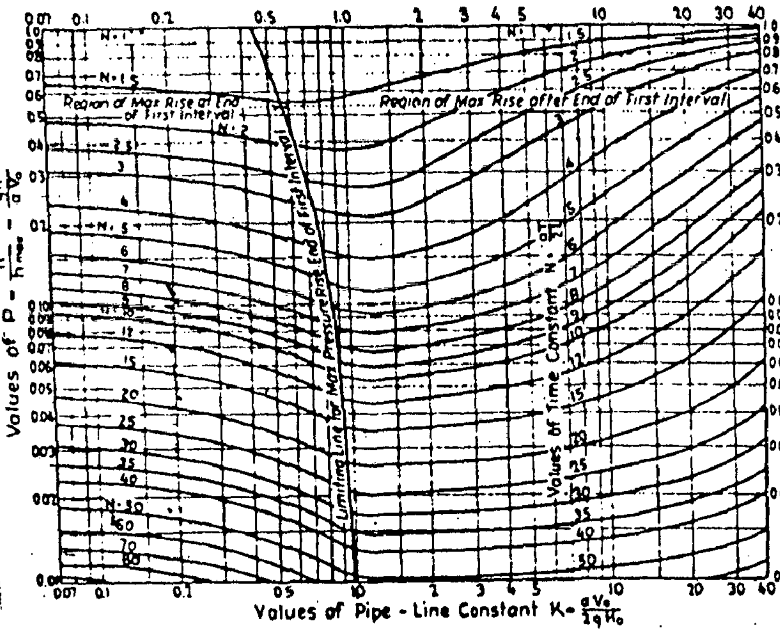

CHART SHOWING MAXIMUM PRESSURE RISE WITH UNIFORM GATE MOTION AND COMPLETE CLOSURE : BASED ON ELASTIC-WATER-COLUMN THEORY

NOTE: Ratio of Pressure Rise "h" to Initial Steady Head "H_0" determined from relation $2KP = h/H_0$

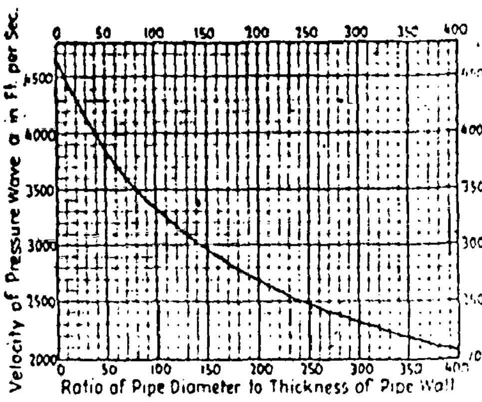

CHART SHOWING VELOCITY OF TRAVEL OF PRESSURE WAVE IN ELASTIC WATER COLUMN

Formula $a = \frac{4660}{\sqrt{1 + \frac{kd}{Eb}}} = \frac{4660}{\sqrt{1 + \frac{d}{100\,b}}}$

Where a - Velocity of Travel of Pressure Wave Ft. per Sec.
k - Bulk Modulus of Elasticity of Water = 294,000 Lbs per Sq. In.
E - Young's Modulus for Pipe Walls = 29,400,000 Lbs. per Sq. In. approx. for Steel.
b - Thickness of Pipe Walls, Inches
d - Inside Diameter of Pipe, Inches.

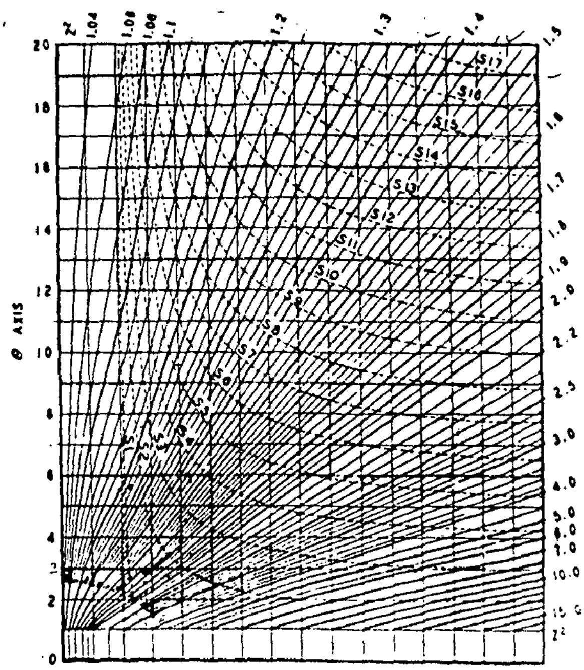

FIG. 8.3 : ALLIEVI DIAGRAM

CHAPTER - 9

CONCLUSION

9.1 GENERAL

A critical review of water conductor system for mini micro hydel projects has been carried out to find out, various design alternatives of different parts of the water conductor system. After studying the various alternatives, results can be concluded as below.

9.2 CONCLUSION

Drop type trench weir is one of the best alternative for the diversion of water from the river in hilly regions. Solid boulder structures can also be considered, where boulders are available in the river and rock is encountered in the river bed within one metre depth. If small dam is to be constructed, in that case earthen dam built of local material, or overflow earth dams which avoid the excavation of spillway or rubber dam technology can be used. In China for mini-micro hydropower projects the rubber dam technology is widely used. Compared with a gated dam structure, the cost of rubber dam can be decreased by 30 - 70%.

Desilting system is an essential component of mini-micro hydropower schemes, but tank size should be determined carefully so that with a small size tank reasonably silt free water could be obtained for power generation.

Power canal alignment selection is very important from economy point of view. The length of the canal system should be as small as possible. In uniform terrains the power canal can be designed with an open cross-section and in high ridges tunnel should be provided which will reduce the total length of the canal and head losses will also be smaller.

Forebay and balancing reservoir should be combined together wherever possible.

Pressurised water conductor system should be avoided as far as possible. Still if it is unavoidable alternatives of providing a surge tank, or a water wasting relief valve or an air vessel should be thouroughly studied. If relief valve is used then money as well as constructed time can be saved.

In penstocks a good economy can be achieved by using concrete pipes instead of steel. Concrete penstocks have been widely used in small hydropower stations in China. The use of concrete penstock has the advantages of reducing the cost, economizing on steel, simplifying the erection and avoiding the use of construction and expansion joints. A lighter pipe for a penstock is under study in China i.e., the application of fibre glass reinforced plastic pipe to mini-micro hydel projects is being put into practice. Some other points which should be considered for economy are as under.

i) R/M should be given to the local authorities

ii) As far as possible energy should be consumed locally and long transmission systems should be avoided.

iii) The design should be such that maximum use of locally available material could be made.

iv) The design should be simple so that most of the civil works could be got constructed by local man power.

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