DESIGN OF WATER TREATMENT PLANT FOR A HILLY TERRAIN

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

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JUNE, 2008

CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation entitled, ***"Design of Water Treatment Plant for a Hilly Terrain"***, in partial fulfillment of the requirement for award of the degree of ***Master of Technology in Water Resources Development***, submitted in Water Resource Development and Management Department, Indian Institute of Technology, Roorkee, is an authentic record of my own work, carried out during the period from July 2007 till the date of submission under the supervision of **Dr. M.L. Kansal**, Professor, WRD&M Department, Indian Institute of Technology, Roorkee.

I have not submitted the matter embodied in this dissertation for the award of any other degree or diploma.

Place : Roorkee
Dated : 5th June, 2008

(ISAAC LALCHHUANMAWIA)

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

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ACKNOWLEDGEMENT

I express my heartfelt gratitude to ***Dr. M. L. Kansal***, Professor, WRD&M Department, Indian Institute of Technology Roorkee, for his expert guidance, constant encouragement and valuable indefinite help throughout the period of preparing this dissertation work. Indeed, it would not be possible to carry out the thesis without his help.

I also express my sincere gratitude to ***Dr. S. K. Tripathi*** and ***Dr. Ram Pal Singh,*** both of them Head of WRD&M Department in different tenure, for extending various facilities during the course of this work.

I am also really very grateful to all the faculty members and staffs of WRD&M Department for their various kinds of help during my dissertation work.

I wish to express my thanks to the staffs of Central Library, Computer Centre and Architecture Department, who are part to the completion of my work.

All my colleagues and friends at IIT Roorkee are all important for my successful carrying out of the thesis. Hence, I am very thankful to them.

Without the permission and leave grant from the Government of Mizoram and Public Health Engineering Department of Mizoram to undergo M.Tech. Course, this thesis would not come into picture.

Lastly, I am even more indebted to my parents, wife and son, for their unusual sacrifice and constant encouragement throughout the course, which enables me to come out to this stage.

God is above All.

Roorkee, 5th June, 2008

ISAAC LALCHHUANMAWIA

#ABSTRACT

The primary objective of **water treatment plant** is to provide water that is safe for human consumption, is appealing to the consumer, non corrosive and non scaling. Moreover, the quantity of treated water should be sufficient to meet the maximum water demand of the population to be served at the design year. The design of water treatment plant mainly involves the selection of appropriate treatment processes/train and design of various treatment units selected. The treatment process can be as simple as involving only one unit operation or unit process, or it can be a combination of few or many unit operations and processes. The selection of a particular water treatment train mainly depends on raw water characteristics, desired level of treated water and the availability of funds, whereas, the capacity of treatment plant depends on design period, population and water demand. The selection and design of treatment processes also depends on state design criteria, topography and site conditions, hydraulic requirements, land area available, sludge disposal, and energy requirements. In the present study, the water treatment plant for **Almora town** of Uttarakhand has been discussed.

Almora is an important and fast growing **hill town** situated in the Kumaon region of Uttarakhand State of India having a population of 43960 in 2001 census. The town has been facing a shortage in water supply since last few years mainly due to increase in population, and if the present growth rate continues, there will be severe water crisis in the area in the near future. Hence, augmentation of the existing water supply scheme is projected by the authority by tapping Kosi River. The proposed intake point located at about 13 km from Matela, where centralized water treatment plant is proposed to be located.

Single well type of a river intake or Jack well is most suited by the site condition, hence, selected. Low lift pump house is proposed to be located on top of jack well intake, and raw water shall be pumped first to pre-sedimentation tank located at suitable place downstream of the intake. The clearer water from plain sedimentation tank is collected into raw water storage/sump which is then lifted to main water treatment plant at Matela through a static head of about 58 m. The analysis of Kosi raw water indicates that most of the parameters are within the allowable limits of Indian drinking water standards except that for turbidity,

iron, total coliform organisms and slight excess in chromium, dissolved oxygen and biochemical oxygen demand. The turbidity level of Kosi water is high and is as much as 1500 NTU during high flow season. Hence, conventional water treatment plant which consists of mechanical rapid mixer, clariflocculator and rapid singular media filter, preceded by pre-sedimentation and aeration is selected to remove or reduce the undesirable characteristics of raw water. Pre-sedimentation tank is longitudinal flow plain sedimentation tank. For aeration, gravity cascade aerator is selected. Chlorine is proposed for disinfection of filtered water and to ensure residual chlorine of about 0.2 mg/l in the distribution lines.

The design of various treatment units are carried out as per design parameters and guidelines given in I.S. Codes, GOI Manual on water treatment plant (1999). The various issues and challenges in the design of water treatment plant have been discussed in detail.

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References

ABBREVIATIONS & SYMBOLS

AC	Asbestos Cement
BDL	Below Desirable Level
BOD	Biochemical Oxygen Demand
BSI	Bureau of Indian Standard
C.I	Cast Iron
c/c	Center to Center
cm	Centimeter
CPCB	Central Pollution Control Board
CPHEEO	Central Public Health and Environmental Engineering Organization
D.I	Ductile Iron
DO	Dissolved Oxygen
ERW	Electric Resistance Welded
FB	Free Board
FF	First Floor
G.I	Galvanized Iron
GAC	Granular Activated Carbon
GL	Ground Level
GOI	Government of India
H.G.L	Hydraulic Gradient Line
HFL	High Flood Level
HP	Horse Power
IS	Indian Standard
Kg	Kilogram
KN	Kilo Newton
KW	Kilo Watt
LPCD	Liter Per Capita Per Day
lt.	Liter
m	Meter
m.s.	Mild Steel
m/s	Meter Per Second

m^3/s	Meter Cube Per Second
MCLG	Maximum Concentration Limit Goals
mg/l	Milligram Per Liter
MLD	Million Liter Per Day
mm	Millimeter
MPa	Mega Pascal
MPN	Most Probable Number
msl	Mean Sea Level
MWL	Minimum Water Level
N/m^2	Newton Per Meter Square
$NPSH_r$	Net Positive Suction Head Required
NRV	Non Return Valve
NTU	Nephelometric Turbidity Unit
NWL	Normal Water Level
PAC	Powdered Activated Carbon
ppm	Parts Per Million
PSI	Pound Per Square Inch
RCC	Reinforced Cement Concrete
RGF	Rapid Gravity Filter
RL	Reduced Level
T/m^3	Ton Per Meter Cube
THM	Trihalomethane
USEPA	United States Environmental Protection Agency
WHO	World Health Organization
WL	Water Level

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CHAPTER 1
INTRODUCTION

1.1 GENERAL

Access to safe drinking water is essential to health, a basic human right and a component of effective policy for health protection. It is important as a health and development issue at a national, regional and local level. It has been shown that investments in water supply and sanitation can yield a net economic benefit, since the reductions in adverse health effects and health care costs outweighs the cost of undertaking the interventions. Experience has also shown that interventions in improving access to safe water favour the poor in particular, whether in rural or urban areas, and can be effective part of poverty alleviation strategies. Hence, a sincere and collective effort should be made to make available raw water, potable and fit for supply to the general public for their domestic, industrial or any other uses by giving appropriate treatment. The basic objectives of water treatment plant design is the production of water that is safe for human consumption, is appealing to the consumer, non corrosive and non scaling, and is using facilities which can be constructed and operated at a reasonable cost. In addition to this, designer must also keep in mind all manner of legal mandates, as well as public concerns and environmental considerations, to provide an initial prospective of water works engineering planning, design, and operation.

Although the benefits of source protection are recognized as a 'first line of defense' in preserving water quality, all natural waters require some degree of treatment in order to meet modern drinking-water standards. The nature and extent of treatment required to be given to a particular water will, of course depends upon the nature and extend of impurities, and also upon the quality requirements for the intended use. So, the treatment process can be as simple as involving only one unit operation like plain sedimentation or unit process like simple chlorination, or it can be a combination of few or many unit operations and processes.

Most ground waters are clear and pathogenic-free and do not contain significant amounts of organic materials. In hilly areas, springs and streams which are not polluted by domestic wastes are normally clear and free from harmful substances. Such waters may often be used in potable systems with a minimal dose of chlorine to prevent contamination in the distribution system. However, in the present context when new diseases are still discovered and varieties of impurities are developing by the cause of human development, there can not be any natural water which can be granted as readily potable. Hence, before selecting any treatment process for design purpose, most care have to be taken in the characterization of raw water quality.

Conventional treatments which consist of coagulation, sedimentation and rapid filtration (Fig. 1.1) are normally adopted for medium to highly polluted surface water. In some instances, however, the raw water may contain inorganic constituents that are not removed by conventional treatment processes. Examples are ground waters with excessive dissolved solids and surface waters that contain organic compounds from domestic or industrial wastewaters or naturally occurring organics such as humic and fulvic acids or products of algae bloom. Such waters are treated by Non-conventional water treatment processes like ion-exchange, membrane, lime softening, and activated carbon processes.

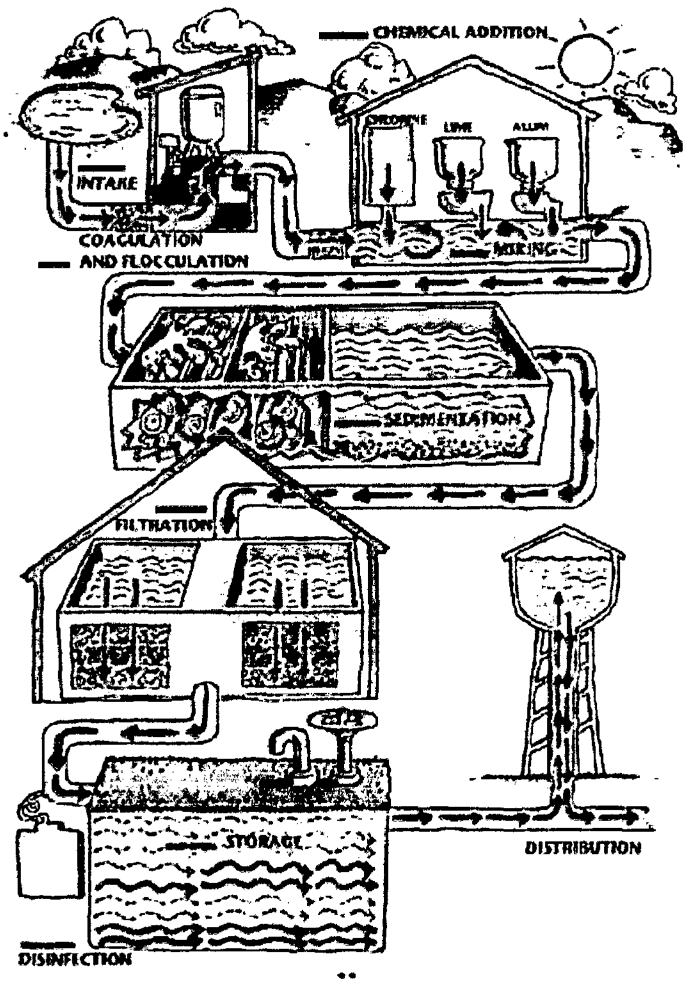


Fig.1.1: Typical Conventional Water Treatment System

1.2 DESIGN OF WATER TREATMENT PLANT

Various factors are taken into consideration for the design of water treatment plant. Analysis of raw water quality and drinking water quality standard is important for selecting the type of treatment to be provided and up to what extend of purity is required to be made. The other important design criteria are plant capacity, plant economics, equipment selection, and energy and resource requirement, etc. Calculation of plant capacity requires fixing of design period, sound judgment on population growth and the assessment of water demand. The plant capacity may be split into smaller sizes if staging of design period is made. While planning and designing, one has to observe the cost-effective approach to ensure that the construction cost as well as operation and maintenance costs are reasonable and appropriate for the planned level of treatment; knowledge on the availability of appropriate equipment and required skill of operation, and the availability of skilled staff; and power to run the machines. During early planning and design stages, a designer also has to be careful about the site selection of the proposed water treatment plant. Conditions such as natural and man made disasters, topography, available land area and land acquisition, access roads, available head, economy of conveying water to and from the plant, and disposal of all residuals should be all considered.

1.3 SCOPE OF THESIS

The objective of this thesis is, to study on various informations on the theory and design of water treatment plant by giving emphasize on hilly region, and the design of water treatment plant for Almora city augmentation of water supply scheme, the process of which includes the following:

- *Study of existing water supply scheme of Almora and importance of the proposed augmentation project,*
- *Generation of various design parameters from available data,*
- *Study of design parameters and selection of appropriate treatment processes,*
- *Selection of alternative options of selected treatment units and the detailed design.*

1.4. ORGANISATION OF THESIS

The present study is organized in following chapters: **Introduction, Study of Almora Water Supply Scheme and Its Importance, Design Considerations of Treatment Plant, Design of Water Treatment Plant, and Summary and Conclusions.**

Introduction (Chapter 1) emphasizes the importance and basic objectives of water treatment plant design, methods of water treatment required for different sources in general, design aspects of water treatment plant and scope of the thesis.

Study of Almora Water Supply Scheme (Chapter 2) briefly describes about the existing and proposed augmentation water supply scheme of Almora city, importance of the proposed scheme, and the challenges in the design of water treatment plant.

Basic Design Considerations of Treatment Plant (Chapter 3) presents the various design factors such as drinking water standards, raw water quality, plant capacity, selection of water treatment processes, etc., followed by generation of actual design factors for Almora water treatment plant from the data obtained. In this chapter, the design capacity of treatment plant for Almora water supply augmentation project is calculated as 19 mld for the design year of 2041, and the treatment train is selected which includes conventional treatment plant along with pre-sedimentation tank and aerator.

Design of Water Treatment Plant (Chapter 4) highlights the theories and design parameters of various units of water treatment system including intake facilities, water conveyance, pumping stations and treatment unit operations/processes such as aeration, coagulation-flocculation, sedimentation, filtration and disinfection. The design of various units of treatment system selected for Almora augmentation water supply scheme is also carried out in detail along with drawing of designed figures.

Summary and Conclusions (Chapter5) gives summary of important points and conclusions, and also suggests suggestions for further research.

CHAPTER 2

STUDY OF ALMORA WATER SUPPLY PROJECT

2.1 INTRODUCTION

Almora town is located at around 28 ^{0}36 'N latitude and 78 ^{0} 40 'E longitude in the *'hilly'* Kumaon region of Uttarakhand State of India. Being the administrative headquarter of Almora District , Almora town is one of the biggest and important towns of the state especially among the hill districts. It has got a steady growth in population over the past many years and has reached to 43960 in the 2001 census. It is one of the attractive hill resorts in the region with a lot of potentials in tourism industry. Almora is well connected by road from the neighboring areas, and it is at a distance of 103 km from Nainital and 378 km from Delhi. Geographical location map of Almora is shown in Figure 2.1. Topographically, Almora lies on a hilly terrain of about 5 km long horseshoe shaped ridge with an average elevation of 1600 m above sea level.

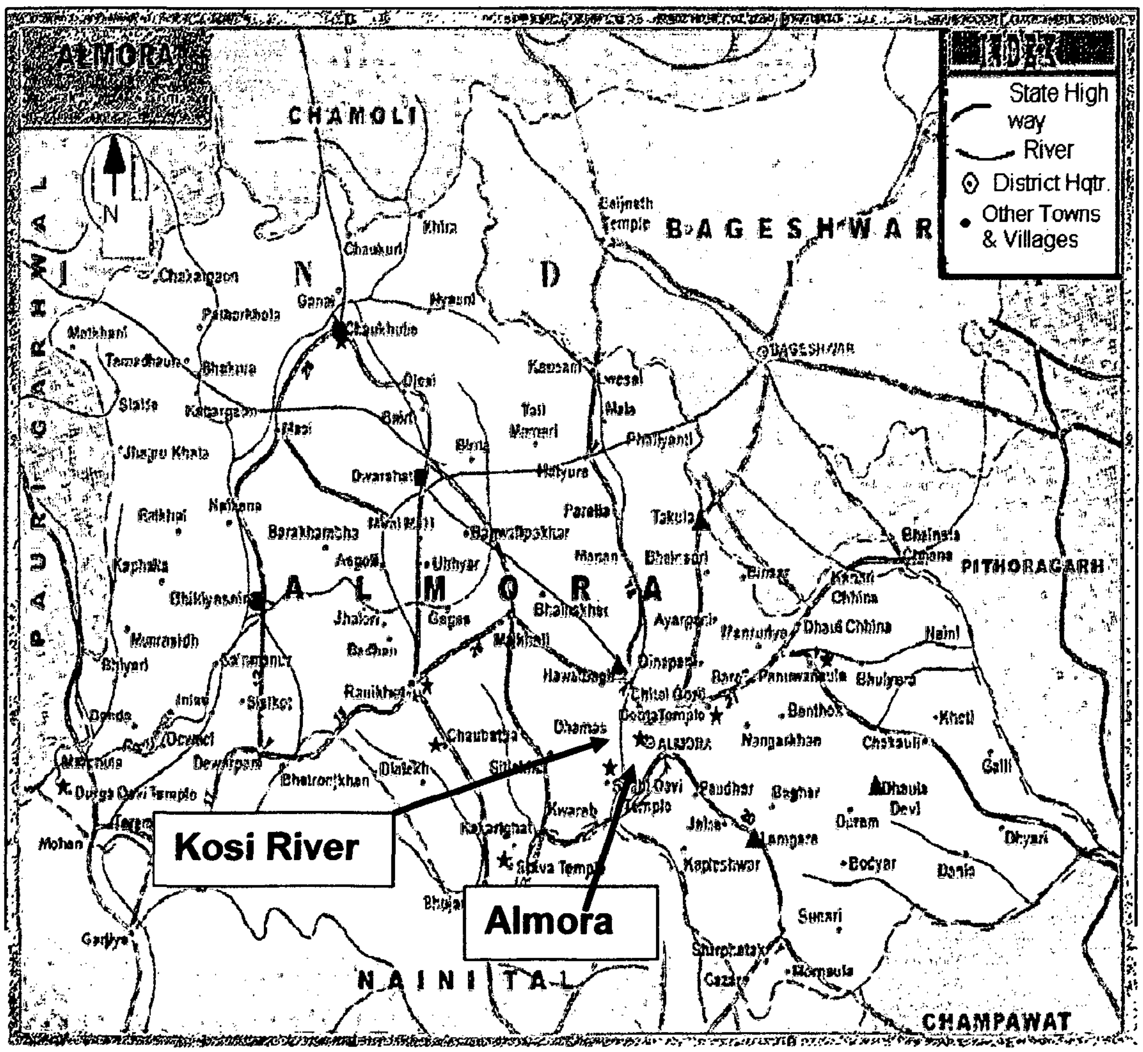


Fig. 2.1: Location Map of Almora Town

2.2 EXISTING WATER SUPPLY SCHEME

At present Almora town and the adjoining villages are getting drinking water through pumping scheme from Kosi river. The raw water is pumped through pumping stations located on the left bank of river Kosi near village Muleta, about 300m d/s of Kosi bridge to Matela centralized water treatment plant. From Matela, water is supplied by pumping to various service reservoirs. The existing water supply system can supply water at the rate of 4.5 mld during lean period.

2.3 PROPOSED PROJECT OF ALMORA WATER SUPPLY SCHEME

To supplement the water supply to Almora city, source sustainability is the main issue. Kosi river is the only reliable river which is within the economical zone for using as a source for Almora. However, the discharge is not sufficient during lean period. In search of sustainable sources, Uttarakhand Pey Jal Nigam suggests different options such as construction of barrage on Kosi river d/s of existing intake well, multipurpose dam at Bershimi on Kosi river, pumping from Saryu and Pindar river. Among this suggestions, barrage on Kosi river is recommended, which will be constructed to a height of about 10 m above the existing river bed level (1129) so that there is small pondage up to RL of 1140 and will create a storage of more than 4 MCM which is sufficient to meet Almora's water requirement by the year 2041 [Kansal, 2007].

Single well river intake type or Jack well is proposed to be located on the left bank of Kosi river just upstream of barrage (See Fig.3.4 & 4.6). Water will be drawn into the intake through off-take channel. Low lift pump is proposed to be installed on top of intake well, and raw water will be transmitted to pre-sedimentation tank proposed to be located downstream of barrage from where water shall be lifted to centralized water treatment plant at Matela through a static head of about 58 m and pumping main of about 13 km long. The treated water will be distributed to various sites through the existing distribution system which will need a major overhaul and expansion. Sketch Map of Almora water supply scheme is shown in Figure 2.2.

2.4 IMPORTANCE AND NECESSITY OF PROPOSED PROJECT

At present, the existing water supply scheme of Almora supplies only 4.5 mld which is at the rate of 90 lpcd (The present [2008] population is estimated to be 50000). As per norms of Government of India, the minimum per capita water supply level for a city

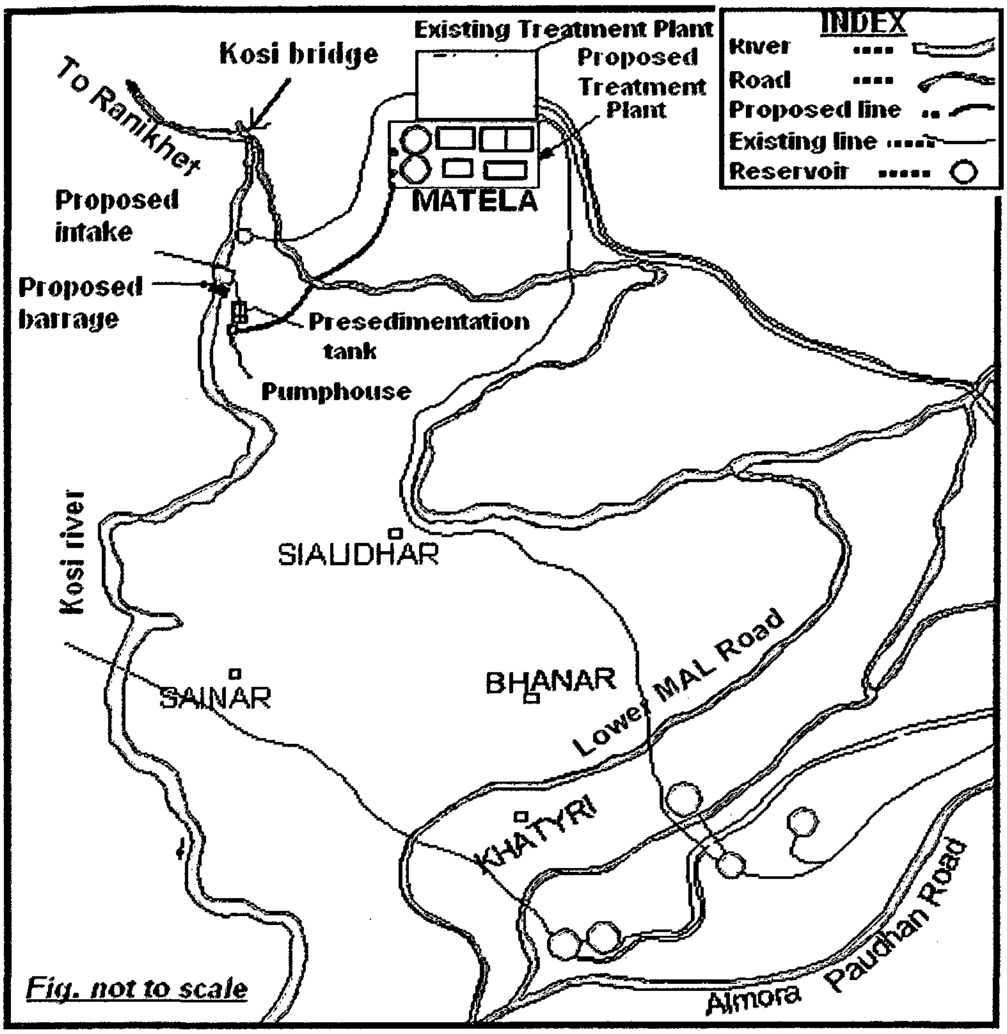


Fig. 2.2 : Sketch Map Of Almora Water Supply Scheme

with piped water supply is 135 lpcd. So, the present water supply has already become inadequate. And if the present growth rate continues, the water supply for Almora and adjoining villages will become highly deficit in the near future. Hence, new project for augmentation of water supply in the region is very important and necessary.

2.5 CHALLENGES IN THE DESIGN OF ALMORA WATER TREATMENT PLANT

Most of the books, literatures available on water works engineering are made naturally fit to suit the condition of plain areas or regular topography. Since the data available from the hilly areas are always different, to slightly modify or tilt the orientation in the methods or procedure of the design may be required in so many cases. Therefore, design of surface water treatment plant for Almora augmentatation water supply scheme is very challenging, at the same time encouraging and needs a lot of practical working knowledge in the same field.

CHAPTER 3
DESIGN CONSIDERATIONS OF TREATMENT PLANT

3.1 INTRODUCTION

There are certain design factors that must be considered in the planning and design of the water treatment plants. The basic design factors are:

- Drinking water standards or regulatory objectives
- Raw water quality
- Design period
- Service area
- Plant capacity
- Selection of water treatment processes
- Plant siting, layout, and hydraulics
- Equipment selection
- Energy and resource requirement
- Plant economics

These issues are briefly discussed in the following sections.

3.2 DRINKING WATER STANDARDS

Water supplied to the public must be in general, colorless, odorless and tasteless. It should be free from turbidity, and excessive or toxic chemical compounds, and harmful micro-organisms and radio activity must be absent. The permissible characteristics of water for drinking purpose are given in 'Drinking water quality standards'. The design of a water treatment plant depends upon the drinking standards of the area. In India, drinking water quality is governed by Indian standard drinking water specifications. The water quality standards as prescribed by *USEPA, WHO, and Bureau of Indian Standards* are shown in Appendix -I.

3.3 RAW WATER QUALITY

Type of water treatment depends on the quality of raw water and on the desired quality of the finished water. If the raw water quality is good, it is possible to treat water with minimum number of treatment processes and at the least treatment cost. Otherwise, the treatment required can be very complicated one and specific. For example, for treatment of saline coastal water, demineralization processes like distillation, reverse osmosis plants, etc. are popular. For general turbid surface water, conventional treatment process has been the most common choice. However, if the raw water contains minerals more than the permissible limit, certain chemical compounds are always required to feed into raw water to neutralize or reduce the level of the undesired mineral contents.

3.3.1 Raw Water Quality of Kosi River

For Almora water treatment plant, the source is Kosi River. The analysis results of Kosi water and ground water derived from tube wells in the surrounding areas of Kosi River is shown in Tables 3.1 and 3.2. The study on the quality of groundwater is also carried out considering that ground waters may seep or flow out into Kosi River at any points. The raw water analysis results obtained are compared with Indian drinking water standards and comparison remarks are given in the tables.

Table 3.1: Results of Surface Water Analysis (Kosi River)

Location : Kosi River (Near Pump House of Pey Jal Sansthan Almora)
Season : Summer

Sl. No.	Parameters	IS : 10500		Results on Sampling Date			Remark
		*	**	18/5/2007	23/5/2007	12/6/2007	
1	Total coliform organism, MNP/100ml			**300**	**400**	-	**X**
2	pH	6.5-8.5	6.5-8.5	7.4	7.2	7.6	O.K
3	Dissolved oxygen, mg/l	$\geq 6^a$		6.03	6.5	**5.4**	X
4	Total hardness (as $CaCO_3$), mg/l	300	600	64	70	84	O.K
5	Calcium (as Ca), mg/l	75	200	25	28	34	O.K
6	BOD, mg/l	$\leq 3^a$		**16**	**8**	2.1	**X**
7	Alkalinity	200	600	92	102	108	O.K
8	Iron (as Fe), mg/l	0.3	1.0	**1.79**	**1.84**	**1.96**	**X**
9	Arsenic (as As), mg/l	0.01	0.01		-	-	
10	Boron Max, mg/l	1	5	BDL	BDL	-	O.K
11	Mercury (as Hg), mg/l	0.001	0.001	BDL	BDL	-	O.K
12	Lead (as Pb), mg/l	0.05	0.05	BDL	BDL	BDL	O.K
13	Copper (as Cu), mg/l	0.05	1.5	BDL	BDL	BDL	O.K
14	Zinc (as Zn), mg/l	5	15	BDL	BDL	BDL	O.K
15	Manganese (as Mn), mg/l	0.1	0.3	0.018	0.021	0.031	O.K
16	Chromium (as Cr), mg/l	0.05	0.05	**0.054**	0.038	0.028	**X**

** Desirable limit. ** Acceptable limits in absence of alternate source. X= Undesirable*

[a] As per CPCB National water quality monitoring in India. BDL= Below desirable limit

Table 3.2: Results of Underground Water Analysis (Almora)

Source : TubeWell
Season : Summer

S/N	Parameters	IS : 10500		Result for the Location		Remark
		*	**	Kosi Village	Kathpuria	
1	Color, Hazen units	5	25	< 5	< 5	O.K.
2	Odour	+	-	Unobjectionable	Unobjectionable	O.K
3	Taste	#	-	Agreeable	Agreeable	O.K
4	Turbidity, NTU	5max	10	< 5	< 5	O.K
5	pH	6.5-8.5	6.5-8.5	7.0	7.0	O.K.
6	Total hardness, mg/l	300	600	88	68	O.K
7	Iron, mg/l	0.3	1.0	0.22	0.105	O.K
8	Dissolved solids, mg/l	500	2000	200	200	O.K.
9	Calcium, mg/l	75	200	24	19	O.K
10	Magnesium, mg/l	30	100	6.11	4.3	O.K
11	Copper, mg/l	0.05	1.5	BDL	0.008	O.K
12	Manganese, mg/l	0.1	0.3	BDL	BDL	O.K.
13	Phenolic compounds, mg/l	0.001	0.002	BDL	BDL	O.K
14	Mercury, mg/l	0.001	0.001	BDL	BDL	O.K
15	Selenium, mg/l	0.01	0.01	BDL	BDL	O.K
16	Cadmium, mg/l	0.05	0.05	0.002	0.011	O.K.
17	Arsenic, mg/l	0.01	0.01	BDL	-	O.K
18	Cyanide, mg/l	0.05	0.05	BDL	BDL	O.K
19	Lead, mg/l	0.05	0.05	0.015	BDL	O.K
20	Zinc, mg/l	5.0	15	0.12	0.09	O.K.
21	Chromium, mg/l	0.05	0.05	0.045	0.03	O.K
22	Mineral oil, mg/l	0.01	0.03	BDL	BDL	O.K
23	Alkalinity, mg/l	200	600	104	60	O.K

+ *Unobjectionable* ; # *Agreeable*

Apart from the raw water analysis results carried out during lean period, the turbidity level of Kosi river during high flow season is reported as high as 1500 NTU. The desirable limit for drinking water as per IS: 10500 (1991) is 5 NTU.

3.3.2 Evaluation of Raw Water Quality Analysis

The results of analysis shows that most ground waters do not contain objectionable contaminants, whereas, in the case of Kosi River water, certain objectionable limits of contaminants were found as mentioned in the following.

(1) Total coliform organisms is present in 100 ml water sample in two consecutive samplings, or > 10 MPN/100 ml, both of which is undesirable.

(2) Iron concentration exceeds the acceptable limit of the BIS drinking water standard in all the analysis results.

(3) The turbidity level exceeds the desirable level for drinking purpose, is quite high during high flow season.

(4) BOD values not more than 3 mg/l shows very clean water. High BOD values above 5 mg/l indicate a nearby pollution source. High BOD and low DO values in one analysis result shows slight taste & odor problems.

3.4 DESIGN PERIOD

A water treatment plant is generally designed and constructed to serve the needs of a community for a number of years in the future. Initial year is the year when the construction is completed and the operation begins. The design period is the duration in years at the end of which the facility is expected to reach its full design capacity. Selecting the design year requires considerations of several factors like useful life of treatment units, convenience of future expansion, cost of present and future construction, and availability of funds.

The design periods for different components of treatment plant recommended by GOI Manual on water supply(1999) is as below:

Raw water and clear water conveying units	:	*30 yrs.*
Pumping		
- pump house	:	*30 yrs.*
- electric motors and pumps	:	*15 yrs.*
Pipe connections to several treatment units		
and other small appurtenances	:	*30 yrs.*
Water treatment units	:	*15 yrs.*
Clear water reservoirs at head works	:	*15 yrs.*

By putting about two and half years for completion of construction, 2011 is selected as initial year of the Almora water treatment plant, and considering 30 years design period , the design year is 2041 AD.

3.5 SERVICE AREA

Service area is the total land area that will be eventually served by the proposed water treatment plant. It may be based on geographical and political boundaries. Areas outside the city that might become incorporated into the city at future dates should be normally considered in the service area. For Almora water treatment plant, it has been decided that the service area includes the Almora town and 16 adjoining villages.

3.6 PLANT CAPACITY

The design capacity of a water treatment plant is generally based on the maximum day demand that is expected to occur at the design year. This involves forecasting of the demand in the future. The most common approach for forecasting the plant capacity is based on population projections of the service area for the design year and projection of the rate of water demand. Average water demand is obtained multiplying the design population by per capita water demand. Maximum daily demand is generally taken as 180% of average daily demand. However, considering the requirement of additional reserve for breakdowns and repairs of treatment units, the maximum daily demand may be taken as twice the average daily demand [Garg, S.K, 2005].

3.6.1 Population Forecasting

Prediction of future population requires a sound judgment on the growth pattern of the city and sound prediction of developments which may likely occur in the service area that can be responsible for any unusual growth rate. There are many methods for calculation of future population. The particular method to be adopted for a particular city depends largely on various factors. However, none of the methods is exact, and they are all based on the laws of probability, and thus, only approximate estimates for the possible future populations may be made. The following are the widely employed methods:

- Arithmetic increase method
- Geometric increase method
- Incremental increase method
- Decreasing rate of growth method
- Comparative graphical method
- Master plan or zoning method
- Logistic curve method

Population Forecasting for Almora Water Supply Scheme:

Table 3.3: Historical record of population:

Year	1921	1931	1941	1951	1961	2001
Population	8359	9688	10995	12757	16602	43960
* includes rural population of 10630.						

Since there is no population data in between 1961 and 2001, to avail population record in 2001 which is the most recent data available (negligence of most recent data will not give correct projection), we may consider the population data in four decadal so that the last known data can be considered.

Table 3.4: Analysis of population data

Year	Population	Increase in population	% increase	Incremental increase
1921	8359			
1961	16602	8243	98.61	
2001	43960	27258	164.78	19115
Average		17800	131.69	19115

(i) **By Arithmetic Increase Method :**

We have Pn = Po + nx where,

Pn = forecasted population
Po = last known census
n = no. of decade
x = avg. increase in population.

Year	*Probable population*
2011	43960 + 0.25 x 17800 = 48410
2026	43960 + 0.625 x 7800 = 55085
2041	43960 + 1 x 17800 = 61760

(ii) **By Goemetric Increase Method :**

$Pn = Po (1 + r/100)^n$ Where, r = growth rate per decade in %

r (arithmetic) = 131.69 %

r (geometric) $=(r1.r2.r3...)^{1/t}$ =127.5 %

Take highest value of r ,i.e, 131.69 for safe side.

Year	*Probable population*	
2011	$43960 (1 + 131.69/100)^{0.25}$ =	54235
2026	$43960 (1 + 131.69/100)^{0.625}$ =	74323
2041	$43960 (1 + 131.69/100)^{1}$ =	101850

(iii) **By Incremental Increase Method :**

Pn = Po + nx + (n (n + 1)/2) y where, x = avg. increase per decade

y = avg. of incremental increase

Year	*Probable Population*	
2011	43960 + 0.25 x 17800 + 0.156 x 19115 =	51396
2026	43960 + 0.625 x 17800 + 0.508 x 19115 =	64792
2041	43960 + 1.0 x 17800 + 1.0 x 19115 =	80875

(iv) **Comparative Graphical Method**

In this method, graph is plotted using historical data and forecasted population above and developed a trend line which the growth of the town is most likely to follow considering various factors. Almora is a growing city but it has limitation to grow so fast and to become a very big city due to its hilly and remote location and being merely capital of one of the hill districts. Hence, let us assume the polynomial pattern of population growth of incremental increase curve. So, from the curve, using the equation, $y = -1593x^3 + 19115x^2 - 37952x + 28789$, where x = 3.25, 3.625, 4 for the year 2011, 2026 and 2041 respectively. The population is projected as below:

Year	*Probable Population*
2011	52663
2026	66514
2041	80869

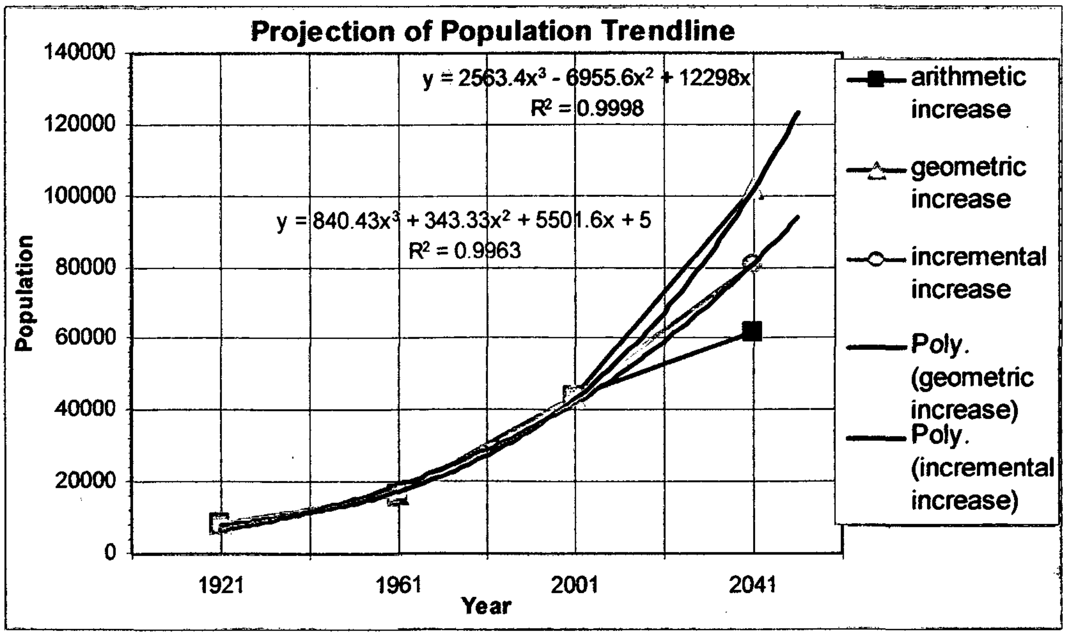


Fig.3.1: Population Forecast of Almora Town by Graphical Method

From the results obtained by the various methods, the following is adopted for design purpose of Almora water supply scheme :

Year	*Projected population*
2011	53000
2026	67000
2041	81000

Justification for Final Projected Population

Even though Almora is a growing town and has great potential to grow at a much faster rate in future, it may be difficult for Almora to become big city, because of its remote location and terrain problem. Hence, the population prediction obtained from polynomial trend line of incremental increase which is also slightly more than incremental increase projection seems to be most appropriate. For safe side, they are rounded off to higher side.

3.6.2 Per Capita Water Demand

Per capita demand of water can vary from place to place. It depends on various factors such as types of gentry and habits of people, industrial and commercial activities, development of sewerage facilities, size of the city, climatic conditions, quality of water supply, system of supply, cost of water, policy of metering and method of charging, and pressure in the distribution system. Normally, this figure ranges between 100 to 360 liters/capita/day for Indian conditions. Indian standard code of basic requirements of water supply, drainage and sanitation (IS :1172-1983) recommended per capita water supply levels for designing schemes which includes domestic and non-domestic needs as given in the table below:

Table 3.5: Recommended Per Capita Water Supply Levels

Sl/ No.	Classification of towns/cities	Recommended max. Water supply level (lpcd)
1	Towns provided with piped water supply but without sewerage system	70
2	Cities provided with piped water supply where sewerage system is existing/contemplated	135
3	Metropolitan and mega cities provided with piped water supply where sewerage system is existing/contemplated.	150

Notes for table 3.5:

(1) In urban areas, where water is provided through public stand posts, 40 lpcd should be considered.

(2 Figures exclude 'unaccounted for water (UFW)' which should be limited to 15%.

(3) Figures include requirements of water for commercial, institutional and minor industries. However, the bulk supply to such establishments should be assessed separately with proper justification.

Per capita water demand for Almora town may be assessed as below:

Demand	*Almora (lpcd)*	*Adjoining villages (lpcd)*
Domestic and non domestic use	135	70
Unaccounted for water (@15%)	24	13
Total	159	83

Considering about 20 % population in the design year is from the villages, then, per capita water demand becomes 0.8 x 159 + 0.2 x 83 = 143.8 lpcd, say **145 lpcd.**

Table 3.6: Projected Water Demand for Almora Water Supply Scheme

Year	Population	Per Capita Consumption (lpcd)	Average day demand (mld)	Maximum day demand (mld)
2011 (initial year)	53000	145	7.7	15.4
2026 (intermediate)	67000	145	9.7	19.4
2041 (design year)	81000	145	11.75	23.5[a]

[a] 11.74 x 2 = 23.5

Actual Water Demand for Design of Treatment Plant of Almora

Actual demand of water for design purpose is obtained by deducting the existing water supply level (4.5 mld) as below.

Year	*Actual Water Demand (mld)*		
2011	15.4 – 4.5	=	10.9
2026	19.4 – 4.5	=	14.9
2041	23.5 – 4.5	=	19

3.7 WATER TREATMENT PROCESSES

The method of water treatment to be employed depends in general on the nature of raw water constituents and the desired standards of treated water quality. It may be so simple consisting only one unit operation or very complex with different combination of various unit operations.

A water treatment plant may utilize many treatment processes to produce water of a desired quality. These processes generally fall into two divisions: unit operations and unit processes. In unit operations, physical forces bring about the removal of contaminants. In the unit processes, however, the treatment is achieved by chemical and biological reactions. Water treatment plant may utilize a raw water intake, pumping, conveyance system, and flow measurement. Although, these systems do not provide any treatment, they are necessary as part of the overall treatment process train. Hence, they may be included among the units. A summary of different unit operations (UO) and unit processes (UP) generally considered in the design of a water treatment plant [Qasim et al,2000] is given in the following.

A. *Trash rack* (UO): Trash racks provided at the intake gate for removal of floating debris and ice.
B. *Coarse screen or fish screen* (UO): Mechanically cleaned screens provided at the intake gate or in the sump well ahead of pumps. Protect fish and remove small solids and frazil ice.
C. *Micro- strainer* (UO): Micro-strainer removes algae and plankton from raw water.
D. *Aeration* (UP): Strips and oxidizes taste and odour causing volatile organics and gases and oxidizes iron and manganese. Aeration systems include gravity aerator, spray aerator, diffuser, and mechanical aerator. Aeration in the reservoir helps in desertification and taste and odor control.
E. *Mixing* (UO): Mixing provides uniform and rapid distribution of chemicals and gases into the water.
F. *Pre-oxidation* (UP): Application of oxidizing agents such as ozone, potassium permanganate, and chlorine compounds in raw water and in other treatment units; retards microbiological growth and oxidizes taste, odour and colour causing compounds.

G. *Coagulation* (UP): Coagulation is the addition and rapid mixing of coagulant resulting in destabilization of the colloidal particle and formation of pin-head floc.

H. *Flocculation* (UO): Flocculation is aggregation of destabilized turbidity and colour causing particles to form a rapid-settling floc.

I. *Sedimentation* (UO): It is gravity separation of suspended solids or floc produced in treatment processes. It is used after coagulation and flocculation and chemical precipitation.

J. *Filtration* (UO): Filtration removes of particulate mater by percolation through granular media. Filtration media may be single (sand, anthracite, etc.), double, mixed, or multi-layered.

K. *Chemical precipitation* (UP): Addition of chemicals in water transforms specific dissolved solids into insoluble form. Removal of hardness, iron and manganese, and many heavy metals is achieved by chemical precipitation.

L. *Lime-soda ash* (UP): Lime-soda ash is a chemical precipitation process used for water softening. Excess amounts of calcium and magnesium ions are precipitated from water.

M. *Recarbonation* (UP): Restores the chemical balance of water after softening by lime-soda process. Bubbling of carbon dioxide converts supersaturated forms of Ca and Mg into more soluble forms. The pH is lowered.

N. *Activated carbon*: It removes taste and odour causing compounds, chlorinated compounds, and many metals. It is adsorption (UP) used as powdered activated carbon (PAC) at the intake or as a granular activated carbon (GAC) bed after filtration.

O. *Activated alumina* (UP): It removes certain species from water by hydrolytic adsorption. Fluoride, phosphate, arsenic and selenium have been effectively removed by activated alumina.

P. *Disinfection* (UP): Disinfection destroys disease-causing organisms in water supply. Disinfection is achieved by ultraviolet radiation and by oxidative chemicals such as chlorine, bromine, iodine, potassium permanganate, and ozone, chlorine being the most commonly used chemical.

Q. *Ammoniation* (UP): Ammonia converts free chlorine residual to chloramines. In this form, chlorine is less reactive, lasts longer, and has less tendency to combine with organic compounds, thus reducing taste and orders and THM formation.

R. *Fluoridation* (UP): Sodium fluoride, sodium sillicofluoride, and hydrofluosilicic acid may be added in finished water to produce water that has optimum fluoride level for control of tooth decay.

S. *Biological denitrification* (UP): Nitrate in excessive concentration in drinking water may cause methemoglobinemia in infants. Nitrate is reduced to gaseous nitrogen by microorganisms in anaerobic environment. An organic source such as ethanol or sugar is needed to act as a hydrogen donor (oxygen accepter) and to supply carbon for synthesis.

T. *Demineralization* (UP): Involves removal of dissolved salts from the water supply. Demineralization may be achieved by ion exchange, membrane processes, distillation, and freezing.

T1 *Ion Exchange* (UP): The cations and anions in water are selectively removed when water is percolated through bed containing cation and anion exchange resins. The beds are regenerated when the exchange capacity of the bed is exhausted. Selective resins are available for hardness, nitrate, and ammonia removal.

T2 *Reverse osmosis* (RO) and ultra filtration (UO):Semi permeable membranes are used to permeate high quality water while rejecting the passage of dissolved solids. Reverse osmosis is also used for nitrate and arsenic removal.

T3 *Electro dialysis* (UO): An electrical potential is used to remove the cations and anions through the ion-selective membranes to produce demineralised water and brine.

T4 *Distillation* (UO): Multiple-effect evaporation and condensation and distillation with vapour compression are common methods used for large-scale demineralisation systems.

T5 *Freeze* (UO): Saline water is cooled to reach the freezing point. The ice contains pure water, while the dissolved salts remain in solution. The ice is removed and melted to recover fresh water. This method is used in colder climates.

The collective arrangement of various unit operations and processes is called a Process Train or Treatment Train. Choice of proper treatment processes and development of a process train is not a simple task. It requires understanding of the processes, performance, and operational capabilities. Table 3.8 provides a general guideline for selection of treatment processes for desired applications [Qasim,S.R. et al,2000].The following considerations, however, influence the selection of treatment process train.

(a) Ability to meet finished water quality objectives, considering both seasonal and long-term changes in raw water quality;

(b) Topography and site conditions, existing treatment facilities, land area availability, and hydraulic requirements;

(c) Overall system reliability;

(d) Flexibility and simplicity of operation;

(e) Capability of process to upgrade performance in case raw water quality or drinking water regulations change;

(f) Capability of process to meet the hydraulic peaks;

(g) Availability of skilled operation and maintenance personnel, major equipment, and chemical-delivery system;

(h) State and federal requirements;

(i) Preference and experience of the design engineer;

(j) Ease of construction of facilities;

(k) Economy of construction and operation.

Table 3.7 Treatment Desired and Process Selection

Contaminant Removal	Process	Remarks
Turbidity	a. In-line filtration (G + J)	Application to water having low turbidity and color
	b. Direct filtration(G+H+J)	Applicable to water having low to medium turbidity and low to medium color
	c. Conventional(G+H+I+J)	Applicable to low and high turbidity and color
Algae and Plankton	a. Micro strainer (C)	Micro strainers will not handle silt, sand, and other abrasive material
	b. Conventional (G+H+I+J)	High population of algae and plankton are difficult to coagulate. They usually float.
Color	a. Oxidation (ozone, chlorine, chlorine dioxide, Potassium permanganate) (F)	Applicable to water having low color.
	b. Low pH coagulation (G + H)	Applicable to water having from low color to high. (Alum performs better than iron salts) Opt. pH 5-6
	c. Adsorption (N)	Applicable to water having low to moderate color episodes.
	d. Adsorption (N/GAC)	Applicable to water having moderate to low level soluble color for routine color control.
	e. Ion- exchange (J + T1)	Synthetic resin bed after filtration removes soluble

		color of industrial origin.
Iron and Manganese	a. Oxidation (D + F + I)	Iron and manganese are removed by oxidation and precipitation.
	b. Precipitation (D + K)	Fe & Mn are precipitated by aeration at high plumbing pH. Lime is generally used to raise pH
	c. Conventional (G+H+I+J)	Iron and manganese are removed by conventional coagulation-flocculation.
	d. Ion exchange (T1)	Selective ion exchange resins are generally used for removal of Fe and Mn from ground waters
Taste & Odor (T&O)	a. Oxidation (D or F)	Aeration in the reservoir, at the head of plant, and in treatment units may reduce T&O. Oxidation with free chlorine may cause THM formation
	b. Adsorption (N/PAC or GAC)	PAC is used for moderate and intermittent T&O episodes. GAC is used for prolonged T&O due to industrial sources
Hardness	a. Precipitation (L)	Used for medium to hard water.
	b. Ion exchange (T1)	Zeolite beds remove divalent cations but add Na+. Cation exchangers remove all cations.
Pathogens	a. Disinfection (P)	Free chlorine enhances THM formation potential
THM	a. Enhanced coagulation (G+ K + H + I)	Efficient removal of precursors (organics) by low pH coagulation or in conjunction with softening.
	b. Adsorption (N)	PAC and GAC will remove TOC, precursors, and THM.
	c. Aeration (D)	Aeration removes organics.
	d. Preoxidation (F)	Preoxidation of precursors by O_3, H_2O_2, $KMNO_4$, chloramines or chlorine dioxide.
Nitrate	a. Denitrification (S)	Biological denitrification provides an effective method of NO_3 removal.
	b. Ion-exchange (T1)	Selective resins are available.
	c. Demineralization (T2)	Demineralization with membranes removes NO_3 with other ions. Deionization with RO fluoride. High concentrations of fluoride in drinking water cause fluorosis and toxicity.
Fluoride	a. Activated alumina (O)	Activated alumina selectively removes F.
Arsenic	a. Enhanced (G + H + I)	Removes by precipitation of arsenate As (V), at low pH with coagulation and at a high pH with partial softening with lime.
	b. Activated alumina (O)	Bed is effective in removing arsenic from ground water.
	c. Demineralization (T2)	Arsenic is removed with other ions by membrane Processes.

GOI Manual on Water Supply and Treatment also provided recommendation on treatment process train for different types of sources as shown in Figure 3.2. The various types of sources in figure 3.2 are described below.

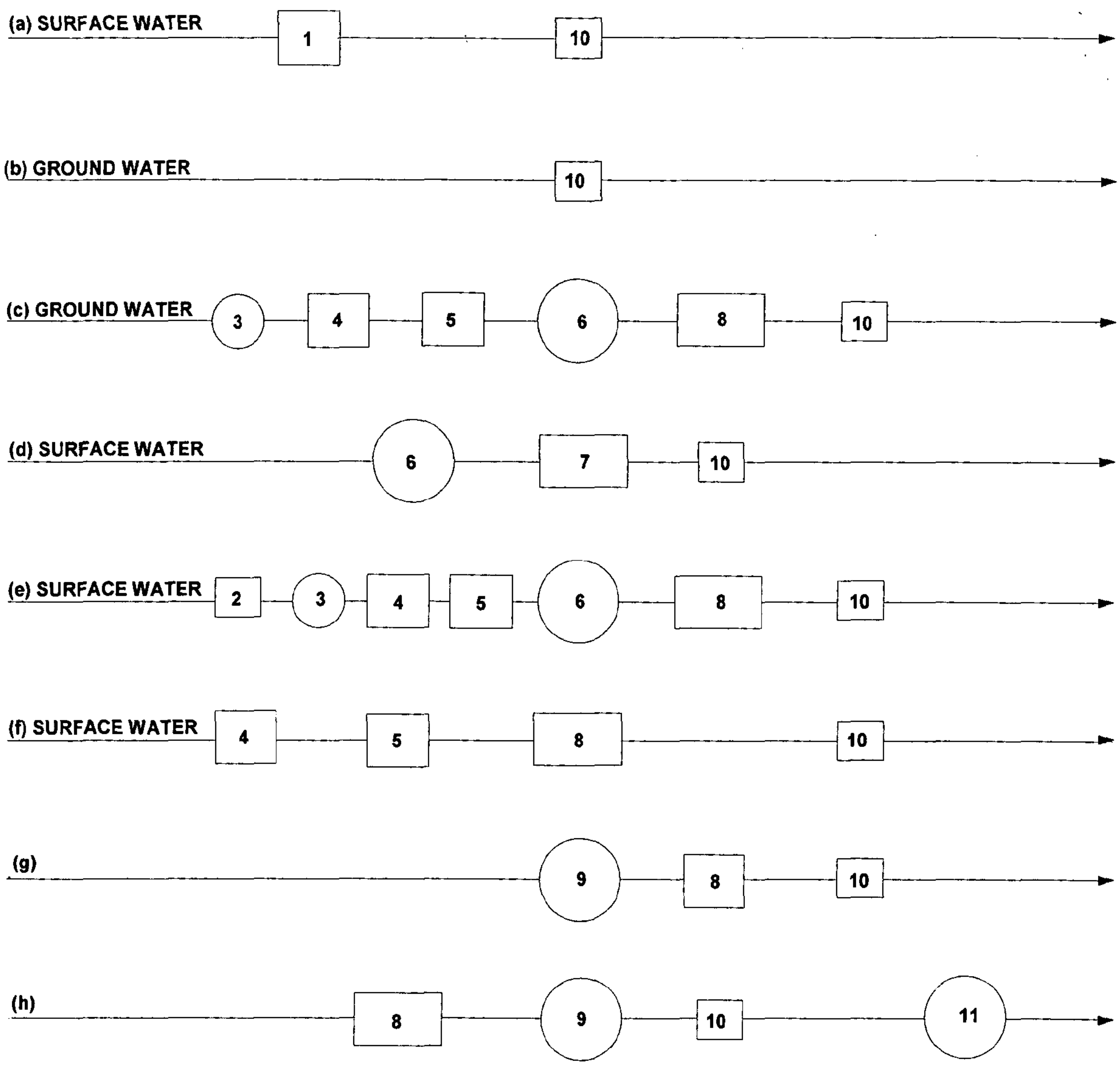
Source	***Description***
(a)	Surface water with storage which are well protected, turbidity below 10 NTU, and free from odor and color.
(b)	Ground water which are well protected, free from odor and color.
(c)	Ground water which contains excessive iron, dissolved carbon dioxide, and odor gases.
(d)	Surface waters with turbidities not exceeding 50 NTU and where sufficient area is available.
(e)	Highly polluted surface waters laden with algae or other micro-organisms.
(f)	Surface waters of low turbidity (below 10 to 15 NTU) and containing low concentration of suspended matter (less than 50 mg/l)
(g)	Water with excessive hardness.
(h)	Water with high concentration of dissolved solids.

3.7.1 Selection of Treatment Process Train for Almora Treatment Plant

The results of raw water analysis make it necessary that the water treatment processes be designed to accomplish the following:

1. high turbidity removal
2. disinfection
3. taste & odor control , and
4. Iron and chromium removal.

Pre-sedimentation tank shall be used to settle the coarser and larger sediments which can be easily settled by gravity. This is especially beneficial if done prior to raw water pumping so that, the pumped water will be slightly less turbid to prevent erosion of internal wall of pipe and possible damage of pumping main in the long run. Taste & odor shall be removed or controlled using aeration. Removal or reduction of iron contents can be done through aeration followed by coagulation and sedimentation. Chromium may be removed by Coagulation/Filtration [USEPA] or toxic chromium $\left(C_r O_4^{2-}\right)$ by adsorption on alum [Komori et al, 1990]. The heart of the



1.	**STORAGE**	**6.**	**SEDIMENTATION**
2.	**CHLORINATION (PRE)**	**7.**	**SLOW SAND FILTRATION**
3.	**AERATION**	**8.**	**RAPID SAND FILTRATION**
4.	**RAPID MIXING**	**9.**	**SOFTENING**
5.	**FLOCCULATION**	**10.**	**CHLORINATION (POST)**
		11.	**DEMINERALIZATION**

FIG. 3.2 : VARIOUS WATER TREATMENT PROCESS TRAIN

treatment process will be turbidity removal which is required to make the water fit from aesthetic point of view as well as for effective disinfection. This will be taken cared by coagulation-sedimentation and filtration processes. Hence, the treatment plant will consists of state-of the-art conventional treatment plant preceded by plain sedimentation and aeration as shown in Fig. 3.3.

Generally, anionic polyacrylamides with very high molecular weights are the most effective types of coagulant aids, and is proposed as coagulant aid for Almora water treatment plant to improve and accelerate the process of coagulation and flocculation and it shall be fed in the Parshall flume. **Pre-chlorination** shall be done and chlorine shall be applied at raw water transmission line near intake to control biological growths in raw water conduits and pre-sedimentation tanks, improve coagulation, prevent formation of mud ball and slime formation in filters, reduce taste, odor and color, iron and manganese, and to minimize the post chlorination dosage. Provision for feeding **filter aid** (high molecular weight nonionic polymer) should be made in between sedimentation tank and filter to enhance filtration during periods of poor settled-water quality. Several other chemical feed points and equipments may also be provided for operational flexibility under variable raw water conditions. Line diagram of proposed Almora water treatment system is shown in Fig.3.4.

1 Intake
2 Lowlift Pump
3 Pre-Sedimentation Tank
4 Storage Tank/Sump
5 1st Stage Pumphouse
6 Aerator

7 Rapid Mix Unit
8 Parshall Flume
9 Clariflocculator/Coagulation-Sedimentation Tank
10 Rapid Gravity Filter
11 ClearWell/Sump
12 Highlift Clear water Pumphouse

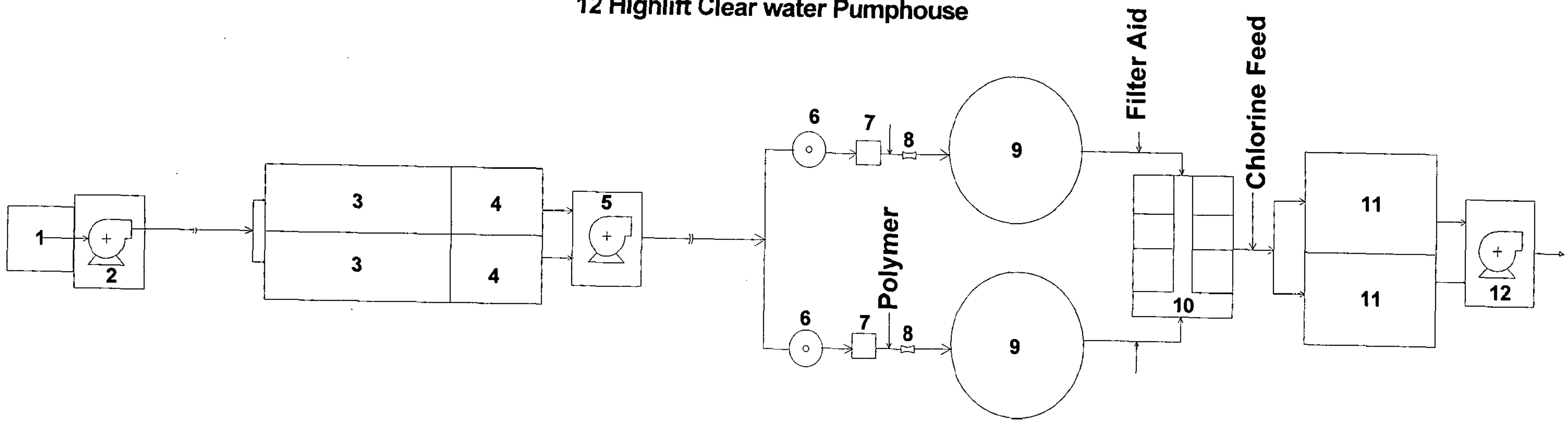


Fig.3.3: Proposed Flow Diagram For Almora Water Treatment Plant

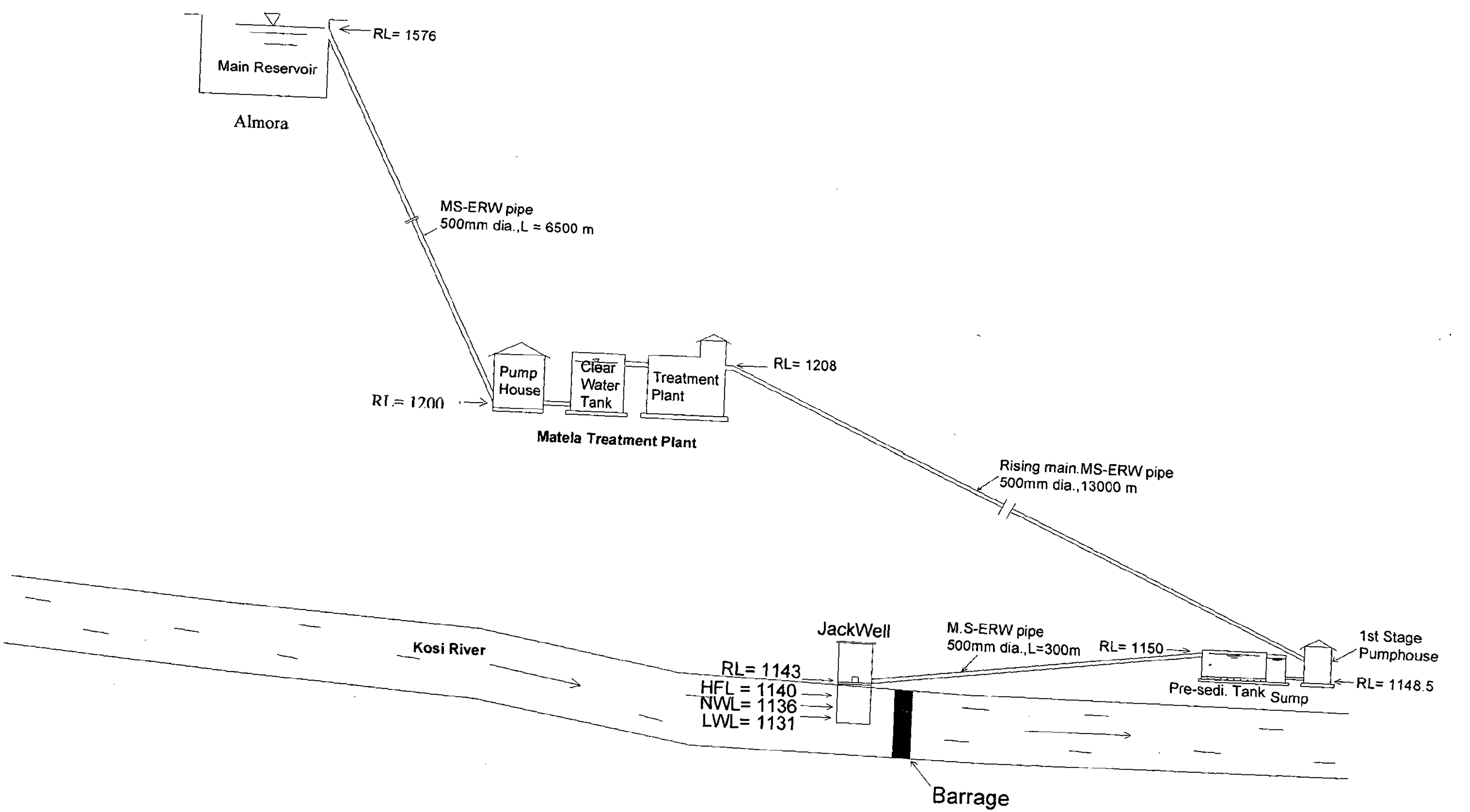


FIG. 3.4 PROPOSED LINE DIAGRAM OF ALMORA WATER SUPPLY SCHEME

CHAPTER 4

DESIGN OF WATER TREATMENT PLANT

4.1 INTRODUCTION

Treatment plant is one of the key components of a water supply scheme, which has to satisfy the drinking water needs of the population on continuous basis by converting the raw water into potable water. A conventional water treatment plant consists of rapid mixing, coagulation- sedimentation and rapid gravity filter units. However, water treatment plants may utilize a raw water intake, pumping, conveyance, and flow measurement although these systems do not provide any treatment, they are necessary as part of the overall treatment process train. Hence, purification system of community water supplies includes screens, intake, pumping system, conveyance and treatment plants.

4.2 INTAKE FACILITIES

The intake structure is a hydraulic structure, located in a surface water source to permit the withdrawal of water from the source. The primary purpose of intake is to selectively withdraw the best quality water while excluding fish, floating debris, coarse sediment, and other objectionable suspended matter. In case of a reservoir where gravity flow is possible, the water may be directly taken though the conduit up to the water treatment plant, whereas, incase of direct river supplies, the water after entering the intake well may have to be lifted by low lift pumps and then taken to the treatment plant through conduits. Screens are required in any case to protect the conduit from being clogged by trash, debris, etc., and to protect the pump equipment.

4.2.1 Types of Intakes

Selection of an intake structure depends on the nature of the water source and the quantity and quality of the water being withdrawn. Various types of intakes that are commonly in use are described in the following.

(a) Tower Intakes. Exposed intake gatehouse or tower structures are generally used for large projects on rivers or reservoirs with large water-level fluctuations. They may be built either sufficiently near the shore to be connected by a bridge walkway or at such distance that they can be reached only by boat. Gate-controlled openings (also called ports) are generally provided at several levels in order to permit selection of the best-quality water. There are two major types of exposed or tower intakes: *wet-intake towers,* and *dry-intake towers.*

Wet Intake Tower. Wet-intake towers consist of a concrete circular shell filled with water up to the reservoir level, with an inner well that is connected to the withdrawal conduit. Ports are provided into the outer concrete shell and into the inner well. The purpose of the water-filled outer shell is to provide weight needed to balance the buoyant forces when the inner well is dry. The withdrawal may be directly taken to the treatment plant in case no lift is required or to the sump well in case a low lift is required.

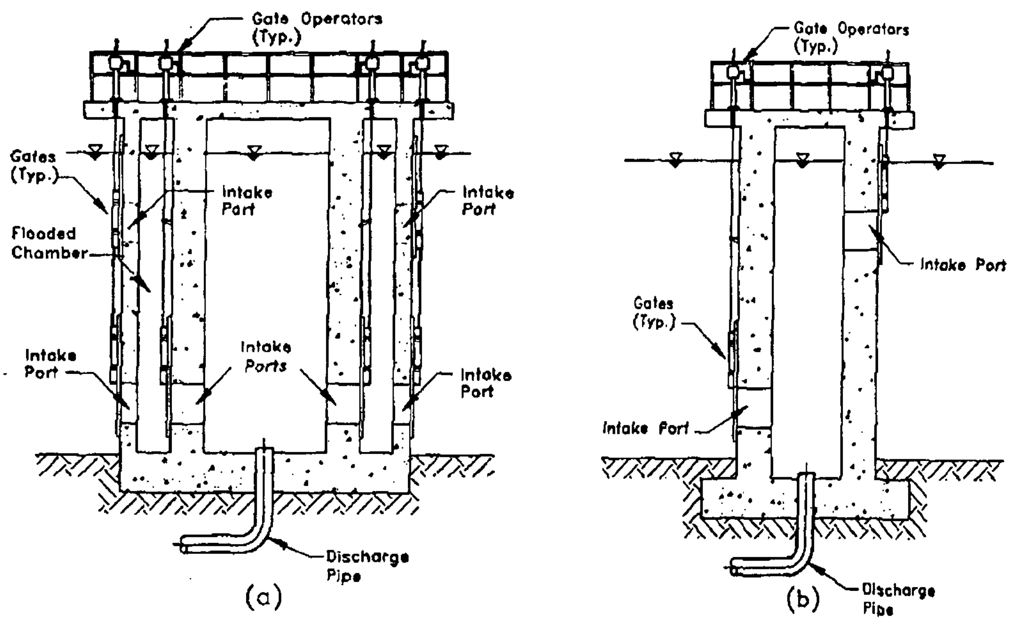


Fig. 4.1: Typical intake towers (a) Wet-intake tower (b) Dry-intake tower

Dry-Intake Tower. In a dry-intake tower, water enters the well through gate-controlled ports. In the absence of a water-filled outer shell, the dry-intake tower must be heavier in construction than a wet-intake tower, to resist the buoyant forces when empty; however, these intakes are beneficial, because water can be withdrawn from any selected level by using the proper port.

(b) Submerged Intakes. Submerged intakes are used to withdraw water from streams or lakes that have relatively little change in water-surface elevation throughout the year. These intakes are constructed as *cribs* or screened *bell mouth.* They may consist of a simple concrete box or of a rock-filled timber crib to support the influent end of the withdrawal pipe. The top opening is covered with cast iron or mesh grating. In lakes with heavy siltation, the intake opening is raised about 2 to 2.5 meters above the bottom. They are not used on bigger projects on rivers and reservoirs.

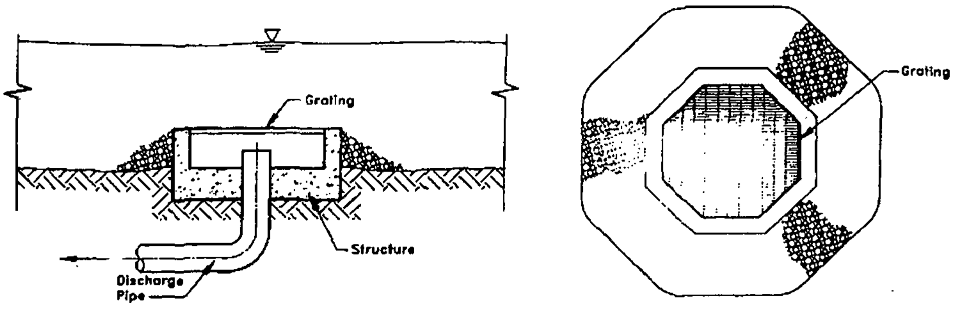


Fig. 4.2 : Typical Submerged Intake Structure

(c) Medium Sized River intake Structures. Medium sized river intakes are generally constructed for withdrawing water from almost all rivers, and are via. media between the submerged intakes (usually adopted for small streams) and the intake towers (usually adopted for reservoirs). River intake structures may be two types- Twin well type of intake structure, and Single well type of intake structure.

Twin Well Type of River Intake Structure. This type of intake is generally constructed on almost all types of rivers, where the river water hugs the river bank. Such a condition is usually available on non-alluvial rivers. Typical twin well type river intake structure consists of inlet well, inlet pipe, and a jack well. Water enters through the ports or openings into the inlet well which is connected to the jack well, which is constructed on the river bank, by an intake pipe. The intake pipe should be designed as gravity main since the water on both ends of this main will be under atmospheric pressure. Water entering the jack well is lifted by pumps and is fed into the rising main through delivery pipe of the pump.

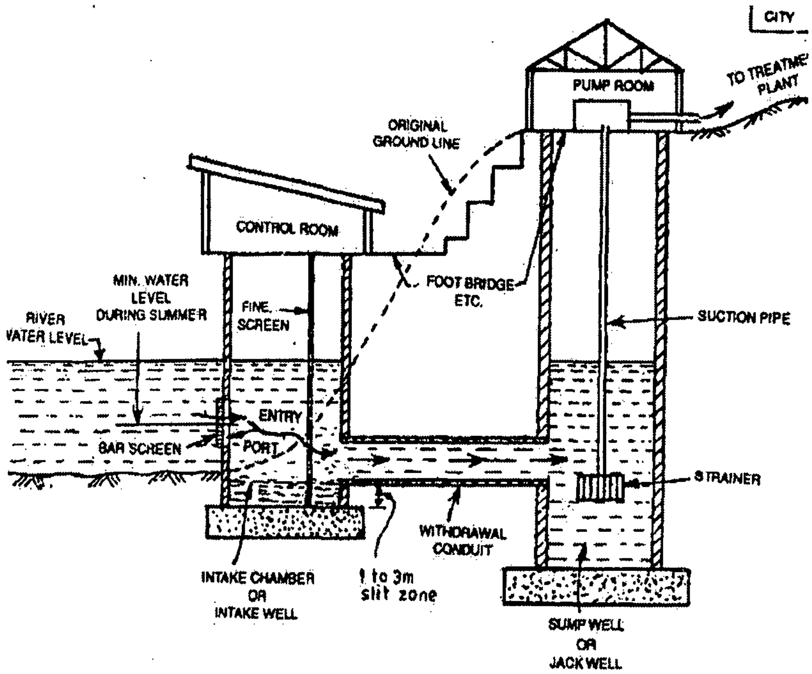


Fig.4.3 : Typical Twin Well Type of a River Intake

Single Well Type of a River Intake. In alluvial rivers, water is always ponded up by constructing a weir across the river. From the upstream side of such a weir, a channel may sometimes be taken off, as in a usual diversion head works scheme. The intake structure is located in this off-take channel. Sometimes, an approach channel may be excavated from the deep river towards the river bank up to the site of intake. In such cases, the intake structure may be constructed across the off-take or approach channel in the form of a single jack well (sump well) provided with direct entry ports, as shown in Fig. 4.4. Openings or ports fitted with Bar screens are provided in the jack well itself.

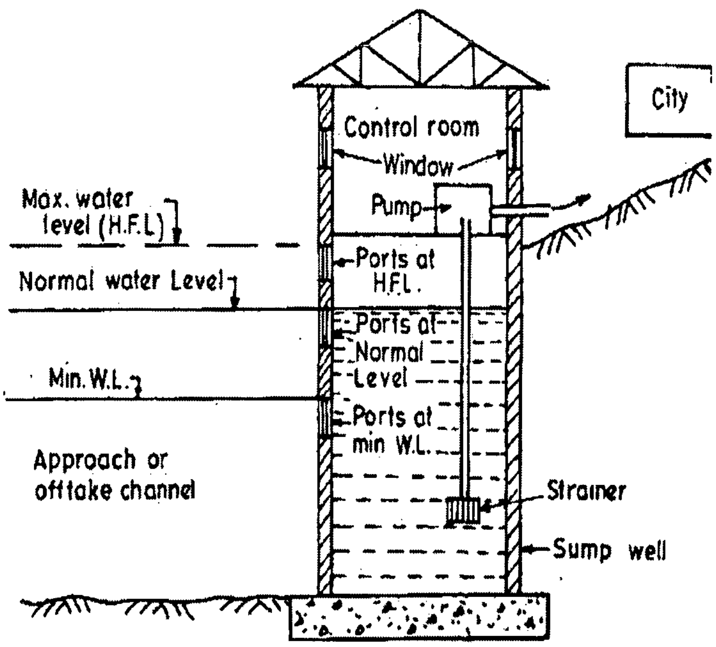


Fig. 4.4: Single Well Type of a River Intake

(d) Floating Intakes. Floating intake structures offer a relatively inexpensive method of constructing an intake in an existing lake or reservoir. Floating intake structures are particularly suitable in water sources that have unsuitable geological conditions and where relatively little difference in water-surface elevation over time occurs. They consist of a barge-type structure that floats on the water surface and supports pumps, screens, valves, electrical switches, gears, and other equipment.

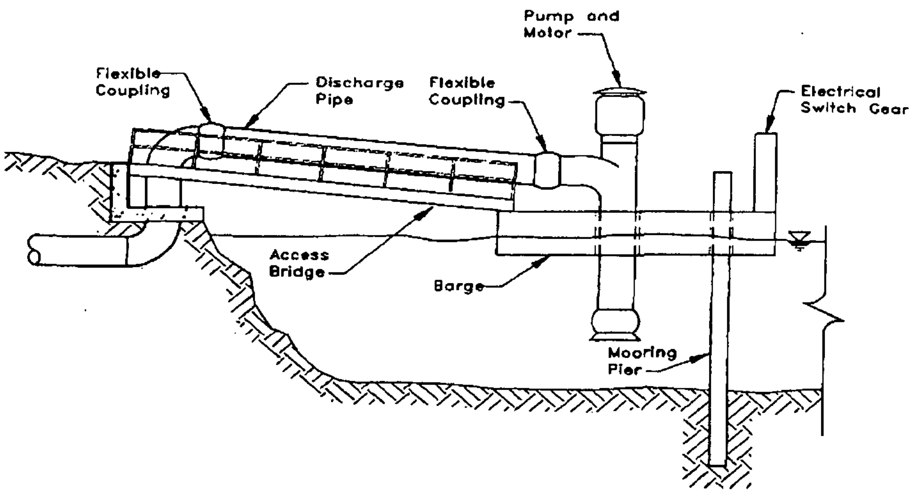


Fig. 4.5: Typical Floating Intake Structure

4.2.2 Location of Intake

The following factors should be considered for locating the intake:

(i) The location where the best quality of water is available
(ii) Absence of currents that will threaten the safety of the intake
(iii) Absence of ice floats, etc.
(iv) Formation of shoal and bars should be avoided.
(v) Navigation channels should be avoided as far as possible.
(vi) Floods
(vii) Availability of power and its reliability.
(viii) Accessibility.
(ix) Distance from pumping station.
(x) Possibilities of damage by moving objects and other hazards.

4.2.3 Intake Design Considerations

The design of intake structures is generally site-specific. Rarely can a standard design be adapted for a given site without major modifications. The intake structures design should provide for withdrawal of water from more than one level to cope up with seasonal variations of depth of water. The bottom *port* should be located at least 1 m (3 ft) above the bottom to prevent the suspension of bottom sediments. Under sluices should be provided for release of less desirable water held in storage.

The capacity of the conduit and the depth of the suction well should be such that the intake ports to the suction pipes of pumps will not draw air. A velocity of 0.6 to 0.9 m/s in the intake conduit gives satisfactory performance. The horizontal cross sectional area of the suction well should be 3 to 5 times vertical cross sectional area of the intake conduit.

Coarse screens consist of vertical flat bars, or, in some cases, round pipes spaced with 5 to 8 cm of clear opening. Coarse screens should be installed outside (on the water side) of any sluice gate or stop log slot. The velocity through the coarse screen is generally less than 0.08 m/s.

Fine screens may be located either at the intake structure or at the raw water pump station. In the case of gravity conveyance systems, the fine screen may be provided at the water treatment plant. These screens consist of heavy wire mesh with 0.5 cm square openings or of circular passive screens with similar opening widths. The screen area efficiency factor is 0.5 to 0.6, and the typical design velocity through the effective area is in the range of 0.4 to 0.8 m/s.

Micro strainers are used for the removal of plankton and algae from impounded waters. Raw water containing a heavy population of algae and plankton is difficult to coagulate. Removal of algae prior to prechlorination and coagulation has beneficial effects on taste and odor control and on reduction in THM formation and other chlorine by-products. Also, chlorine dosage for prechlorination is significantly reduced after micro straining.

Gates typically used in intake structures are sluice gates. These are large cast-iron gates that slide vertically on a guide track. The gate is moved by a threaded rod (stem) mat extends to the top of the structure.

4.2.4 Design of Intake For Almora Water Supply Scheme

Intake is to be located just upstream of the barrage on the left bank of Kosi river. Due to the river channel system and site condition, *Single Well type of a river intake or Jack well* is found to be most suitable and comparatively economical since there is no separate inlet well. The jack well will have direct water entry ports from the side facing river through off-take channel. The other side will be retaining the river embankment.

1.*Hydraulic Data* :

Design clear water demand	:	19 mld or 19000 m^3/day
*Raw water demand considering 7 % treatment loss.	:	20.377 mld or 20377 m3/day
R.L. of river bed	:	1129 m(msl)
R.L. of lowest water level	:	1131 m
R.L. of normal water level	:	1136 m
R.L. of HFL/pondage level	:	1140 m

*Inflow to RGF for required outflow of 19 mld considering 3% of filtered water for backwash = 19 x 1.03 = 19.57 mld . Inflow to Clariflocculator with 2% of inflow for desludging = 19.57/0.98 = 19.969 mld. Inflow to Pre-sedimentation tank with 2% of inflow for desludging = 19.969 / 0.98 = 20.377 mld .

2. *Location and Configuration of head works*:

The proposed raw water intake cum sump will be located on the right bank of Kosi river just upstream of barrage. It will be rectangular in shape with the face side having ports and will withdraw water from the off-take channel of length about three meters. Low lift pump will be housed on the top of the sump for pumping raw water to the Pre-sedimentation tank to settle the larger and heavier sediments of water.

3. *General design guidelines*:

Intake shall be provided with coarse screen and fine screen to prevent large and small debris entering the sump. Coarse screen shall be provided at the entrance of the off-take from the river. The velocity through rack should be less than 8 cm/s. Mechanically cleaned fine screen shall be located at the intake ports, slightly projected away from the intake gate. The velocity through the fine screen shall be less than 0.2 m/s.[Qasim et al, 2000].

Design:

1. **Size of intake ports :**

Q = 20377 m^3/d = 0.2358 m^3/s

Assuming pumping hours to be 23 hours a day,

Discharge to be pumped = 20377/(23 x 60 x 60)

= 0.246 m3/s

Area of ports = Discharge / allowable velocity of flow

= 0.246 / 0.2

= 1.23 m^2

Let us provide inlet ports at 3 levels, one layer of opening(s) shall be kept below lowest water level ; and the other may be kept 1 meter below HFL . A middle layer of ports will be provided 1 meter below normal water level (NWL).

Let us provide 1 m height of opening, then the clear length of the openings required is = 1.23 /1

= 1.23 m

Provide Fine screens made up of 6 mm diameter stainless steel bars @ 10mm clear opening horizontally & @ 50 mm clear opening vertically.

No. of openings = 1.23 / 0.01

= 123 nos.

No. of vertical bars = 122 nos.

Length occupied by vertical bars horizontally = 0.006 x 122 = 0.73 m

Total length of screen required = 1.23 + 0.73 = 1.96 m

Provide 2 ports at each level with size of each port **1 m ht. x 1 m** ℓ ($2m^2$)

i.e, > 1.23 m^2.......(O.K)

2. Fine Screens :

Provide 1.2 m x 1.2 m size of fine screen covering each port and slightly projecting away from the port so that operation of gates is not hampered.

No. of horizontal bars	=	1.2 / 0.05 – 1 = 23 nos.
Height of openings	=	1.2 – 23 x 0.006
	=	1.06 m > port height of 1 m (O.K)
Length of one screen	=	1.2 m ... > 1.96/2 = 0.98 m ...(O.K)

3. Coarse Screens :

Coarse screen shall be located at the entry of the off-take. Galvanized iron pipes shall be used which shall be fixed vertically in a concrete frame having concrete beams spaced @ 3m vertically. Let us use 50 mm ϕ G.I pipe with external diameter of about 55 mm, arranged vertically @ 7 cm (5 to 8 cm) clear opening from one pipe to the next.

Width of off-take channel	=	5.1 m
No. of pipes required	=	(5.1 x 100)/12.5 – 1 = 40 nos
Area of pipes & frame	=	5.1 x 0.45 x 3 + 40 x 0.055 x 3.15
		+ (40 x 0.055 x 3.25) x 2 = 28.11 m^2
Area of screen	=	11 x 5.1 = 56.1m^2
Open area	=	56.1 – 28.11 = 27.99 m^2
Velocity of flow	=	0.246 m^3/s / 27.99 m^2/s = 0.009 m/s ..< 0.08 m/s (O.K)

Designed diagram of Intake is shown in Fig. 4.6

4.3 PUMPS AND PUMPING STATIONS

Pumping is needed to supply water, chemicals and residual streams. Sometimes, the first lift is required, just at the source, to lift the water to the treatment plant, and then a bigger lift from treatment plant to the city service reservoir. A pumping station is one of the major components of water supply systems and consists of one or more pumping units supported by four major sub-subcomponents: pump, driver (motor, engine, etc), power transmission, and controls. Each of these sub-subcomponents must function properly if the pumping unit is to be considered in an operational state. A pump can impart three types of hydraulic energy to a fluid: lift, pressure, and velocity. A common design practice is to install sufficient pumps to handle peak flows and include a spare pump of equal size to accommodate any downtime of the other pumps.

4.3.1 Types of Pumps

Out of various types of pumps, hydraulic engineers generally encounter with Roto-dynamic pumps and Displacement pumps. Roto-dynamic pump may be Centrifugal pump or Axial flow pump. Displacement pump may be reciprocating or rotary type pump. Depending upon the type of flow, roto-dynamic pump may be radial, mixed or axial flow pump. Radial flow and mixed flow machines are commonly called centrifugal pumps which are the most common and widely used, whereas the axial flow machines are called axial flow pumps.

The impeller of centrifugal pump may be open or closed type. The efficiency of open impeller centrifugal pump is generally much less than a closed one. But, since open impeller is less likely to be clogged by debris, etc., it is usually adopted for pumping raw water. For clear water closed type impeller are recommended. Depending upon the height of lift, pump required may be single or multistage pump. If only one impeller is used, it is known as single stage pump. If the impeller used is two or more, such that the discharge from one impeller enters the eye of the next impeller, it is called double stage or multistage pump. Such pumps are useful for high lifts.

4.3.2 Factors Effecting Selection of Pumps

The following factors must be thoroughly considered while selecting a particular type of a pump for a particular project:

1) Capacity of pumps
2) Importance of water supply scheme
3) Initial cost of pumping arrangement
4) Maintenance cost
5) Space requirement for locating the pump
6) Number of units required
7) Total lift of water required
8) Quantity of water to be pumped.

Based on the consideration of suction lift capacity, head and discharge, GOI 'Manual on water supply and treatment' provided general guidelines on the application and suitability of various types of pumps as shown in Table 4.1.

Table 4.1 : Application of Pumps

Pump type	**Suction-capacity to lift**			**Head range**			**Discharge range**		
	Low 3.5m	**Medium 6m**	**High 8.5m**	**Low upto 10m**	**Medium 10-40m**	**High > 40m**	**Low Upto 30L/s**	**Medium Upto 500L/s**	**High above 500L/s**
Centrifugal Horizontal End-suction	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	No
Centrifugal Horizontal Axial split Casing	Ok	No	No	Ok	Ok	No	No	Ok	Ok
Centrifugal Horizontal Multistage	Ok	Ok	No	No	Ok	Ok	Ok	Ok	No
Jet – Centrifugal Combination	When limitation of suction lift are to be overcome			Ok	Ok	No	Ok	No	No
Centrifugal Vertical turbine	When suction lifts is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok
Centrifugal Vertical submersible	When suction lifts is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok
Positive Displacement pumps	Normally self Priming			Limited only by the pressure which casing can withstand			Ok	Ok	Ok

4.3.3 Head, Power and Efficiency of Pumps

The total head (H) against which a pump has to work consists of

1) The suction lift (H_s)
2) The delivery head (H_d); and
3) The head lost due to friction, entrance and exit in the rising main (H_L)

$$\therefore H = H_s + H_d + H_L \qquad \text{.....................} \qquad (4.1)$$

The work done by the pump in lifting Q cumecs of water by a head H

$$= \gamma_w . Q . H. \quad \text{kN.m/s (i.e.J/s or Watts)} \quad \text{.........}(4.2)$$

Where, γ_w = unit wt. of water in kN/m^3; i.e, $9.81 kN/m^3$

Q = discharge to be pumped in m^3/s

The water horse power of the pump is then given as:

$$W.H.P = \frac{\gamma_{w.} Q.H}{0.735} \quad (\because 1\ HP = 735\ W) \quad \text{.........} \quad (4.3)$$

If η is the efficiency of the pump set, then the brake horse power of the pump is given by

$$B.H.P = \frac{W.H.P}{\eta} \quad \text{...........................} \quad (4.4)$$

4.3.4 Pump Stations

Pump stations are the structures that house pumps, piping, equipment, and chemicals. The design must ensure that the pump station meets the functional requirements of the pumping facility. In general, pump stations can be classified as wet-pit or dry-pit based on location of the pump. In wet-pit pump stations, the pumps are located in the wet well (Fig. 4.7a), whereas, in dry-pit pump stations, pumps are located in a dry enclosure separated from the wet well or sump (Fig.4.7b).

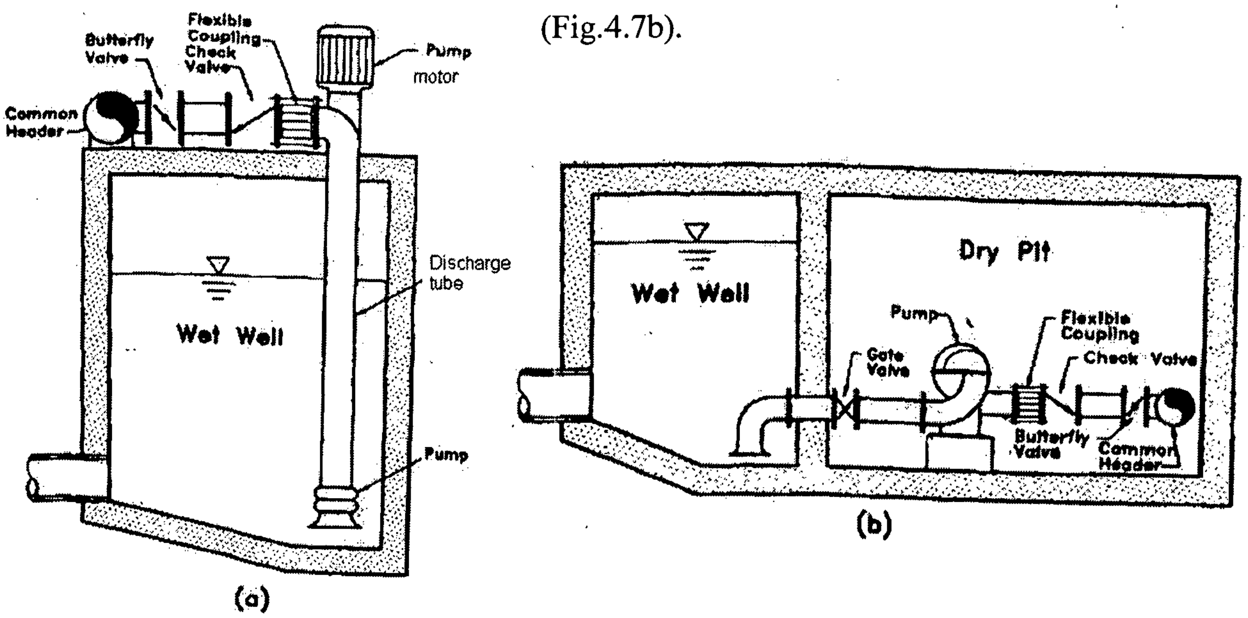


Fig.4.7.Common pump station design.(a) dry pit pump station. (b) wet-pit pump station.

Wet Well Design

The objectives to be considered in the design of intake are as follows:

(a) to prevent vortex formation to obtain uniform distribution of the inflow to all the operating pumps and to prevent starvation of any pump,

(b) to maintain sufficient depth of water to avoid air entry during draw down.

Guidelines for Intake (Well) Design

Figs. 4.8 (a, b, c) illustrate the recommended and non recommended practices for intake design. The following points are to be noted in this respect.

(a) To avoid mutual interference between two adjoining pumps, the dimension 'S' in fig.4.8 should be minimum 2D. It is also advisable to provide dividing walls between the pumps. The walls should have rounded or Ogive ends.

(b) Avoid dead spots by keeping rear clearance, the dimension B to about 5D/6 from the centre line of the pump. A dummy wall must be provided, if necessary.

(c) Provide tapered walls between the approach channel and the sump as shown in fig.4.8c to reduce the velocity to about 0.3 m/s near the pump.

(d) Avoid dead spots at the suction bell mouth by maintaining the bottom clearance, dimension C between D/4 to D/2, preferably D/3.

(e) Avoid sudden drop between the approach channel and the sump. A slope of maximum 15^{o} is recommended. The floor underneath the pump suction should be flat up to 3D.

(f) Keep adequate submergence of the pump under LWL, dimension H in Fig. 4.9, so as to prevent entry of air during draw down and to satisfy $NPSH_r$.

For the typical dimensions A and H in Fig. 4.8, GOI 'Manual on water supply, 1991' give recommendations based on main stream velocity of 0.6 m/s as given in Table 4.2.

Table 4.2: Recommendations Regarding Intake Design

Flow rate in m^3/h	Minimum submergence Dimension 'H', m	Position of trash rack dimension 'A', m
1000	1.23	3.28
1600	1.5	5.20
2500	1.8	8.07
4000	2.17	12.83
6400	2.63	20.37
10000	3.15	31.61

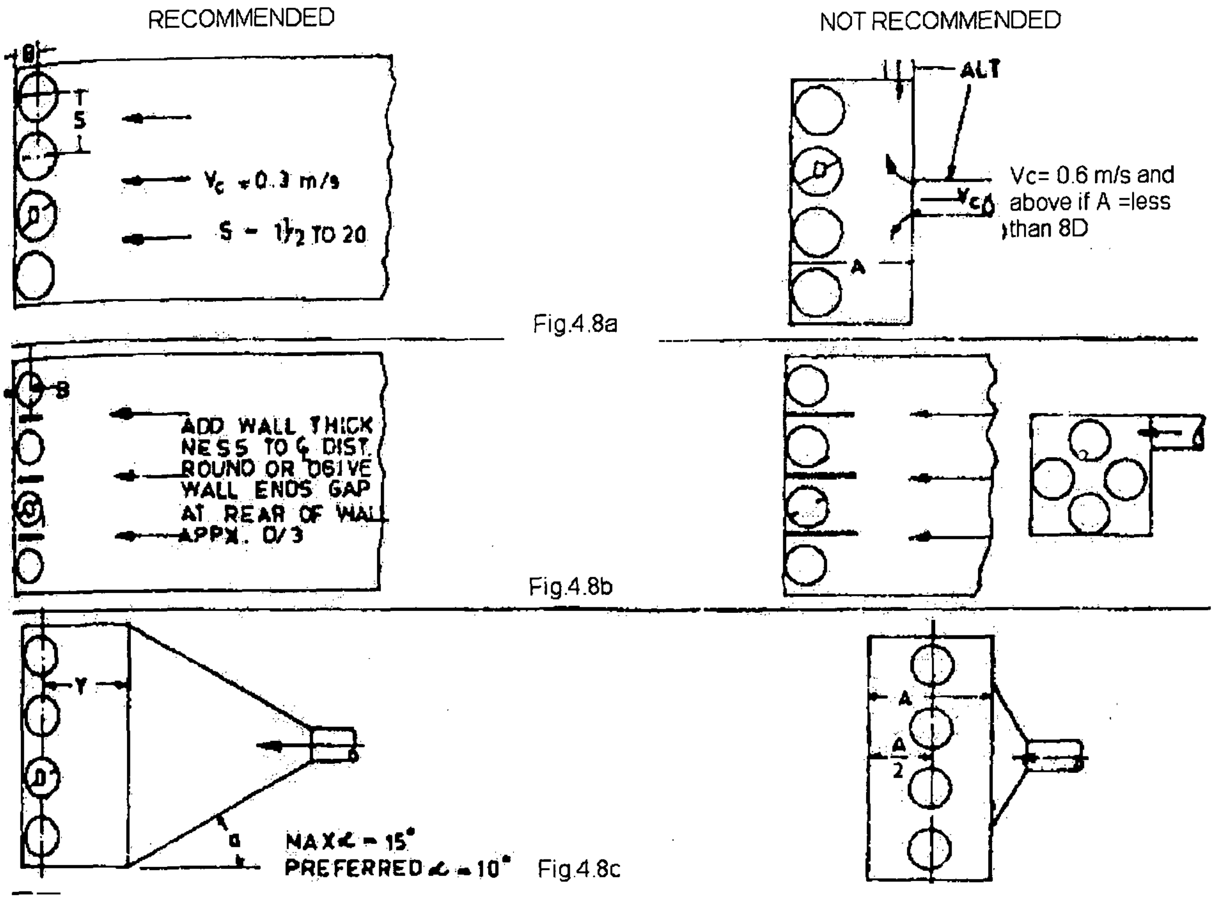


Fig.4.8. Multiple Pump Pits

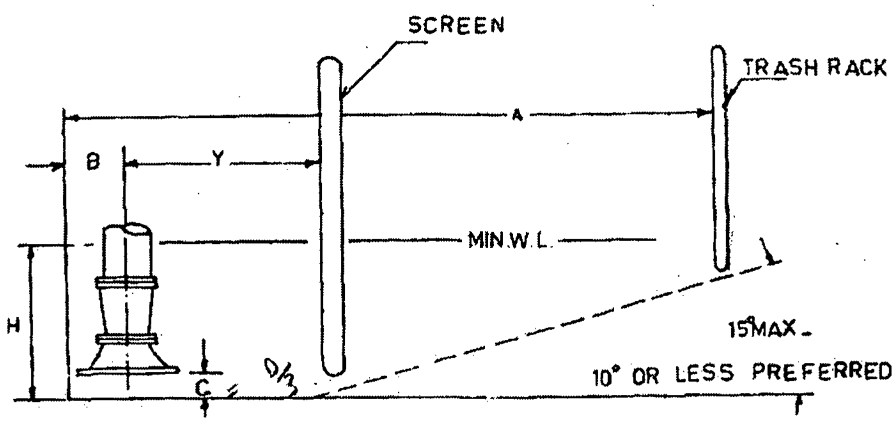


Fig.4.9. Sump Dimensions Elevation View

Suction Piping

For suction piping design, the following should be considered.

(a) The suction piping should be considered as short and straight as possible. Bends or elbows should be of long radius.

(b) As a general rule, the size of the suction pipe should be one or two sizes larger than the nominal suction size of pump or velocity of flow shall be about 2m/s. Where bell mouth is used, the diameter shall be such that velocity through it shall be about 1.5m/s.

Discharge Piping

(a) The diameter of the discharge pipe may be one size higher than the nominal delivery size of the pump or the velocity through it shall be about 2.5 m/s.

(b) Discharge piping connection to a common manifold or header should be connected by radial tee or by 30° or 45° bend.

Valves

(a) When suction lift is encountered, a foot valve is provided to facilitate priming. The pump can be primed also by vacuum pump, if the pump is of large size.

(b) When there is positive suction head, a sluice or butterfly valve is provided on the pump suction, for isolation.

(c) A non-return (reflux) valve and a delivery valve (sluice or butterfly valve) should be provided near to the pump. The non-return valve shall be in between pump and delivery valve.

Space Requirement and Layout Planning of Pumping Station

(f) Sufficient space should be available in the pump house to locate the pump motor, valves, pipings, control panels and cable trays with easy access and with sufficient space around each equipment for maintenance and repairs. The minimum space between two adjoining pumps or motors should be 0.6m for small and medium units and 1m for large units.

(f) For control panels, there should be a clear space of not less than 915 mm in width in front of the switch board.

4.3.5 Design of Raw Water Pump House for Almora

As mentioned previously, the intake for Almora water treatment system will be of jack well intake. Raw water pump house shall be located on top of jack well for low lifting of water to pre-sedimentation tank. Design for capacity of pump is done as below.

Data:

- RL of LWL in the sump	:	1131 msl
- RL of center line of pump	:	1143 msl
- RL of point of discharge in sedimentation tank	:	1150 msl

- Raw water pumping main	:	
Length	:	300 m
Diameter	:	500 mm
- Design discharge	:	$0.2358\ m^3/s$ to be pumped in 23 hrs.
	≈	$0.2358 \times 24 / 23 = 0.246\ m^3/s$

Total head against which water has to be pumped (H)

H = suction lift + discharge lift + head loss due to friction

Let us use 500 mm diameter m.s. ERW pipe for transmission main.

Head loss due to friction in the line:

Using Modified Hazen-Williams formula, i.e.,

$$H_L = \frac{L\left(\frac{Q}{C_R}\right)^{1.81}}{994.62 d^{4.81}}$$

$$= \frac{300\left(\frac{0.246}{1}\right)^{1.81}}{994.62 x 0.5^{4.81}}$$

$$= 0.67\ m$$

Other losses in the fittings	=	10 % of friction loss
	=	0.067 m
Total head loss due to friction	=	0.73 m
H = 12 + 7 + 0.73	=	19.73 m
Water power required	=	$\gamma_w . Q . H$
	=	9.81 x 0.246 x 19.73 KN.m/s or KW
	=	47.61 KW
	=	47.61/0.735 = 64.78 HP

Assuming combined efficiency of pump set to be 0.65.

Power of motor required to run pumps

	=	64.78/0.65 = 99.6 HP
Add 10% for prime mover	=	9.96
Total power of motor	=	99.6 + 9.96 = 109.56 HP, Say 110 HP

Provide **single stage, centrifugal vertical turbine pump, 3 pumps, each of 55 HP** capacity, 2 pumps will be operating and 1 pump as a standby for repairs, etc.

Design of Sump and Other Components of Pump house

Size of pipes & fittings for the pumping system

Inlet bell mouth

Design velocity	....	1.5 m/s
Bell mouth diameter	....	0.323 m
		Say 350 mm

Column pipes

Design velocity	....	2.5 to 3 m/s
		Say 2.75 m/s
Column pipe diameter	....	0.238 m
		Say 250 mm

Delivery pipes and valves

Design velocity	....	2.5 m/s
Diameter of delivery pipe, delivery valve & NRV		0.2503 m
		Say 300 mm

Sump Dimensions

(a) Average clearance between bottom of sump and lip of suction bell mouth,

C = D/3 [D/4 to D/2]

= 350/3

= 116.6 mm , Let us take 150 mm

(b) Distance between rear wall and centre line of bell mouth

B = 3D/4 or 5D/6

= 3 x 350 / 4 or 5 x 350/6

= 262.5 mm or 291 mm

Let us take 280 mm

(c) Spacing between pumps

Desirable spacing between pumps is 2.5D i.e., 2.5 x 350 = 875 mm or $3.5D_{column}$ = 3.5 x 250 = 875 mm. Considering to accommodate flexible coupling,etc., and clearance required for routine operations and maintenance needs, take the c/c spacing of pumps at 1600 mm.

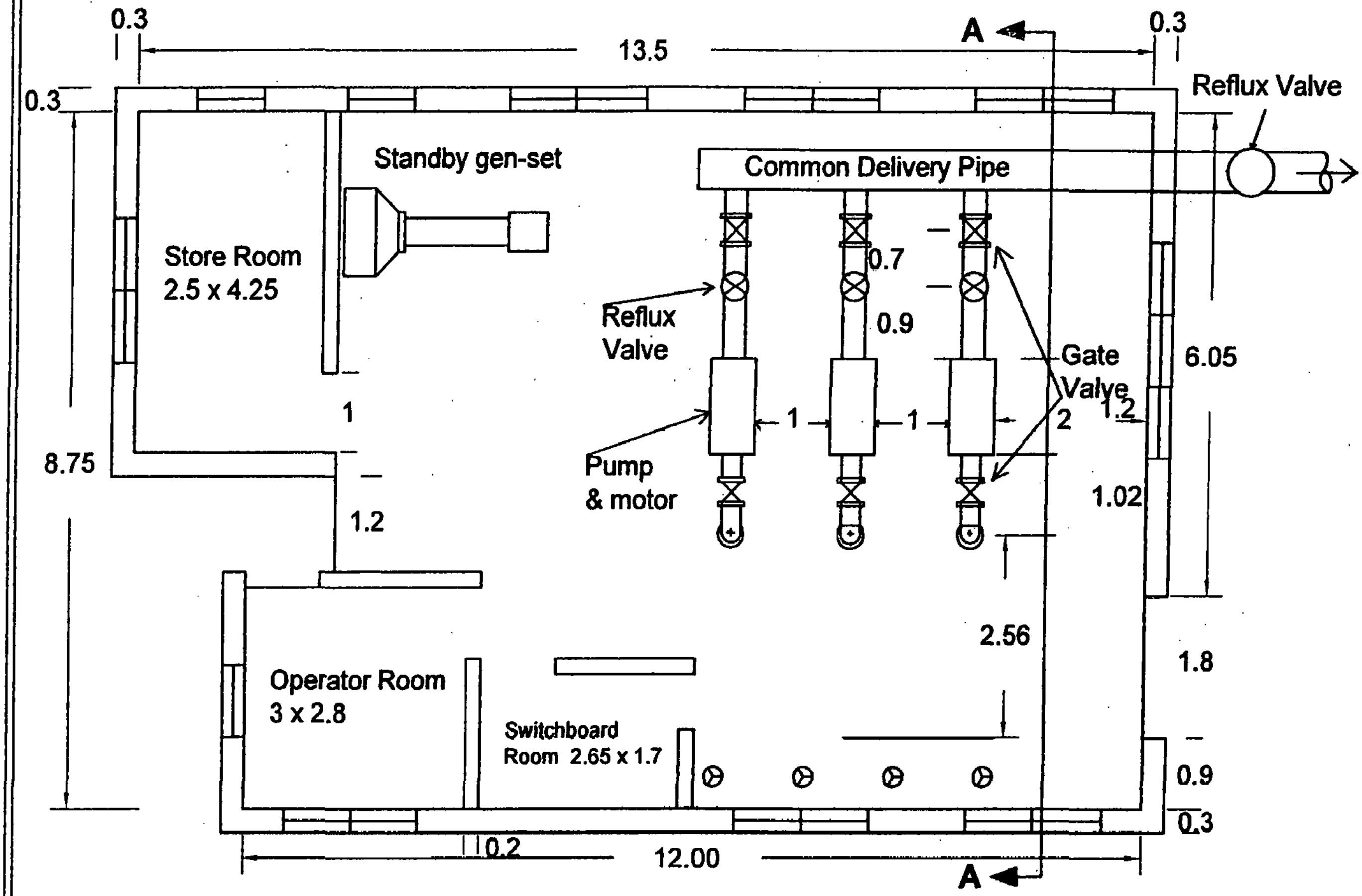


Fig.12.7 (a) PLAN OF PUMPHOUSE

Note :
All Dimensions are in m

Fig.4.6. DETAILS OF JACKWELL INTAKE CUM PUMPHOUSE. (a) Plan of Pumphouse (b) Front View of Jackwell Intake showing Ports,etc. (c) Section AA

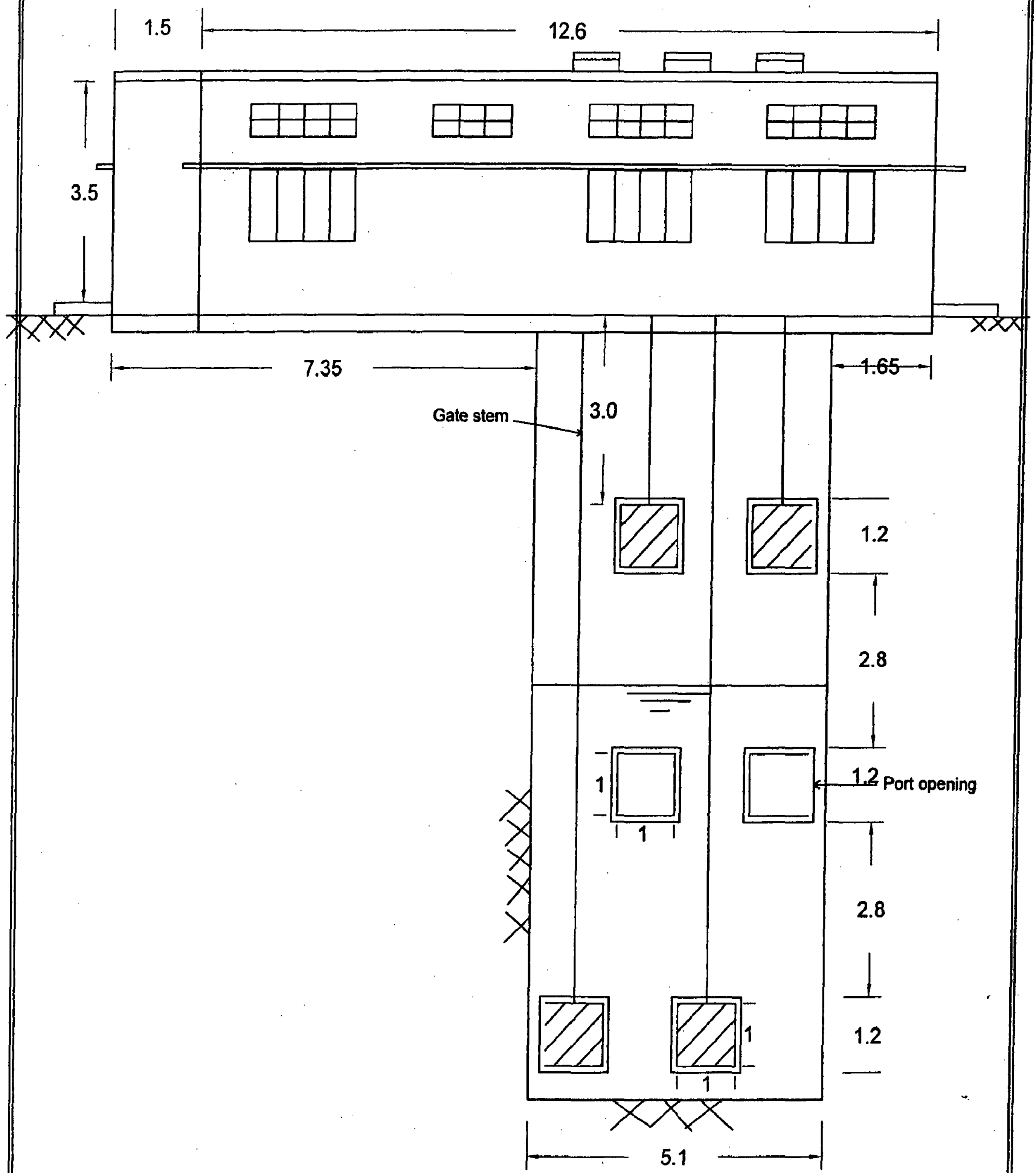


Fig. 4.6 (b) FRONT VIEW OF JACKWELL INTAKE

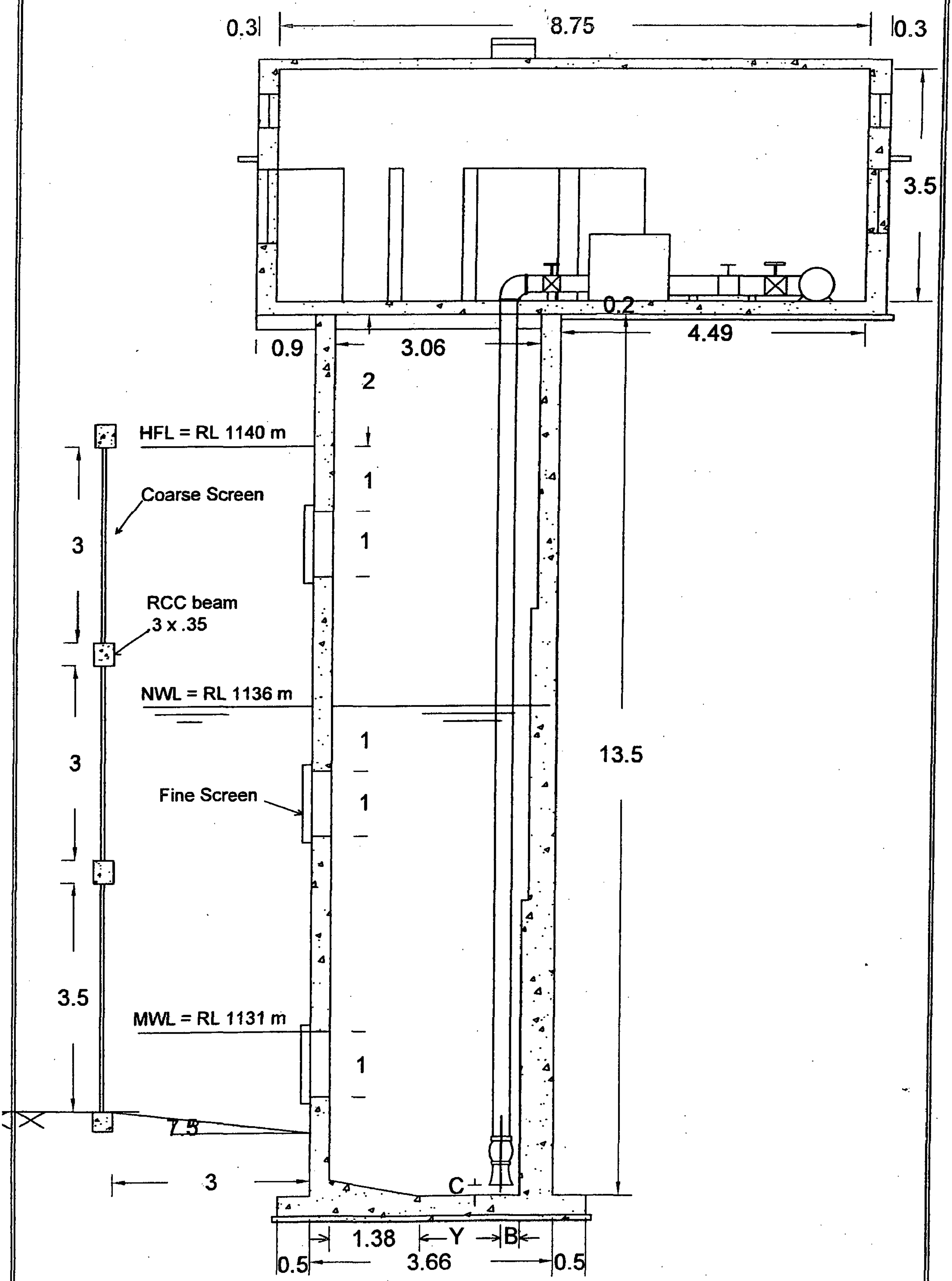


Fig.4.6 (c). SECTION - AA

4.4 CONVEYANCE OF WATER

In most cases, water conveyance is necessary from its source to the water treatment facility and from treatment facility to the consumers, through free flow channels or conduits or pressure mains. Depending on topography and local conditions, conveyance may be in free/gravity flow and/or pressure conduits. Transmission of water accounts for an appreciable part of the capital outlay and hence careful consideration of the economics is called for, before deciding on the best mode of conveyance. Based on the condition of flow, conduits are gravity or pressure conduits.

4.4.1 Conduits

Gravity Conduits

Gravity conduits are those in which water flows under the mere action of gravity. In such a conduit, the hydraulic gradient line will coincide with the water surface and will be parallel to the bed of the conduit, as shown in Fig. 4.10. This is so, because in such a flow, water is all along at atmospheric pressure and thus there is no pressure term in Bernoulli's equation. Gravity conduits can be in the form of canals, flumes, or aqueduct.

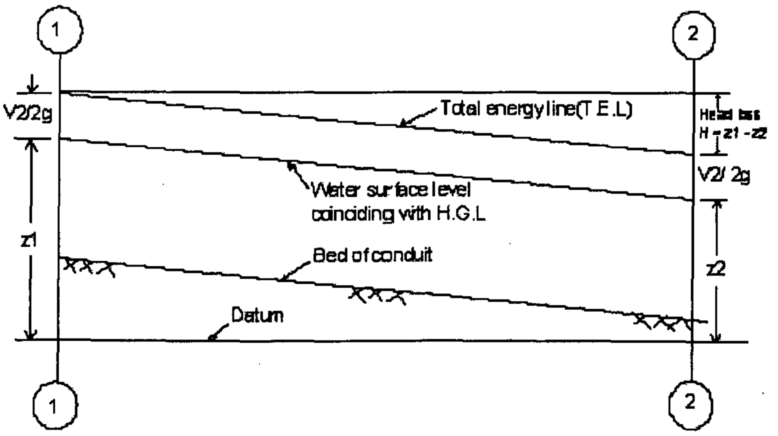


Fig.4.10. Flow Illustration in Gravity Conduit

Pressure Conduits

Pressure conduits are enclosed pipes, closed aqueducts and tunnels called pressure aqueducts or pressure tunnels. Water flows at a pressure higher than the atmospheric pressure. In other words, the hydraulic grade line is above the crown of the conduit; therefore, pressure pipes can follow the natural ground surface, provided the ground surface is below the hydraulic grade line. If pipe line is laid at any point above the hydraulic gradient line, it would develop siphonage and negative pressure in the pipe.

The hydraulic gradient line for such a conduit can be obtained by joining the water surface elevations in the piezometers installed in the conduit at various places, as shown in Fig. 4.11.

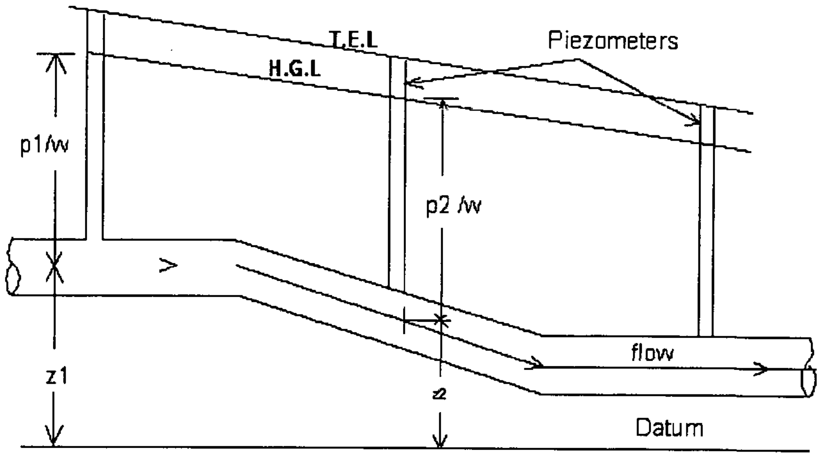


Fig.4.11. Flow Illustration in Pressure Conduit

4.4.2 Hydraulics of Conduits

The design of supply conduits is dependent on resistance to flow, available pressure head, allowable velocities of flow, scour, sediment transport, quality of water and relative cost. There are a number of formulae available for use in calculating head loss due to friction in the pipe. However, Hazen-Williams formula for pressure conduits and Manning's formula for free flow conduits have been popularly used.

(a) Hazen-Williams Formula

The Hazen-Williams formula is expressed as:

$$V = 0.849\, C\, R^{0.63}\, S^{0.54}$$

For circular conduits, the expression becomes

$$V = 0.355\, C\, D^{0.63}\, S^{0.54}$$

$$Q = 0.278\, C\, D^{2.63}\, S^{0.54}$$

$$h_L = \frac{10.7\, L Q^{1.852}}{C^{1.852}\, D^{4.87}}$$

Where, Q = discharge in m^3 / sec.

D	=	diameter of pipe in m
V	=	velocity in the pipe , m/s
R	=	hydraulic radius in m
S	=	slope of hydraulic gradient (h_f/L), dimensionless
C	=	Hazen-Williams Co-efficient of Roughness
L	=	length of pipe in m
H_L	=	head loss due to friction in the pipe, m

Table 4.3: Hazen-William Coefficients

Pipe Material	Recommended C Values	
	New Pipes	Design Purpose
Unlined Metallic Pipes		
Cast Iron, Ductile Iron	130	100
Mild Steel	140	100
Galvanised Iron above 50 mm dia.	120	100
Galvanised Iron 50 mm dia. and below used for house service connections.	120	55
Centrifugally Lined Metallic Pipes		
Cast Iron, Ductile Iron and Mild Steel pipes lined with cement mortar or Epoxy		
Up to 1200 mm dia.	140	140
Above 1200 mm dia	145	145
Projection Method Cement Mortar Lined Metallic Pipes		
Cast Iron, Ductile Iron and Mild Steel pipes	130 *	110 **
Non Metallic Pipes		
RCC Spun Concrete, Pre-stressed concrete		
Up to 1200 mm dia.	140	140
Above 1200 mm dia.	145	145
Asbestos Cement	150	140
PVC, GRP and other Plastic pipes	150	145

* *For pipes of diameter 500 mm and above; the range of C values may be from 90 to 125 for pipes less than 500 mm*

.***In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher values up to 130 may be adopted.*

(b) Darcy-Weisbach's Formula

Darcy and Weisbach suggested the first dimensionless equation for pipe flow problems as,

$$h_L = \frac{f L V^2}{2 g D}$$

Where, h_L = head loss due to friction over length L, m

f = dimensionless friction factor

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

V = velocity in m/s

L = length of pipe in meters

D = diameter in meters

Table 4.4: Recommended Friction Factors In Darcy-Weisbach Formula

S/No.	Pipe Material	Diameter (mm)		Friction Factor	
		From	To	New	For Design period of 30 yrs
1	R.C.C	100	2000	0.01 to 0.02	0.01 to 0.02
2	A.C	100	600	0.01 to 0.02	0.01 to 0.02
3	HDPE / PVC	20	100	0.01 to 0.02	0.01 to 0.02
4	SGSW	100	600	0.01 to 0.02	0.01 to 0.02
5	C.I. (for corrosive water)	100	1000	0.01 to 0.02	0.053 to 0.03
6	C.I. (for non-corrosive water)	100	1000	0.01 to 0.02	0.034 to 0.07
7	Cement Mortar or Epoxy lined metallic pipes (C.I., D.I, Steel)	100	2000	0.01 to 0.02	0.01 to 0.02
8	G.I.	15	100	0.014 to 0.03	0.0315 to 0.06

(c) dified Hazen-Williams Formula

The Modified Hazen-Williams formula has been derived from Darcy-Weisbach and Colebrook-White equations and obviates the limitations of Hazen-Williams formula. For circular conduits, ν_{20^0C} for water = 10^{-6} m^2/s and g = 9.81 m/s^2, The Modified

Hazen-Williams formula is derived as

$$H_L = \frac{L\left(\frac{Q}{C_R}\right)^{1.81}}{994.62\,d^{4.81}}$$

Where, H_L = friction head loss in m

Q = flow in pipe in m^3/s

L = length of pipe in m

C_R = dimensionless coefficient of roughness

d = pipe diameter

Table 4.5

Recommended C_R Values in Modified Hazen-Williams Formula (at 20^0 C)

Sl. No.	Pipe Material	Diameter(mm)		Velocity(m/s)		CR value when New	CR value For design period of 30 years
		From	To	From	To		
1	RCC	100	2000	0.3	1.8	1.00	1.00
2	AC	100	600	0.3	2	1.00	1.00
3	HDPE and PVC	20	100	0.3	1.8	1.00	1.00
4	CI/DI (for waters with positive Langelier's index)	100	1000	0.3	1.8	1.00	0.85
5	CI/DI (for waters with negative Langelier's index)	100	1000	0.3	1.8	1.00	0.53*
6	Metallic pipes lined with cement mortar or epoxy (for water with negative Langelier's index)	100	2000	0.3	2.1	1.00	1.00
7	SGSW	100	600	0.3	2.1	1.00	1.00
8	GI (for waters with positive Langelier's index)	15	100	0.3	1.5	0.87	0.74

* *These are average C_R values which result in a maximum error of 65% in estimation of surface resistance.*

4.4.3 Discussion on Various Formula for Estimation of Frictional Resistance

(1) If there is a choice for use of pipe friction formulae, Darcy Weisbach yields accurate results but involves extra computational effort and therefore Hazen-Williams formula is commonly used.

(2) Darcy-Weisbach formula is dimensionally consistent. However, the values of its friction factor 'f' which depends upon Reynold number (i.e.R_e= Vd/ν) and relative roughness (i.e.δ = 2e/d) of the pipe are not certain. Hence, to use this formula requires a trial and error solution, making it very complicated.

(3) The Hazen-Williams coefficient C is usually considered independent of pipe diameter, velocity of flow and viscosity. A comparison between estimates of Darcy-Weisbach friction factor f, and its equivalent value computed from Hazen-Williams C for different pipe materials brings out the error in estimation of 'f' up to 645% in using Hazen-Williams formula. It has been observed that for higher 'C' values (new and smooth pipes) and larger diameters, the error is less, whereas it is appreciable for lower 'C' values (old and rough pipes) and lower diameters at higher velocities.

(4) The Modified Hazen-William formula being an improvement was suggested for use in-lieu- of Hazen-William formula. Darcy- Weisbach formula coupled with Colebrook-White equation gives most accurate results followed by modified Hazen-Williams formula and Hazen-Williams formula.

4.4.4 Minor Losses in Pipelines

Pipeline transitions and appurtenances add to the head loss, which is expressed as velocity head as $KV^2/2g$ where V and g are in m/s and m/s^2 respectively or equivalent length of straight pipe. The values of K to be adopted for different fittings are given in Table 4.6. Where the length of pipe is greater than 1000 times its diameter, the loss of head due to valves and fittings is disregarded. However, in individual cases when head losses are required to be calculated in respect of entrance losses, sudden enlargement losses, sudden bends or elbows, etc. the head loss formula have to be used by taking appropriate value of K.

Table 4.6: K Values For Different Fittings

Items	Value of K
Entrance : Bell mouth entry Square edge entry	0.04 0.5
Exit	1.0
Sudden contraction	0.3* - 0.5
Sudden enlargement	0.17** - 1.0
Elbow : 90^o 45^o 22^o	0.5 - 1.0 0.4 - 0.75 0.25 - 0.5
Tee 90^o Take-off Straight run	1.5 0.3
Coupling	0.3
Return bend	2.2
Gate valve (open)	0.3*** - 0.4
Gate valve (half open)	5.6
Glove valve (wide open)	10.0
With reducer and increaser	0.5
Globe	10.0
Angle	5.0
Swing check	2.5
Venturimeter	0.3
Orifice	1.0

* *Varying with area ratio,* *** *Varying with radius ratios*
** *Varying with angle between sides of tapering section*

4.4.5 Cost Effective Design of Pipeline

The hydraulic design of pipe line is mainly concerned with required and allowable velocities of flow, resistance to flow with respect to available or required heads or pressures. The velocities are determined by the quality of water to be transported and the magnitude of water hammer. The design velocity should not be less than 0.6 to 0.75 m/sec in order to avoid depositions and consequent loss of carrying capacity. The maximum velocity should be also such that it neither causes erosion or scour nor should it endanger the pipe with excessive water hammer due to rapid closing of valves, etc. Velocities of 1.22 to 1.83 m/sec are common but the upper limits lie between 3 to 6.1 m/sec for most of the materials.

For pumping mains, the economical diameter is that diameter, which together with the pumping cost, will make the total annual expenses minimum. An empirical formula for assessment of economical diameter given by Lea, which is commonly used in practice, is given by

$$D = 0.97 \text{ to } 1.22\sqrt{Q}$$

Where,

D = economical diameter in m

Q = discharge to be pumped in m^3/s

This relationship gives optimum flow velocity varying between 0.8 to 1.35 m/s

4.4.6 Design of High Pressure Pipeline

In hilly areas, pumping of water at a very high static head is always required. In that case, the working pressure at the bottom section of a pipe line would be very high requiring designing the wall thickness of pipe line for that section properly. In important and high head rising main, it is also usual to incorporate devices to suppress water hammer. The primary factor which determines the wall thickness of a pipe are the materials from which it is manufactured, the process of manufacture and the working pressure to which the pipe is subjected to. The maximum pressure to which the pipe is subjected to is the sum of hydraulic pressure at a particular point and the surge pressure as calculated. The maximum pressure would develop just adjacent to the pump and the least at the end of the main.

The thickness of straight pipe shall be calculated in accordance with the following equation:

$$t_m = t + c$$

where,

t_m = minimum required thickness in inches including mechanical corrosion and erosion allowances.

t = pressure design thickness in inches as calculated by Barlow's equation.

c = the sum in inches of mechanical allowance (thread depth or groove depth) plus corrosion or erosion allowances.

Barlow's Equation: $t = \dfrac{PDO}{2SE}$

Where, P = internal design pressure ,PSI

DO = outside diameter of pipe, inches

SE = allowable stress, PSI

S = basic allowable stress for materials excluding Quality or joint factor

E = joint efficiency or quality factor applied to S as required.

4.4.7 Anchorages

Anchorages are required for one or more of the following reasons:

(a)To resist the tendency of pipe to pull apart at bends and other points when unbalanced pressure occurs that exceeds the resistance of their joints to their longitudinal stresses.

(b)When the resistance of the joints to longitudinal stresses are inadequate and the pipes have the tendency to pull apart when pipe is laid on steep ground.

The unbalanced static pressure at bends is computed by using the following equation:

$$T_t = 2PA \sin\frac{\theta}{2}$$

Where, T_t = total internal radial thrust

P = intensity of pressure in kg/cm^2

A = area of pipe in sq.cm

θ = angular change in direction or angle of a bend in degrees

Anchorages take many forms. For bends, both horizontal and vertical they may be designed as a concrete buttresses or kick blocks that resists the unbalanced pressure by their weight. In addition to to frictional resistance on the bottom of the thrust block and the circumference of the pipeline, there is lateral resistance against the outer face of the pipe and block. The lateral resistance of soil against the thrust block is given by

$$Fp = \gamma_s \frac{H^2}{2}\left(\frac{1+\sin\theta}{1-\sin\theta}\right) + 2CHL\sqrt{\frac{1+\sin\theta}{1-\sin\theta}}$$

This maximum possible resistance will only be developed if the thrust block is able to move into the soil mass slightly. The minimum pressure which may occur on the thrust block is the active pressure, which may develop if the thrust block were free to yield away from the soil mass.

$$f_a = \gamma_s h \left(\frac{1-\sin\theta}{1+\sin\theta}\right) - 2C\sqrt{\frac{1-\sin\theta}{1+\sin\theta}}$$

Where, Fp	=	Lateral resistance of soil against the thrust block in tons
f_a	=	Lateral resistance of soil against the projection of pipe in t/m^2
γ_s	=	soil density in T/m^3
h	=	depth on m
θ	=	angle of friction in degrees
C	=	cohesion of soil in N/m^2
H	=	height of thrust block in m
L	=	length of thrust block in m.

4.4.8 Design of Pipelines for Almora Augmentation W/S/S

The pipelines to be designed for Almora water treatment system includes raw water transmission main from intake to pre-sedimentation tank, rising main from pre-sedimentation tank to main water treatment plant at Matela, as shown in Fig.3.4.

Economical Diameter of Raw Water Transmission Main

Using empirical formula given by Lea, which is as below:

D = 0.97 to $1.22\sqrt{Q}$,

where, Q = discharge to be pumped in cumecs considering 23 hrs a day pumping assuming one hour loss due to tripping and other minor interruptions.

= 0.2358 m^3/s x 24/23

= 0.246 m^3/s

∴ D = 0.48 m to 0.605 m

= 480 mm to 605 mm, *Say **500 mm***

Design For Economical Diameter of Pumping Main Between 1^{ST} Stage Pump House To Main Water Treatment Plant

Problem :

(1) *Period*	*Year*	*Demand* (mld)	*Demand including 5 % treatment loss* (mld)	m^3/sec.
Initial	2011	10.9	11.45	0.132
Intermediate	2026	14.9	15.66	0.181
Ultimate	2041	19	19.97	0.231

(2) Length of pumping main : 13000 m

(3) Static head for pump : 58 m

(4) Design period : 30 years

(5) Combined efficiency of pumping set : 60 %

(6) Cost of pumping unit : Rs. 12,000/KW

(7) Interest rate : 10 %

(8) Life of electric motor and pumps : 15 years

(9) Energy charges : Rs. 4.50 / unit

(10)Design value of C_R for M.S-ERW pipe : 1

Solution :		1^{st} 15 years	2^{nd} 15 years
1.	Design installation	11.45 mld	15.66 mld
2.	Discharge at end of 15 yrs	15.66 mld	19.97 mld
3.	Average discharge	13.55 mld	17.81 mld
4.	Hrs. of pumping for discharge at end of 15 yrs.	23 hrs.	23 hrs.
5.	Average hrs. of pumping for average discharge	23x 13.55/15.66 = 19.9 hrs	23 x 17.81/19.97 = 20.51 hrs.
6.	KW required at 60 % combined eff. of pumping set (at end of 15 yrs.)	$\frac{9.81 \times 0.181 H_1}{0.60} = 2.96\ H_1$	$\frac{9.81 \times 0.231\ H_2}{0.60} = 3.77\ H_2$

(where, H is total head in m for discharge at end of 15 yrs.)

7.	Annual cost in Rs. of electrical energy @ Rs.3.15 /unit	KW_1x 19.9 x 365.25 x 4.5 = 32708.14 KW_1	KW_2 x 20.51 x 365.25x x 4.5 = 33710.75 KW_2

8. Pump cost capitalised

   $P_n = C = P_o(1 + r)^n$

   Where,

   P_o = initial capitalized investment

   C = amount needed after 15 yrs to purchase 2[nd] stage pumping set.

   r = rate of compound interest

   n = no. of years, 15

   $P_o = C/(1 + r)^n$

   $= C/4.177$

9. Capitalised energy cost

   $Cc = C_R \{(1 - (1 + r)^{-n})/n\}$

   $Cc = 7.606\ C_R$

   $Cc_1 = 7.606\ Cr_1$

   $Cc_2 = 7.606\ Cr_2$

10. Tables 4.7, 4.8 & 4.9 show the calculations to arrive at the most economical pumping main size for the given data of Almora augmentation water supply scheme. It is seen that 500 mm diameter M.S-ERW pipe is most economical diameter. Hence, it may be selected.

Design for Wall Thickness of Rising Main 1 (From 1[ST] Stage Pump House to Treatment Plant)

Data :

1. Pipe status : 500 mm diameter M.S.-ERW , Grade Yst 240 pipe , welded and standard weight conforming to IS- 1978/1982
2. Working pressure : 88.5m total head i.e., 8.85 kg/cm^2
3. Velocity of flow : 1.22 m/s (4'/s)

Solution :

Working pressure = 8.85 kg/cm^2

= 8.85 x 14.2233 PSI = 126 PSI

Water hammer effect (value as per table 246 of practical handbook, for r/t =250/9.5 = 26.31 for a velocity of 1'/s, i.e., for 4 secs. = 52

= 52 x 4 PSI = 208 PSI

Total design pressure rounded higher unit of 10 (P) = 340 PSI

Outer diameter of pipe (D.O.) = 508 mm or 20"

Standard test pressure for Yst 240, S = 145 MPa

Table – 4.7

TABLE SHOWING VELOCITY AND LOSS OF HEAD FOR DIFFERENT PIPE SIZE

S/No.	Pipe size (mm)	Frictional head loss per 1000 m		Velocity in m/s		Total head in m for 13000 m pipe lengths including 58 m of static head					
						1st stage flow			2nd stage flow		
		1st stage flow of 15.66 mld	2nd stage flow of 19.97 mld	1st stage flow of 15.66 mld	2nd stage flow of 19.97 mld	Frictional loss (m)	Other losses (m)	Total (m)	Frictional loss (m)	Other losses (m)	Total (m)
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
1	300	14.9	23.2	2.48	3.35	193.7	19.37	271	301.6	30.16	390
2	350	7.1	11.05	1.84	2.4	92.3	9.23	160	143.65	14.37	216
3	400	3.74	5.81	1.45	1.85	48.6	4.86	111	75.53	7.55	141
4	450	2.12	3.3	1.18	1.49	27.6	2.76	88	42.9	4.29	105
5	500	1.28	1.98	0.9	1.23	16.6	1.66	76	25.74	2.57	86
6	600	0.53	0.83	0.65	0.85	6.9	0.69	66	10.79	1.08	70

Note : Assuming other losses 10 % of frictional loss.

Table – 4.8

TABLE SHOWING KILOWATTS REQUIRED AND COST OF PUMP SETS FOR DIFFERENT PIPE SIZE

Sl/No.	Pipe size (mm)	1st stage flow of 23.05 mld				2nd stage flow of 29.4 mld			
		H1 total head	KW required (2.96 H1) (KW_1)	KW required incl. 50 % standby	Cost of pumpset at Rs.12000 /KW (Rs. In '000)	H2 total head	KW required (3.77 H2) (KW_2)	KW required incl. 50 % standby	Cost of pumpsetat Rs. 12000/KW (Rs. In '000)
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
1	300	271	802	1203	14439	390	1470	2205	26465
2	350	160	474	710	8525	216	814	1221	14658
3	400	111	329	493	5914	141	532	797	9568
4	450	88	260	391	4689	105	396	594	7125
5	500	76	225	337	4049	86	324	486	5836
6	600	66	195	293	3516	70	264	396	4750

Table – 4.9

COMPERATIVE COST OF OVERALL COST STRUCTURE OF PUMPING MAIN FOR DIFFERENT PIPE SIZE

Sl/no	Pipe size (mm)	Class of pipe required	Rate per m length (Rs.)	Cost of 13000m pipe length in '000 (Rs.)	1st stage flow of 23.05 mld				2nd stage flow of 29.4 mld			initial capi. investment for pumpset & annual energy cost in '000 Rs.	Grand total of capitalised cost for 30 years in '000 Rs.
					cost of pumpset in '000 Rs	annual energy cost in 000 Rs.	capitalised energy cost in 000 Rs.	total capitalised cost in 000 Rs.	cost of pumpset in '000 Rs	annual energy cost in 000 Rs.	capitalised energy cost in 000 Rs.		
1	**2**	**3**	**4**	**5**	**6**	**7***	**8***	**9***	**10**	**11***	**12***	**13***	**14***
1	300	m.s.- ERW	3902.27	50730	14439	26232	199521	264689	26465	49555	376915	96572	361261
2	350	240 grade	4284.94	55704	8525	15504	117923	182153	14658	27441	208716	53477	235630
3	400	pipe as	4897.5	63668	5914	10761	81848	151430	9568	17934	136406	34947	186377
4	450	per	5508.1	71605	4689	8504	64681	140976	7125	13349	101532	26013	166989
5	500	IS:1978	6123.62	79607	4049	7359	55973	139629	5836	10922	83073	21285	**160914**
6	600	-1982	7381.12	95955	3516	6378	48511	147982	4750	8900	67693	17343	165325

7* = 32708.14KW1 = CR1
8* = CC1= 7.606 CR1
9* = (5) + (6) + (8)

11* = 33710.75 KW2 = CR2
12* = CC2 = 7.606 CR2
13* = (col 10 + 12)/4.177

14* = (9) + (13)

* Cost of pipe includes cost of specials, carriage, laying and jointing .

REMARHS : From this table, it is seen that 500 mm M.S-ERW pipe is most economical diameter.

(Table 134 of practical handbook on public health engineering by Bajwa)

Joint factor for weld, E (Table 142)= 0.85

Using formula ...... $t_m = t + C$

Where, tm	=	min. required thickness in inches including mechanical corrosion and erosion allowance
t	=	PDO/2 SE (pressure design thickness in inches as calculated for internal pressure in PSI
C	=	corrosion allowance 0.1" from table 250 [Bajwa]

Now, SE = 145 x 0.85 = 123.25 Mpa = 17876 PSI

Mill tolerance (IS 1978)= 12.5 %

$\therefore$ t = (340 x 20)/ (2 x 17876)

= 0.19"

$\therefore$ t_m = 1.125 (0.19 + 0.1)

= 0.326 "or 8.29 mm

Provide next higher manufactured thickness i.e., **9.5 mm**

4.4.9 Design of Thrust/Anchor Blocks along Rising Main 1

On survey of the pumping main pipelines, it was found that there would be numerous bends in the proposed pipelines due to hilly and undulating topography of the land with deviation angle generally not more than 30°. Hence, we shall design for a thrust/anchor block for this angle of deviation which may be used throughout the pipeline wherever these thrust/anchor blocks are felt necessary to be on the safe side.

Design Data:

1.	Diameter of pipe	:	500 mm
2.	Type of pipe	:	ERW pipe as per IS-1978/1982
3.	Angle of deviation (α)	:	30°
4.	Internal water pressure (p) (at bottom most section)	:	6 kg/cm^2 for rising main 1
5.	Density of concrete	:	2300 kg/m^3
6.	Soil density (γ)	:	1800 kg/m^3
7.	Angle of internal friction(Φ)	:	30°
8.	Minimum cover of earth(assumed)	:	600 mm
9.	Cohesion for sandy soil(C)	:	0

Solution :

Horizontal thrust, F = $2\,p\,A\,Sin\dfrac{\alpha}{2}$

Cross sectional Area, A = $\pi/4 \times 50^2$

= $1963.5\ cm^2$

$\sin \alpha/2 = 0.258$

Horizontal thrust, F_1 = $2 \times 6 \times 1963.5 \times 0.258 \times 10^{-3}$

= 6.08 tons

Try a concrete block of size 1.2 m x 1.2 m x 1.2 m

(i) Lateral resistance to counteract the horizontal thrust :

Weight of thrust block	=	1.2 x 1.2 x 1.2 x 2.3	=	3.97 tons
Weight of water in pipe	=	$0.785 \times 0.5^2 \times 1 \times 1.2$	=	0.235 tons
Weight of earth	=	0.5 x 0.6 x 1.2 x 1.8	=	0.65 tons
		Total Weight	=	4.85 tons

Total force available considering frictional resistance of soil

= 4.85 x 0.3

= 1.45 tons

(ii) Lateral resistance of soil against the block:

$$f_p = \gamma_s.\frac{H^2}{2}.L.\left(\frac{1+\sin\phi}{1-\sin\phi}\right)+2CHL\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

= $1.8 \times 1.2^2/2 \times 1.2 \times 1.5 / 0.5$

= 4.67 tons

(iii) Lateral resistance of soil when the thrust block is free to yield away from the soil mass is neglected because of so small quantity.

Total lateral resistance	=	1.45 + 4.67	=	$6.12\ tons/m^2$
Total horizontal thrust	=	$6.08\ tons/m^2$		
Factor of safety	=	6.12 / 6.08		
	=	1.006 …..> 1.00 ………….. O.K.		

Hence, provide concrete thrust block size of 1.2m x 1.2 m x 1.2m.

4.5 AERATION

Aeration involves bringing air or other gases in contact with water to transfer volatile substances from the liquid to the gaseous phase and to dissolve beneficial gases into the water. The volatile substances that may be removed include dissolved gases, volatile organic compounds, and various aromatic compounds responsible for tastes and odors. Gases that may be dissolved into water include oxygen and carbon dioxide. The purposes of aeration in water treatment are:-

1. To reduce the concentration of taste and odor-causing substances, such hydrogen sulfide and various organic compounds, by volatilization or oxidation.
2. To oxidize iron and manganese, rendering them insoluble.
3. To dissolve a gas in the water (examples: addition of oxygen to groundwater and addition of carbon dioxide after softening).
4. To remove those compounds that may in some way interfere with or add to the cost of subsequent water treatment (examples: removal of hydrogen sulfide before chlorination and removal of carbon dioxide prior to softening).

4.5.1. Types of Aerators

Aerators are mainly two types depending upon the mechanics of aeration: (a) Those aerators in which water is exposed to the atmosphere, and (b) Those in which air is brought in contact with the water. Spray, multiple tray, cascade and mechanical aerators can be considered under type (a), while diffusion aerators fall under type (b).

Gravity Aerators

Gravity or water fall aerators are very common in water treatment plants. Gravity aerators utilize weirs, waterfalls, cascades, inclined planes with riffle plates, vertical towers with updraft air, perforated tray towers, or packed towers filled with contact media such as coke or stone. Various Types of Gravity Aerators are shown in Fig.4.12.

Cascade has the merit of being resistant to fouling or blockade with debris or calcium carbonate. It is particularly suitable for surface water and needs no fine screening before hand. Cascade waterfall aerator consists of steps or trays built in a structure with the following design parameters:

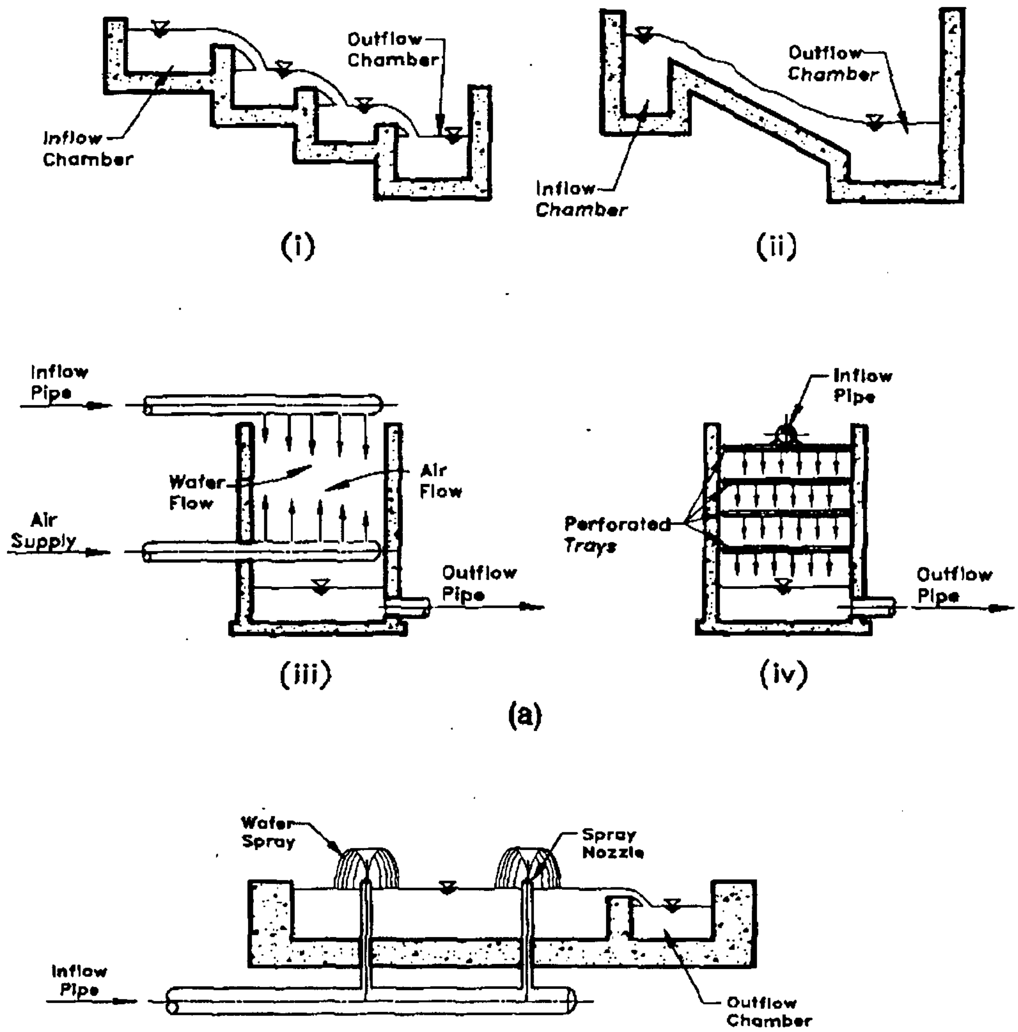
(1) Height of structure = 1.2 to 3 m

(2) No. of steps or trays = 4 to 6

(3) Area of aerator = 0.015 to 0.045 $m^2/m^3/h$

(4) Carbon dioxide removal = about 25 to 45 %

(5) Hydrogen sulphide removal= 20 to 35 %



(b)

Fig.4.12. Various Types of Gravity and Spray aerators. (a) Gravity aerators: (i) Cascade (ii) Inclined apron, (iii) Tower with counter current flow of air and water, (iv) Stack of perforated pans. (b). Spray aerator.

Spray Aerators

Spray aerators spray droplets of water into the air from moving or stationary orifice or nozzles. The water rises either vertically or at an angle and falls onto a collecting apron, a contact bed, or a collecting basin. Spray aerators are also designed as decorative fountains. To produce an atomizing jet, a large amount of power is required, and the water must be free from large solids. Losses from wind carryover and freezing in cold climates may cause serious problems. Spray aerators can remove 70 to 90 % of CO_2, 90 to 99% H_2S. However, they require large area and consequently difficult to be housed readily.

Diffused-Air Aerators

Water is aerated in large tanks. Compressed air is injected into the tank through porous diffuser plates, or tubes. Ascending air bubbles cause turbulence and provide opportunity for exchange of volatile materials between air bubbles and water. Aeration periods vary from 10 to 30 min. Air supply is generally 0.1 to 1 m^3 per min per m of the tank volume.

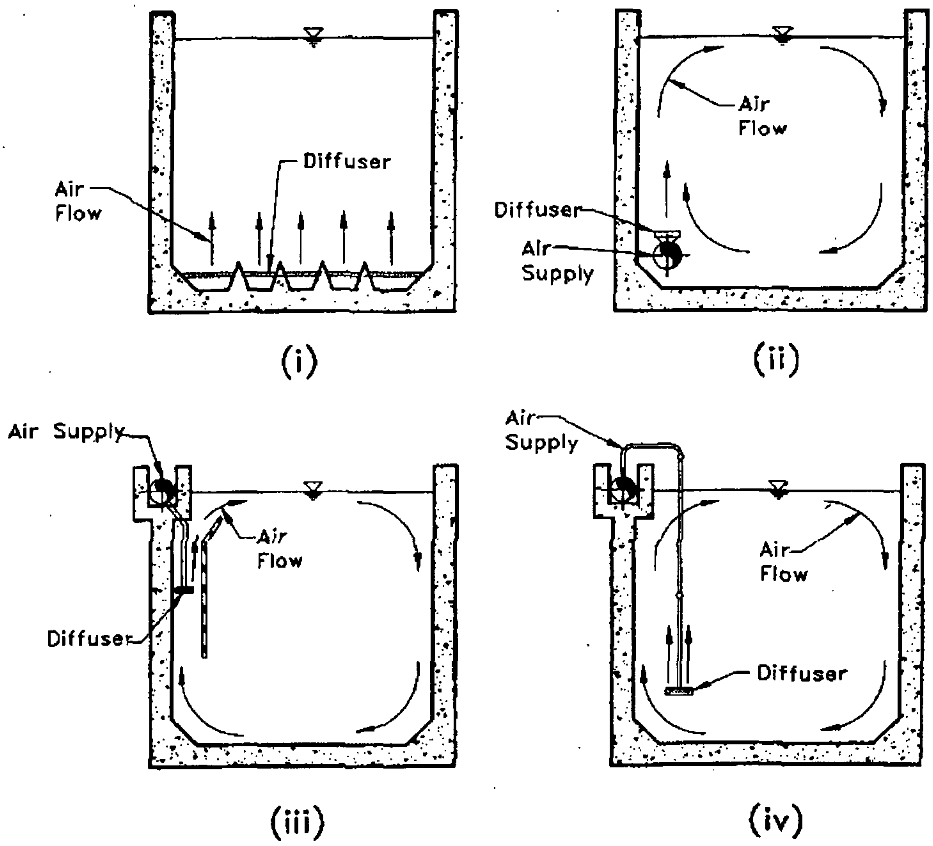


Fig.4.13.Various Type of Diffused Air Aerators (i) longitudinal furrows. (ii) Spiral low with bottom diffusers, (iii) Spiral flow with baffle and low depth diffusers, (iv) Swing diffusers.

Mechanical Aerators

Mechanical aerators employ either motor-driven impellers or a combination of impeller with air-injection devices. These are not normally used in water treatment because of the availability of more economical alternatives.

4.5.2 Design of Aerator for Almora Water Treatment Plant

For Kosi river water, which is not having excessive problem with iron and manganese, taste & odor, let us select the most simple and most commonly used Cascade Aerator. 2 nos. cascade aerator should be provided, one should be constructed initially for use in the initial stage of design period, and the other should be constructed later on when it is required.

Data :

Required discharge : 19.97 mld or 832.08 m^3/hr

Space requirement : 0.015 to 0.045 $m^3/m^2/hr$

Head requirement : 0.5 m to 3.0 m

No. of steps : 4 to 6

Design :

Considering average space requirement of 0.03 $m^3/m^2/hr$

Area required = 832.08 x 0.03

= 24.96 m^2

Provide 2 nos. of aerators.

Area of one aerator = 24.96/2

= 12.5 m2

Diameter, d = $\sqrt{(12.5 \times 4)/\pi}$

= 3.99 m

Provide diameter of 4 m at the bottom

No. of steps = 5

Provide height of each step= 0.4 m

$\therefore$ Ht. of aerator = 0.4 x 5

= 2 m …….. (0.5 to 3 m)….. …………………O.K.

Provide wide of each step = 0.35 m

Diameter of 1st step = 1.2 m

…… do …. 2nd step = 1.9 m

……. do….3rd step = 2.6 m

……..do…..4th step = 3.3 m

…….do…..5th step = 4.0 m

Designed figure of cascade aerator is shown in **Fig. 4.14.**

Channel Around Aerator (R.C.C Channel)

Discharge at one aerator = $832.08 / 2\ m^3/hr$

= $416.04\ m^3/hr$ or $0.116\ m^3/s$

Provide 0.75 m x 0.3 m + 0.3 m FB

Velocity check = 0.116 / (0.75 x 0.3)

= 0.51 m/s...... < 0.6........................(O.K)

Raw Water Channel :

Provide 750 mm wide x 300 mm deep + 0.3 m FB

Velocity = 0.116 / (0.75 x 0.3)

= 0.51 m/s...... < 0.6 .... (O.K)

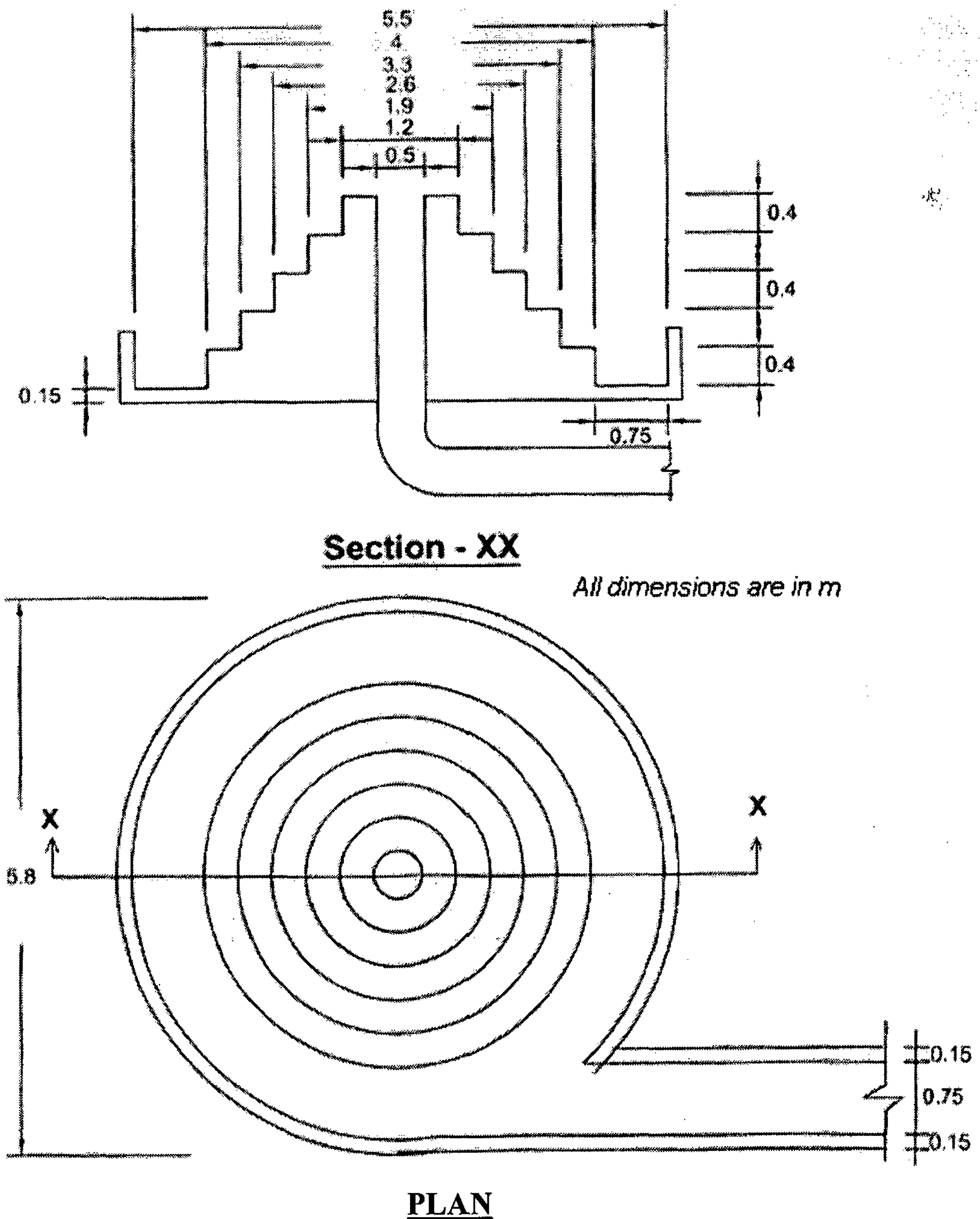


Fig. 4.14. Details of Cascade Aerator

4.6 COAGULATION AND FLOCCULATION

Water from a natural source usually contains many dissolved and suspended solids. Large suspended particles, such as sand, behave as '*discrete*' particles and can readily be removed by sedimentation and/or filtration processes. Suspended particles at the lower end of the size spectrum do not readily settle. These are colloidal particles and can be removed by sedimentation and filtration only after physical and chemical conditioning. Chemical conditioning of colloids is known as ***coagulation*** and involves the addition of chemicals called *coagulants* that modify the physical properties of colloids to enhance their removal. Physical conditioning is known as ***flocculation.*** This process involves gently mixing the suspension to accelerate interparticle contact, thus promoting agglomeration of colloidal particles into larger floc for enhanced settling.

The use of coagulants is generally necessary for clarifying raw waters containing turbidities greater than 30 to 50 mg/l, but in actual practice, plain sedimentation is rarely used these days, and the coagulation before sedimentation is almost universally adopted in all the major water treatment plants, and is followed by rapid sand filtration. [Garg,S.K,2005]. In water treatment plants, chemical coagulation is usually accomplished by the addition of trivalent metallic salts such as $Al_2(SO_4)_3$ (aluminium sulfate) or $FeCl_3$ (ferric chloride). Although the exact method by which coagulation is accomplished can not be determined, four mechanism are thought to occur. These include ionic layer compression, adsorption and charged neutralization, entrapment in a flocculent mass, and adsorption and interparticle bridging. Fig. 4.15 shows interparticle bridging with polymers

4.6.1 Enhanced Coagulation

In recent years, the coagulation process has also been broadly utilized to remove, not only turbidity, but also other undesirable organic and inorganic contaminants from the raw water [Qasim, et al.2005]. These objectives can be achieved by use of an *enhanced coagulation* process, in which an elevated coagulant dosage is usually required. Depending on the initial total organic carbon (TOC) and alkalinity in the raw water, a certain level of total organic carbon removal is required by the regulations. A higher coagulant dosage than that in a conventional process is recommended if a higher TOC removal is required. Most recently, the applications of enhanced coagulation are no longer limited to TOC removal, but also applied to many other contaminants Such as normal organic matters (NOMs), color, arsenic and other heavy metals from water.The only purpose of higher coagulant dosage in an enhanced coagulation process is simply to achieve sweep-floc coagulation. This is necessary to effectively remove the fine hydrous metal oxide crystals on which the contaminants have been successfully adsorbed.

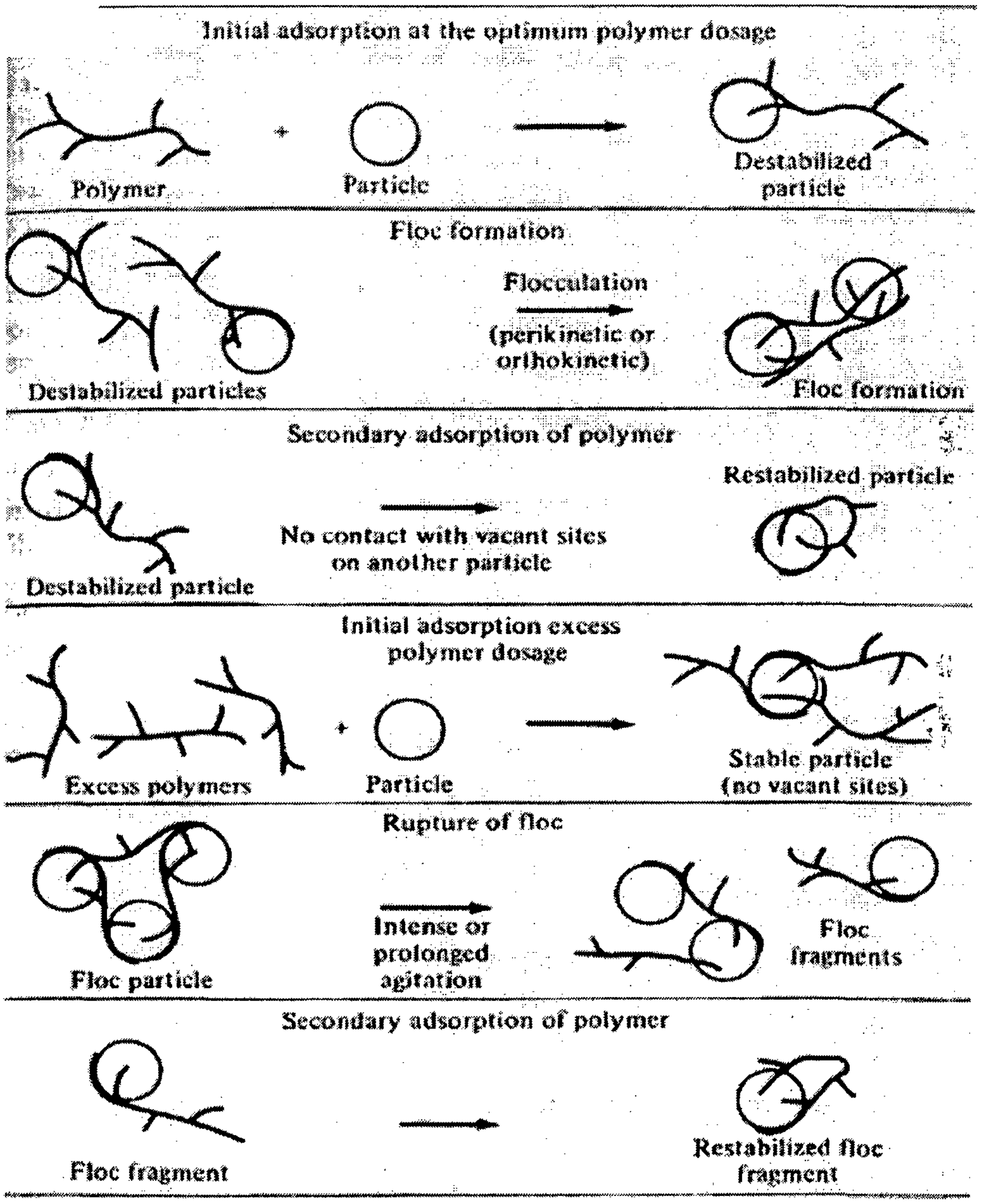


Fig.4.15. Interparticle bridging with polymers

4.6.2 Coagulants Used and Dosage

The common coagulants used in water works practice are salts of aluminum viz. filter alum, sodium aluminate and liquid alum and iron salts like ferrous sulphate, ferric chloride and ferric sulphate, etc. These chemicals are most effective when water is slightly alkaline. In the absence of such an alkalinity in raw water supplies, external alkalies like sodium carbonate, or lime, etc., are added to the water, so as to make it slightly alkaline.

Alum or filter alum has proved to be a very effective coagulant, and is being extensively used throughout the world. The effective pH range for its use is 6.5 to 8.5 [Garg, 1999], and may in many cases, require the addition of external alkali salts. The dose of alum may vary from 5 mg/L to 50 mg/L depending upon the turbidity and nature of the water [Peavy,S. et al,1985]. The average normal dose is about 17 mg/L [Garg, 2005].

Copperas ($FeSO_4.7H_2O$) is extensively used as a coagulant for raw waters that are not colored. It is generally cheaper than alum, and functions effectively in the pH value above 9.5 [GOI Manual,1999]. When chlorine is added to a solution of copperas, it gives ferric sulfate and ferric chloride, the resultant combination of which, known as chlorinated copperous is a valuable coagulant for removing colors, especially where the raw water has a low pH value. Ferric sulfate is quite effective in the pH range of 4 to 7 and above 9, whereas ferric chloride is quite effective in pH range of 3.5 to 6.5 and above 8.5. In conventional coagulation, Fe^{3+} dose is in the range of 2 to 4 mg/L; this may reach as high as 11mg/L in enhanced coagulation.

Jar Test for Optimum Coagulant

The best coagulant dose is determined first in the laboratory and then adjusted by actual observations at the treatment plant. In the jar test apparatus, the sample of raw water to be tested is placed in a number of jars having a capacity of about 1 litre. Normally, six jars are used and different amounts of paddles are rotated. The formation of the floc in each jar is noted. The amount of coagulant in the jar which produces a good floc with a least amount of coagulant, indicates the optimum dosage. The speed of paddles and the time of mixing may also be varied for different tests during determining this least optimum dosage.

4.6.3 Comparison of Alum and Iron Salts (as Coagulants)

- Iron salts produce heavy floc and can, therefore, remove much more suspended matter than alum.

- Iron salts can remove hydrogen sulphide and its corresponding tastes and odours from water.
- Iron salts can be used over a wider range of pH values.
- Iron salts cause staining and promote the growth of iron bacteria in the distribution system.
- Iron salts impart more corrosiveness to water than that which is imparted by alum.
- The handling and storing of iron salts require more skill and control, as they are corrosive and deliquescent. Whereas, no such skilled supervision is required for handling alum.

4.6.4 Coagulation Process Design Consideration

The design of coagulation process involves – selection of proper coagulant chemicals and their dosages, design of rapid mix and flocculation basin. However, the most important consideration in coagulant process design is to provide flexibility in chemical feed equipment and types of chemicals. Such flexibility provides the operator an ability to adjust proper dosages as the chemical make-up of raw water changes with time.

Rapid Mix

Coagulation process requires the addition of coagulants to the water stream. The success of this process depends on rapid and dispersion of the chemicals. The process of dispersing chemicals is known as *rapid mix* or *flash mix.* The coagulant is normally introduced at some point of high turbulence in the water. The source of power for rapid mixing to create the desired intense turbulence is gravitational, mechanical or pneumatic. The intensity of mixing is dependent upon the temporal mean velocity gradient (G).The turbulence and resultant intensity of mixing is based on the rate of power input to the water and G can be measured in terms of power input by the following expression:

$$G = \sqrt{P / \mu(vol)}$$

Where, G	=	temporal mean velocity gradient, s^{-1};
P	=	total input of power in water, watts;
μ	=	absolute viscosity of water, $N.s/m^2$; and
Vol	=	volume of water to which water is applied, m^3.

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds 300 m^3/hr., mechanical mixing also known as flash mixing, is desirable. Gravitational or hydraulic devices are simple but not flexible, while mechanical or pneumatic devices are flexible, but require external power.

(a) Gravitational Or Hydraulic Devices

In this device, the required turbulence is obtained from the flow of water under gravity or pressure. Some of the more common devices are described below.

Hydraulic jump Mixing

This is achieved by a combination of chute followed by a channel with or without sill. Standing wave flumes may also be used for this kind of mixing in which the hydraulic jump takes place at the throat of the flume. This device can be used as a standby in large plants to the mechanical mixers while for small plants, it can be used directly as the main unit.

Baffled Channel Mixing

In this method, the channel section is normally designed for a velocity of 0.6 m/s. The angle subtended by the baffle in the channel is between 40° to 90° with the channel wall. This angle ensure a minimum velocity of 1.5 m/s while negotiating the baffle.

(b) Mechanical Devices

The mechanically agitated mixing basins provide the best type of mixing. The chemical added to raw water is vigorously mixed and agitated by a flash mixer for its rapid dispersion in raw water; the water is then transferred to flocculation basin provided with a slow mixer. There are propeller type and impingement type mixers. In impingement type, water is forced as jet through a nozzle. Propeller type which is using baffles are more commonly employed

Design Consideration of Impeller Type Mixer:

For design criteria of mixer unit, detention time should be 30 to 60 sec. It should be capable of creating velocity gradients of 300 s^{-1} or more. Power requirements are ordinarily 1 to 3 watts per m^3/hr. of flow. Usually , the flash mixers are deep, circular or square tank. The usual ratio of impeller diameter to tank diameter is 0.2 to 0.4, and the shaft speed of propeller greater than 100 rpm imparting a tangential velocity greater than 3 m/s at the tip of the blade. The ratio of tank height to diameter is 1 : 1 to 3 : 1. A typical mixing basin provided with a flash mixer is shown in Fig. 4.16.

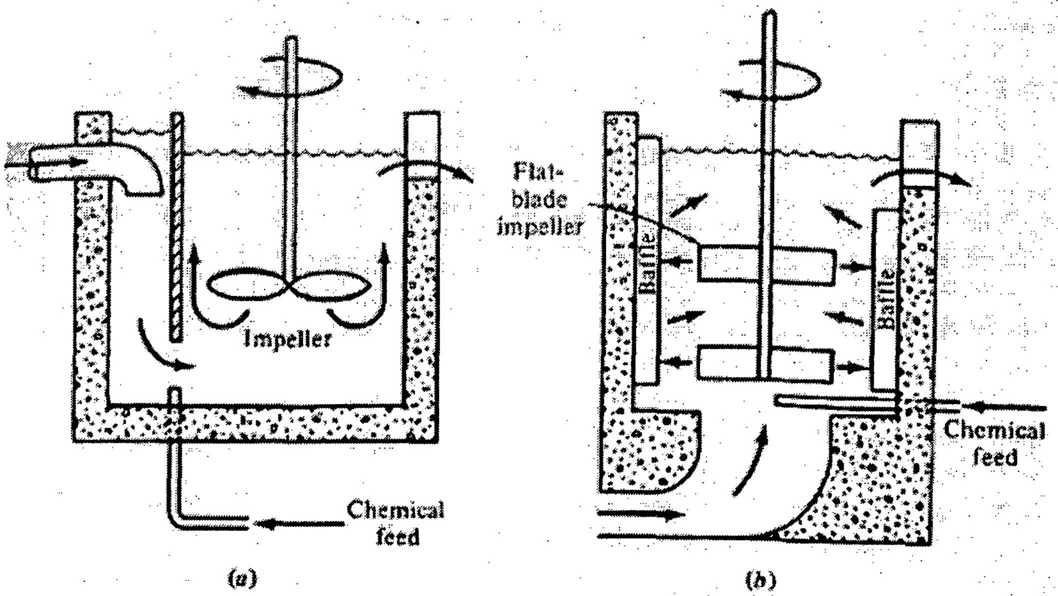


Fig.4.16. Typical Rapid Mixing Tanks (a) back-mix impeller, (b) flat-blade impeller

4.6.5 Design of Rapid Mix Unit for Almora Treatment Plant

Provide 2 numbers of flash mixer. Flash mixer with mechanical mixing with impellers is proposed. 4 baffles are provided.

Data :

Design flow	:	416.04 m^3/hr or 0.116 m^3/s
Detention time	:	30 secs. (30 to 60 secs)
Ratio of tank height to diameter	:	1.5 : 1 ....(1 – 3 : 1)
Ratio of impeller diameter to tank diameter	:	0.4 : 1 ....(0.2 – 0.4 :1)
Rotational speed of impeller	:	100 rpm

Solution :

Dimension of tank:

Volume	=	0.116 x 30	=	3.48 m^3
Diameter of the tank	=	$\Pi D^2/4 \times 1.5D$	=	3.48
∴ D	=	1.43 m		
Ht. of the tank	=	1.5 x 1.43	=	2.15 m

Provide 1.5 m dia. X 2.15 m depth + 0.2 m FB

Actual volume of tank = $\Pi \times 1.5^2/4 \times 2.15$

= $3.8\ m^3 > 3.48\ m^3$................. (O.K)

Power requirement :

Assume 2 $Watts/m^3/hr$ (1 – 3)

HP of motor = 2 x 0.116 x 60 x 60

= 835.2 watts

= 0.835 KW or 1.136 HP

Provide 1.5 HP of motor.

Impeller :

Diameter of impeller = 0.4 x tank diameter

= 0.4 x 1.5

= 0.6 m

Velocity of tip of impeller, Vt= $2 \Pi r n / 60$ m/s

$= \frac{2 \Pi \times 0.6 \times 100}{2 \times 60}$

= 3.14 m /s

Power spent = $\frac{1}{2} C_D . \rho . Ap . Vr^3$

Assuming C_D = 1.8 for flash blades

Vr = 0.75Vt = 2.35 m/s

∴ 835.2 = $\frac{1}{2} \times 1.8 \times 1000 \times Ap \times 2.35^3$

Area of blades of impeller,Ap = $0.071\ m^2$

In total, we provide 4 blades, with each having area $0.02\ m^2$ .Provide 4 blades of size 0.16 m x 0.12 m. Provide 4 nos. of baffles of length 2.25 m projecting 0.1 m from flash mixer wall to reduce vortex formation. Designed rapid mix unit is shown in Fig.4.17.

Pipe Line From Flash Mixer to Clariflocculator

Assume a velocity of 0.8 m/s

From equation, A x V = Q

i.e, $\Pi D^2/4 \times V$ = Q

∴ D = $\sqrt{(0.116 \times 4)/(\Pi \times 0.8)}$

= 0.43 m

Provide 500 mm ϕ M.S. inlet pipe to Clariflocculator from flash mixer with a by-pass of same diameter to filters. Pipes are to be provided inside with epoxy paint.

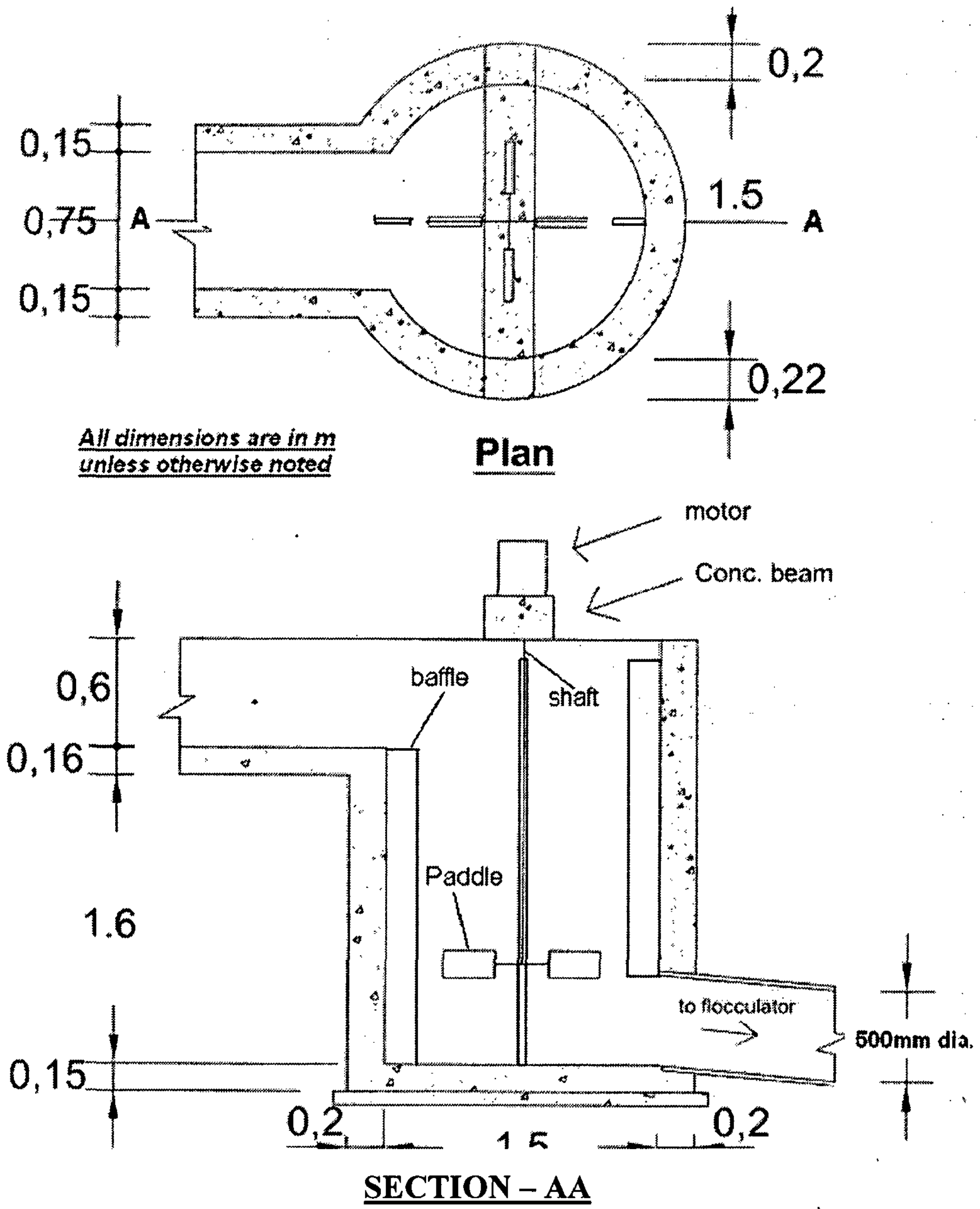


Fig.4.17. Details Of Rapid Mix Unit

4.6.6 Parshall Flume

Parshall flume is a type of standing wave flume widely used for accurate open flow measurement and control device. It is usually located at the entry of each process train, to provide the plant operator with flow data. However, its use requires application of different equations, based on the throat size. The approximate equation applicable for the entire range of its usage, namely, discharge varying from 0.001 m^3/s to 100 m^3/s is given below [GOI Manual,1999].

$$Q = 2.42\, W\, h^{2.58}$$

Where, Q = discharge in m^3/s

W = throat width in m

H = upstream gauge depth in m.

4.6.7 Design of Parshall Flume for Almora Treatment Plant

Parshall Flume may be located between aerator and rapid mix basin.Using equation of discharge through Parshall flume, i.e,

$$Q = 2.42\ W\ h^{2.5}$$

where, Q = discharge in m^3/s

W = throat width in m

H = upstream gauge depth in m.

Provide 300 mm throat width

$\therefore\quad h^{2.5} = 0.116 / (2.42 \times 0.3)$

$h = 0.48$ m

Provide 2.8 m long, 300 mm wide (at throat) and 0.5 m depth parshall flume.

Actual capacity= $2.42 \times 0.3 \times 0.5^{2.5}$

$= 0.13\ m^3/s \quad > 0.116$ .... (O.K).

4.6.8 Flocculation

Flocculation or slow mixing is the hydrodynamic process which results in the formation of large and readily settleable flocs by bringing the finely divide matter into contact with microflocs formed during rapid mixing. These can be subsequently removed in settling tanks and filters.

Types of Flocculators

Just like rapid mixers, flocculation units are often divided into two general groups; (1) hydraulic flocculators, and (2) mechanical flocculators. The hydraulic flocculators simply utilize cross flow baffles or 180^o turns to produce the required turbulence. Examples of such hydraulic flocculators used in practice are horizontal and vertical flow baffled flocculator, hydraulic jet action flocculator, Alabama type and tangential flow type flocculators. These types of flocculators are effective only if the flow rate is relatively constant.They are rarely used in medium and large sized water treatment plants, because of their sensivity to flow changes.

In mechanical flocculators, any type of mixers can be used (at a reduced speed) for flocculation. The mixers typically used in flocculation basins are horizontal and vertical-shaft paddle-wheel flocculator (Fig. 4.18). In addition to mixer types, other common flocculators are the walking-beam type and the oscillating type.

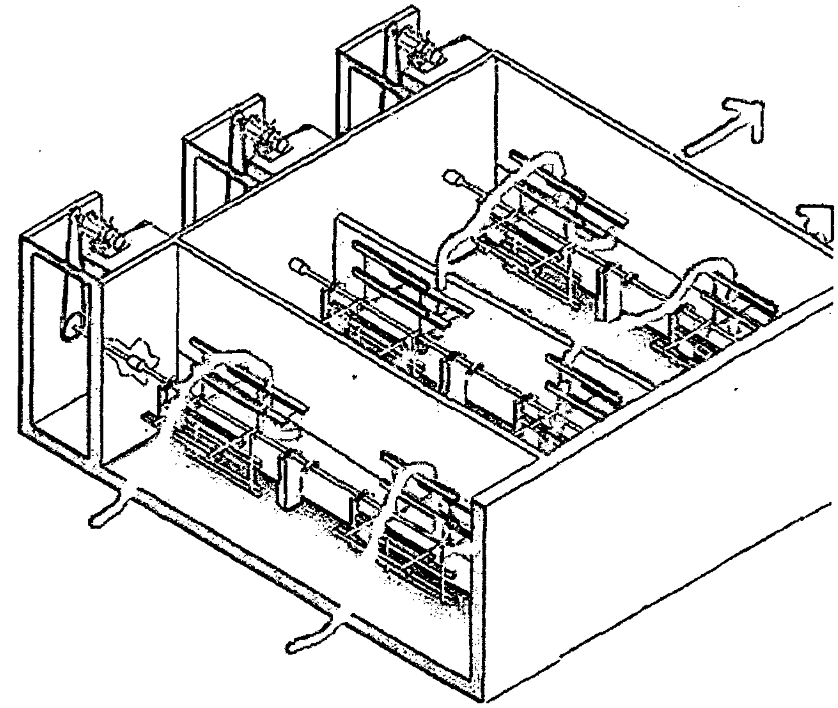


Fig.4.18. Typical Horizontal Shaft Paddle-Wheel Flocculator

Design Parameters of Flocculators

Slow mixing is meant to bring the particles to collide and then agglomerate. The rate at which flocculation proceeds depends on physical and chemical parameters such as charges on particles, exchange capacity, particle size and concentration, pH, water temperature, electrolyte concentration, time of flocculation, size of mixing basin and nature of mixing device.

The product G.t (G= velocity gradient, t= detention time) which is proportional to number of collisions of particles is a useful design parameter of flocculation. The desirable values of G vary from 20 to 75 s^{-1} and G.t from 2 to $6x10^4$ for aluminium coagulants and 1 to 1.5 X 10^5 for ferric coagulants. The usual detention time varies from 10 to 40 minutes normally 30 minute. Very high G values tend to shear flocs, whereas too low G value may not be able to provide sufficient agitation to ensure complete flocculation. Flocculation basins are normally designed with multiple mixing compartments in a series, with velocity gradients successively lower in each compartment. The value of G in that case is made to vary from 100 in the first stage, 50 or 60 in the second stage, and 20 s^{-1} in the third stage in the direction of flow.

The other design criteria are: depth of tank = 3 to 4.5 m; velocity of flow = 0.2 to 0.8 m/s normally 0.4 m/s; total area of paddles = 10 to 25 % of cross sectional area of the tank; range of peripheral velocity of blades = 0.2 to 0.6 m/s; power consumption = 10 to 36 kW/mld; outlet velocity to settling tank where water has to flow through pipe or channel = 0.15 to 0.25 m/s to prevent breaking or settling of flocs. For paddle flocculator, the velocity gradient is given by

$$G = \left[\frac{1}{2} \cdot \frac{C_D A_P \rho (V_P - V_W)}{\mu (vol)}\right]^{\frac{1}{2}}$$

Where,

C_D	=	co-efficient of drag (0.8 to 0.9)
A_P	=	area of paddle, m^2
Vol.	=	volume of water in flocculator, m^3
V_P	=	velocity of tip of paddle, m/s
V_W	=	velocity of water adjacent to the tip of paddle, m/s.

4.7 SEDIMENTATION

Sedimentation is a physical treatment process that utilizes gravity to separate suspended solids from water. Sedimentation (settling or clarification) is used to remove readily settling sediments such as gravel, grit, sand and silt, coagulated impurities such as color and turbidity and precipitated impurities such as hardness and iron. When suspended solids are separated from the water by the action of natural forces alone i.e. by gravitation with or without natural aggregation, the operation is called **plain sedimentation** and the process of settling is called ***discrete or Type 1 settling***. Plain sedimentation is usually employed as a preliminary process to reduce heavy sediment loads from highly turbid raw waters prior to subsequent treatment processes such as coagulation/filtration. Finely divided solids and colloidal particles, which cannot be removed by plain sedimentation within commonly used detention periods of few hours, are converted into settleable flocs by coagulation and flocculation and subsequently settled in sedimentation tanks. The process of settling during flocculation is known as ***flocculent or Type II settling.***

The factors that influence sedimentation are:

(a) Size, shape, density and nature (discrete or flocculent) of the particle;
(b) Viscosity, density and temperature of water;
(c) Surface overflow rate;
(d) Velocity of flow;
(e) Inlet and outlet arrangements;
(f) Detention period; and
(g) Effective depth of settling zone.

4.7.1 Ideal Sedimentation Tank

An ideal horizontal flow sedimentation basin exhibits the following characteristics: (1) the flow through the basin is evenly distributed across the cross section of the basin; (2) the particles are evenly dispersed in water; and (3) the settling of the particles is predominantly *Type 1*.An ideal sedimentation basin is divided into four distinct zones as shown in Fig. 4.19.

4.7.2 Types of Sedimentation Tanks

The tanks are categorized into horizontal or vertical flow tanks on the basis of direction of flow of water in the tank. The tanks may be rectangular, square or circular in plan.

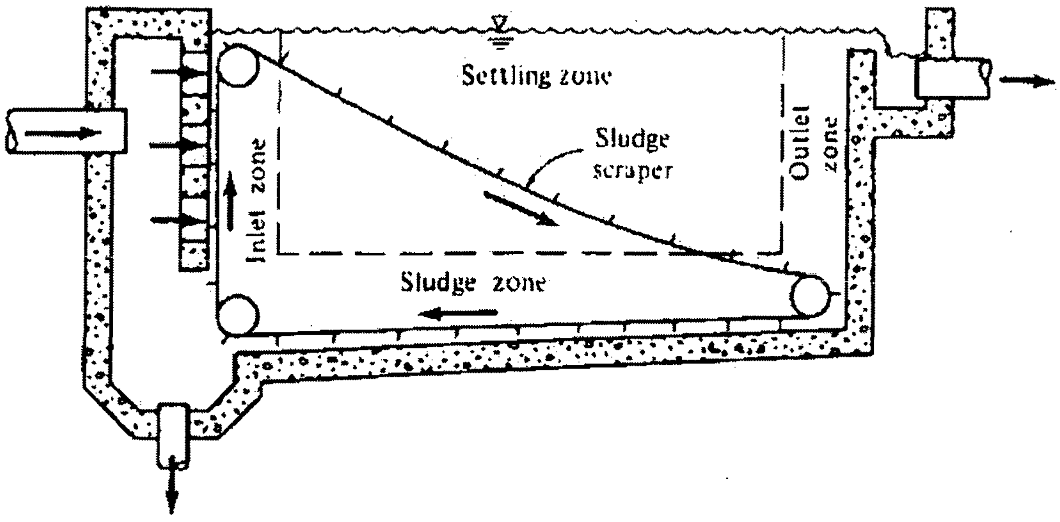


Fig.4.19. Ideal Sedimentation Basin of Rectangular Tank

(a) Horizontal Flow Tanks

In the design of horizontal flow tank, the aim is to achieve as nearly as possible the ideal conditions of equal velocity at all points lying on each vertical line in the settling zone. The horizontal flow tanks may be Radial flow circular tanks with central feed or with peripheral feed, Rectangular tanks with or without mechanical sludge cleaning device. Horizontal flow rectangular settling tank fitted with sludge scraper is shown in Fig.4.19.

In radial flow circular tanks with central feed, the water enters at the center of the tank and emanates from multiple ports of circular well in the center of tank to flow radially outwards in all directions equally. The aim is, to achieve uniform radial flow with decreasing horizontal velocity as the water flows towards the periphery and is withdrawn from the tank through effluent structure.

In radial flow circular tanks with peripheral feed, the water enters the tank from the periphery or the rim. Tracer studies have demonstrated that peripheral-feed sedimentation basin experience less short-circuiting than do center-feed basins, yet centre-feed basins are much more common. Generally, the flow pattern in circular basins is less nearly ideal than that in rectangular basins; however, circular sedimentation basins are more common, partly because they employ mechanically simple circular sludge collection equipment.

In rectangular tanks provided with mechanical scrapping devices, the sludge is continuously or periodically removed without stopping the operation of the tank, whereas operation requires to be stopped if the tank is not provided with scrapper. With properly designed inlet and outlet facilities, the *most nearly ideal flow pattern can be achieved in rectangular basins*. Rectangular basins also offer the advantage of lower construction cost for multiple units (as compared to circular basins) if common-wall construction can be employed.

Vertical Flow Tanks

Vertical flow tanks normally combine sedimentation with flocculation. These tanks are square or circular in plan and may have *hopper bottoms*. The influent enters at the bottom of the unit where flocculation takes place as particles co join into aggregates. The upflow velocity decreases with increased cross sectional area of the tank. There is a formation of blanket of floc through which the rising floc must pass. Because of this phenomenon, these tanks are also called as *upflow sludge blanket clarifier*. The clarified water is withdrawn through circumferential or central weir. These tanks have no moving parts and except for a few valves, require no mechanical equipment. They are compact units requiring less land area.

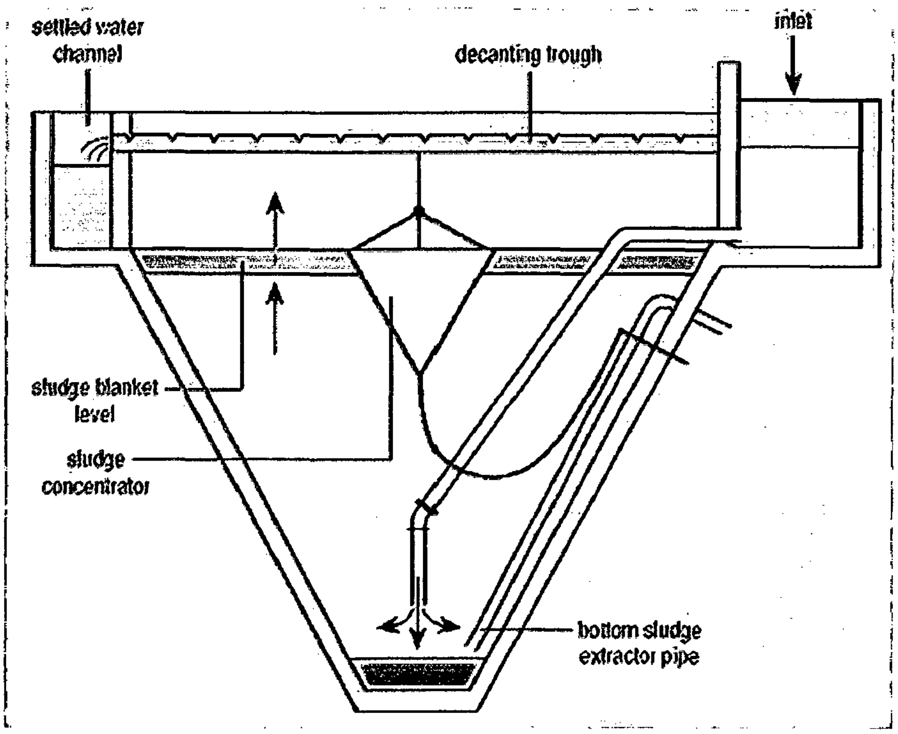


Fig.4.20. Upflow clarifier or Sludge blanket clarifier.

4.7.3 Clariflocculator

Clariflocculators have been used widely in water and waste water treatment. The coagulation and sedimentation processes are effectively incorporated in a single unit. All these units consist of 2 or 4 flocculating paddles placed equidistantly. These paddles rotate on their vertical axis. The clarification unit outside the flocculation compartment is served by inwardly raking rotating blades. The water mixed with chemicals is fed in the flocculation compartment fitted with paddles rotating at slow speeds. The flocculated water passes out from the bottom of the flocculation tank to the clarifying zone through a wide opening, the area of the opening being large enough to maintain a very low velocity. Under quiescent conditions in the annular settling zone the floc embedding the suspended particles settle to the bottom and the clear effluent overflows into the peripheral launder.

4.7.4 Sedimentation Basin Design

The important considerations in sedimentation basin design are basin geometry, surface loading rate, detention time, inlet and outlet zone, weir loading rates, and the sludge collection and removal system. Sedimentation basins often perform poorly in the field. The common reason is that actual detention time is significantly less than the designed detention time. This phenomenon is known as *short- circuiting*. The major causes of short-circuiting are currents induced by influent velocity, density and thermal gradients, wind action and the effluent weir. Even in well designed sedimentation basins, some short circuiting is expected.

(a) Dimension of Tank

There are three basic geometric shapes commonly used for sedimentation basins in water treatment: rectangular, circular and square. Rectangular basins typically are constructed with a length-to-width ratio ranging from 2:1 to 3:1. Monk has proposed that length-to-width ratios as high as 6:1 or 7:1 provide better control of short circuiting [Qasim et al,2000]. However, *GOI Manual recommended this ratio to be from about 3:1 to 5:1.* The diameter of the circular tank is governed by the structural requirement of the trusses that carry the scraping mechanism. Circular tanks up to 60 m are in use but are generally up to 30 m to reduce wind effects. Square tanks are generally smaller usually with sides up to 20 m. Square tanks with hopper bottoms having vertical flow have sides generally less than 10 m to avoid large depths. Square basins are typically of the center-feed type. Depths commonly used in practice vary from 2.5 to 5m with 3 m being a preferred value. Bottom slopes may range from 1% in rectangular tanks to about 8% in circular tanks. The slope of sludge hoppers range from 1.2:1 to 2:1 (vertical: horizontal).

(b) Surface Loading Rate

Surface loading rates are used to calculate surface area requirements of a sedimentation basin. The removal of particles of varying hydraulic subsidence values is solely a function of surface overflow rate also called 'surface loading' and is independent of the depth of the basin for discrete particle and unhindered settling. However, contact opportunities among the particles leading to aggregation increase with increasing depths for flocculent particles having tendency to agglomerate while settling, such as alum and iron flocs. It is recommended to develop design values of surface loading rate by the batch settling test procedure for a flocculent settling suspension prior to the design of major facilities. Typical design parameters (Table 5.7) obtained from GOI, Manual on water supply and treatment, 1999, can, however, be used in lieu of laboratory testing.

$$\upsilon_o = \frac{Q}{A} \qquad \text{...(4.7.4a)}$$

where, υ_o = surface overflow rate , $m^3/m^2.d$

Q = rate of flow, m^3/d

A = surface or plan area of settling basin, m^2

Table 4.10. Water Treatment Sedimentation Design Parameters

Tank type	**Surface loading $M^3/m^2/d$***		**Detention period hr.***		**Weir* loading rate $m^3/m.d$**
	Range	Typical value for design	Range	Typical value for Design	
Plain Sedimentation	up to 6000	15 – 30	0.01 – 15	3 – 4	up to 300
Horizontal flow, Circular	25 – 75	30 – 40	2 – 8	2 – 2.5	up to 300
Vertical flow (Upflow)Clarifier	-	40 – 50	-	1 – 1.5	170**

* *As per GOI Water treatment manual for average design flows*

** *As per Qasim et.al. Water Works Engineering*

(c) Detention Time

Detention time is used in conjunction with the surface loading rate to calculate the volume and side water depth of the sedimentation basin. For a known detention time, Eqn. 4.7.4b is used to determine the basin volume. Recommended detention time for different tanks is given in Table 5.7.

$$T = \frac{V}{Q} \quad \text{..(4.7.4b)}$$

where,

T = detention time, hr.

V = volume of the tank, m^3

Q = rate of flow, m^3/hr.

(d) Inlet or Influent Structure

The inlet zone evenly distributes the flow across the sedimentation basin and dissipates incoming velocity. Influent zones are designed differently for rectangular and circular basins. Typically, rectangular sedimentation basins are constructed to be integral with flocculation basins. A baffle or diffusion wall separates the two basins and serves as the sedimentation basin inlet The influent structure for the center-feed circular sedimentation basin typically consists of a metal skirt surrounding the inlet pipe. They are typically provided as part of the sludge collection equipment and generally are designed by equipment supplier. It is rare to use peripheral-feed circular sedimentation basin in water treatment.

(e) Outlet or Effluent Structure

Outlet zone or effluent structure has a great deal of influence on the flow pattern and settling behavior of floc in a sedimentation basin. Traditionally, overflow weirs or launder troughs have been used for outlet control in sedimentation basin. Either V-notch or submerged orifice weir plates are commonly used. Of these two, submerged orifice plates are preferred, because their use tend to result in less floc break-up between sedimentation and filtration units and also because they reduce ice problems in colder climates. V-notches, if used are generally placed 150 – 300 mm centre to centre. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent. The length of weir required is determined by the weir overflow rate or weir loading rate.

Effluent troughs act as lateral spillway. The widely used equation for the design of effluent trough is:

$$Y_1 = \sqrt{Y_2^2 + \frac{2(qLN)^2}{gb^2 Y_2}}$$

Where,

Y_1	=	water depth at upstream end of launder, m
Y_2	=	water depth in trough at a distance L from upstream End, m
Q	=	discharge per unit length of the weir, $m^3/s.m$
B	=	width of launder, m
N	=	no. of sides the weir receives the flow

In the above formula, channel friction is neglected and the drawdown curve is assumed parabolic. In the absence of any control device, it is reasonable and customary to assume critical flow at the lower end of the channel and hence, Y_2 at lower end of the channel of length L is:

$$Y_2 = \left[\frac{(qL^2)}{b^2 g}\right]^{1/3}$$

(f) Weir Loading

Weir length relative to surface area determines the strength of the outlet current. Normal weir loadings are up to 300 $m^3/d/m$. But when settling tanks are properly designed, well clarified waters can be obtained at weir loadings of even up to 1500 $m^3/d/m$.

(g) Sludge Removal

Sludge is normally removed under hydrostatic pressure through pipes. The size of the pipe will depend upon the flow and the quantity of suspended matter. For non mechanized units, pipe diameters of 200 mm or more are recommended. Pipe diameters of 100 to 200 mm are preferred for mechanized units with continuous removal of sludge with hydrostatic head. The following floor slopes may be given to ensure proper cleaning of sludge [GOI Manual,1999].

Circular tank with mechanical scrapper	:	*1 in 12*
Circular tank without scrapper	:	*1 in 10*
Non mechanized horizontal tank	:	*about 10% from side towards longitudinal centre line & longitudinal slope of atleast 5% from outlet end towards inlet.*

The power required for driving the scrapping mechanism in a circular tank is about 0.75 w/m^2, of tank area. The scrapping mechanism is rotated slowly to complete one revolution in about 30 to 40 minutes or preferably the tip velocity of the scrapper should be around 0.3 m/min. or below.

4.7.5 Pre-Sedimentation and Storage

The turbidity of raw water from rivers and streams may exhibit wide fluctuations and values exceeding a few thousand NTU are not uncommon during high flow season. In such a case, removal of large sized and rapidly settleable silt and other materials can be accomplished by pre-sedimentation and storage before raw water reaches the treatment plant. When removal of coarse and rapidly settling silt is aimed at in pre-sedimentation, lower detention periods of 0.5 to 3 hours and higher surface loading of 20 to 80 m^3/m^2/d may be used.

4.7.6 Design of Pre-Sedimentation Tank for Almora Treatment Plant

The turbidity of kosi river reaches as high as 1500 NTU during high flow season. Hence, it is highly recommended to propose pre-sedimentation tank to remove the coarse and rapidly settleable sediments.

Let us design the tank to remove particles up to 0.1 mm size, and adopt the following general parameters.

Design parameters:

Overflow rate	=	20 - 80 m3/m2/d or up to 36 m3/m2/d as per Cox
Minimum side water depth	=	2.5 m
Detention time	=	0.5 – 3.0 hrs

Side slopes = 10 % from side towards center line (for non mechanical cleaning)

Longitudinal slope = 1 % in rectangular tank

Ratio of length and breadth = 3 : 1 to 5 : 1

Settling velocity = to ensure removal of minimum size of A particle of 0.1mm

Hydraulic Design :

(1)Dimension of tank **:**

Design discharge : 20377 m^3/day

Pumping is to be done 23 hrs a day, hence, discharge coming into the pre-sedimentation tank : 20377 x 24/23 = 21262.95 m^3/d or 885.95 m^3/hr.

Let us provide 2 sedimentation tanks of equal size, therefore, design discharge for each tank will be 885.95/2 = 442.975 m^3/hr

Assume detention period = 1 hr.

Effective storage of sedimentation tank = 442.975 x 1.0 = 442.975 m^3

Assume effective depth = 3 m

Area of tank required = 442.975 / 3 = 147.658 m^2

Assume L : B = 3 : 1

3B x B = 147.658

$\therefore$ B = 7.015 m, and L = 21.05 m

Provide a free board of 0.3m and 0.2 m for sludge storage

Hence, provide **2** tanks of each having size of **21 m x 7.5 m x 3.5m** depth

Loading on the tank/ overflow rate= Q/A

= 442.975 x 24 /(21 x 7.5)

= 67.5 m^3/m^2/day ....(O.K)

(ii) Settlement velocity (v_s) by method of Stoke's Law :-

Size of particle (d) = 0.1 mm

Sp. Gr. (Ss) = 2.65

Kinematic viscosity at 20°C, ν = 1.01 x 10^{-6} m^2/sec

Value of g = 9.81 m/s^2

$\therefore$ v_s = $g\,(Ss - 1)\, d^2/(18 . \nu)$

= $9.81\,(2.65-1)\,.(0.1 \times 10^{-3})^2/\,(18 \times 1.01 \times 10^{-6})$

= 0.0089 m/s = 8.9×10^{-3} m/s

Reynolds Number, R = $v_s\, d/\, v$

= $8.9 \times 10^{-3} \times 0.1 \times 10^{-3}/(1.01 \times 10^{-6})$

= $0.88 < 1$

Hence, the flow is laminar and Stoke's law is applicable.

(iii) Settlement Velocity (v_s) by the method of discharge entering the tank and tank dimensions :

Discharge entering the basin (Q) = 442.975 m^3/h

= 0.123 m^3/s

Horizontal flow velocity, v = Q/(B x H)

= 0.123/(7.5 x 3)

= 0.0055 m/s or 0.33 m/min.

∴ Setlling velocity, v_s = (v x H)/L

= 0.33 x 3/21 = 0.047 m/min

= 7.85×10^{-4} m/s

Thus, particles having settling velocity greater than or equal to 7.85×10^{-4}m/s shall settle in the tanks of dimensions assumed above.

(iv) Time period required for particles to settle in the tank :

Time required to settle down the water depth by the particle

T_1 = $3/(8.9 \times 10^{-3})$ secs

= 337.078 secs = 0.0936 hrs.

Since the horizontal velocity of flow is 0.0055 m/s and the length of the tank is 31 m, the time taken by water to reach the outlet is given as,

T_2 = 21/0.0055 = 3818 secs.

= 1.06 hrs.

Thus, time required by the water to flow from inlet to outlet is greater than time required by the particles to size 0.1 mm to settle down.

Hence, all particles of size 0.1 mm having a settling velocity greater than or equal to 8.9×10^{-3} m/s shall be retained in the sedimentation tank.

Note : Longitudinal slope of 1 in 50 is provided, and side slope of 1 in 30 is provided towards longitudinal central line.

Actual detention time in sedimentation tank = (21 x 7.5 x 3)/442.975

= 1.06 hrs .... (O.K)

(v) Check against scour of deposited particles:

Scour velocity, v_d = $\sqrt{(8\beta g(Ss-1)d/f}$

Where, β = 0.04 (for unigranular particle)

f = 0.03 (Darcy-Weisbach friction factor)

Ss= sp. Gr. Of particles = 2.65

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

d = size of settling particles = 0.1 mm or 0.1 x 10^{-3} m

$\therefore$ v_d = $\sqrt{(8 \times 0.04 \times 9.81 \times (2.65-1) \times 0.1 \times 10^{-3})/0.03}$

= 0.13 m/sec.

Thus to avoid scour, the flow velocity should not exceed v_d.

Here, horizontal flow (v) is 0.0055 m/s < v_d ....(O.K.)

(vi) Influent Structure :

Provide 0.7 m wide and 0.6 m deep influent channel that runs across the width of the tank. Provide 4 submerged orifices 0.2 m x 0.2 m in the inside wall of influent channel to distribute the flow uniformly into the basin. A baffle of 1 m deep is provided at a distant of 1 m away from the orifices to reduce turbulence.

Velocity of flow in channel, v = Q/A

(assuming a depth of 0.4 m flow) = 442.975 /(0.7 x 0.4 x 3600)

= 0.43 m/s ......< 0.45 m/s..........(O.K)

***(vii) Effluent Structure*:**

Outflow from sedimentation tank = 442.975 m^3/hr – 2 % lost in de-sludging

= 434.11 m^3/hr

Assuming a weir loading of 250 $m^3/d/m$ length of weir (< 300, O.K)

$\therefore$ Weir length = 434.11 x 24/250

= 41.67 m

Provide 8 nos. of 0.3 m wide effluent launder troughs, and a 0.4 m wide center collection channel that conveys the settled water into a final effluent box. A baffle board is provided between the settling and outlet zones.

Actual length of weir = 3.3 m x 14 = 46.2 m > 41.67............(O.K)

Details of pre-sedimentation tank is shown in Fig.4.21.

4.7.7 Design of Raw Water Storage Tank/Sump

The pre-sedimentated water is to be pumped to treatment plant for pumping of 23 hrs. a day. Hence, capacity of storage tank with ½ hour detention time is sufficient to be provided for un-interrupted supply of raw water to treatment plant.

Volume of raw water tank = 885.95 x ½

= 442.97 m^3

Assume 3 m as effective depth,

Area required = 442.97 / 3

= 147.65 m^2

Provide square tank of dimension 12.2m x 12.2 m x 3.3 m depth in 2 compartments for the purpose of periodical cleaning, inspection and repairs.

Details of raw water storage tank is shown in Fig.4.22.

4.7.8 Design of Flocculation-Sedimentation Tank

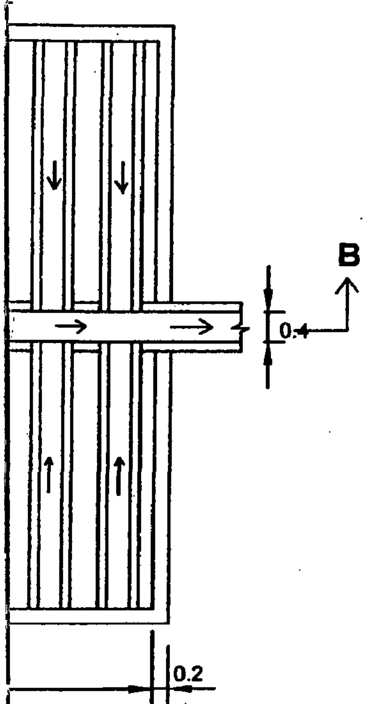
Let us select **clariflocculator**, because clariflocculators effectively combine coagulation and sedimentation processes in a single unit, hence proves to be compact and space saving. Moreover, clariflocculators have been widely and successfully used for treating river water having almost the same quality as that of Kosi river. Let us select circular shaped unit which is even more compacted than rectangular shaped clariflocculator. Let us provide 2 nos.

Data and parameters:

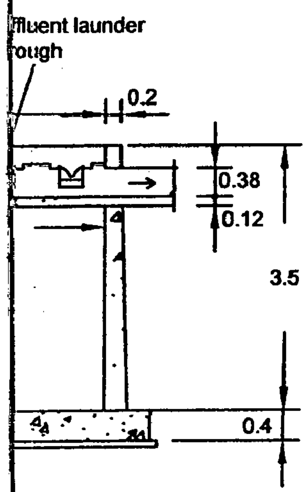
1.Desired average outflow from clariflocculator = 19 mld + 3 % lost in filtration

= 19 x 1.03 = 19.57 mld

2. Water lost in desludging from clariflocculator = 2 %



ensions are in m



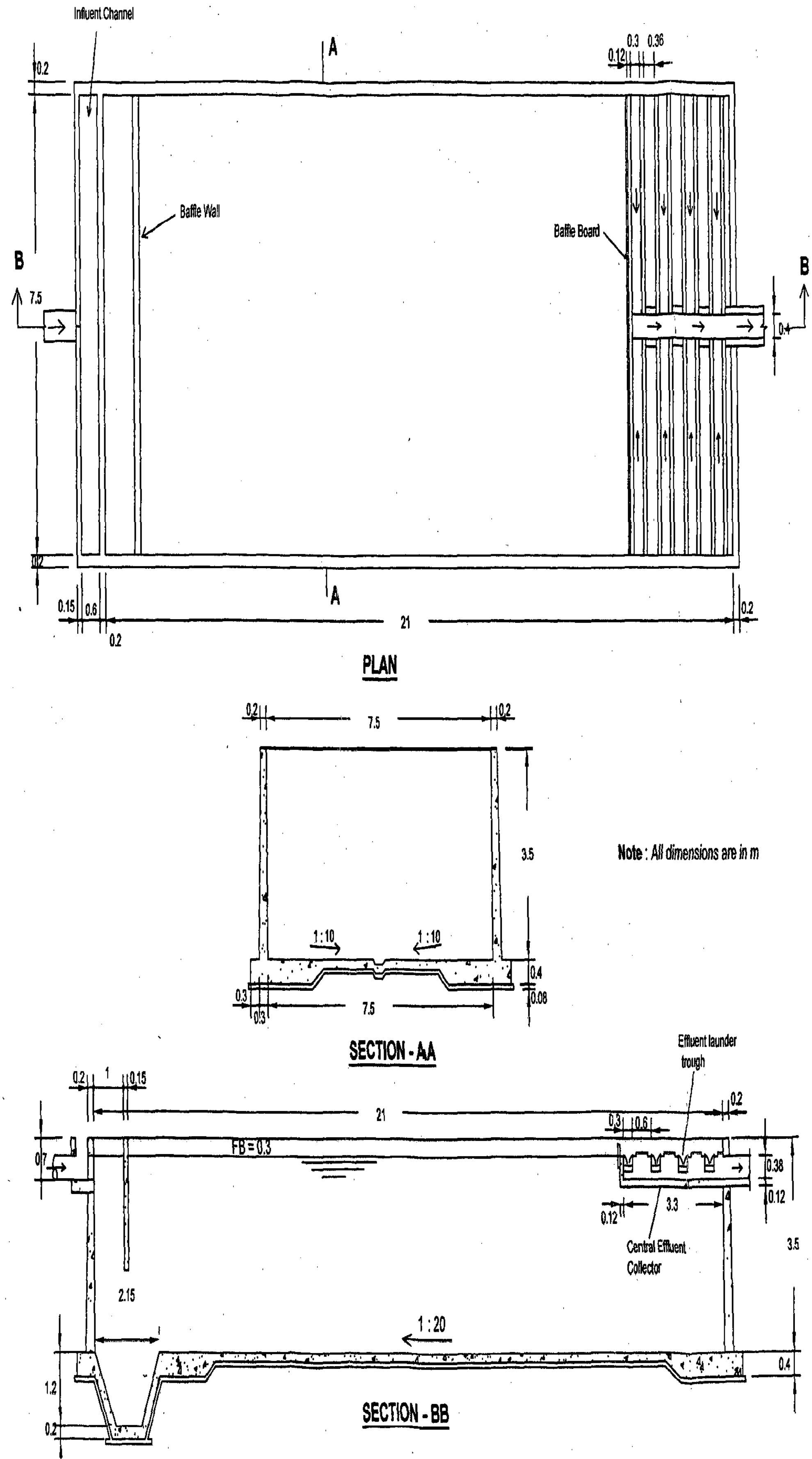


Fig.4.21. DETAILS OF SEDIMENTATION TANK

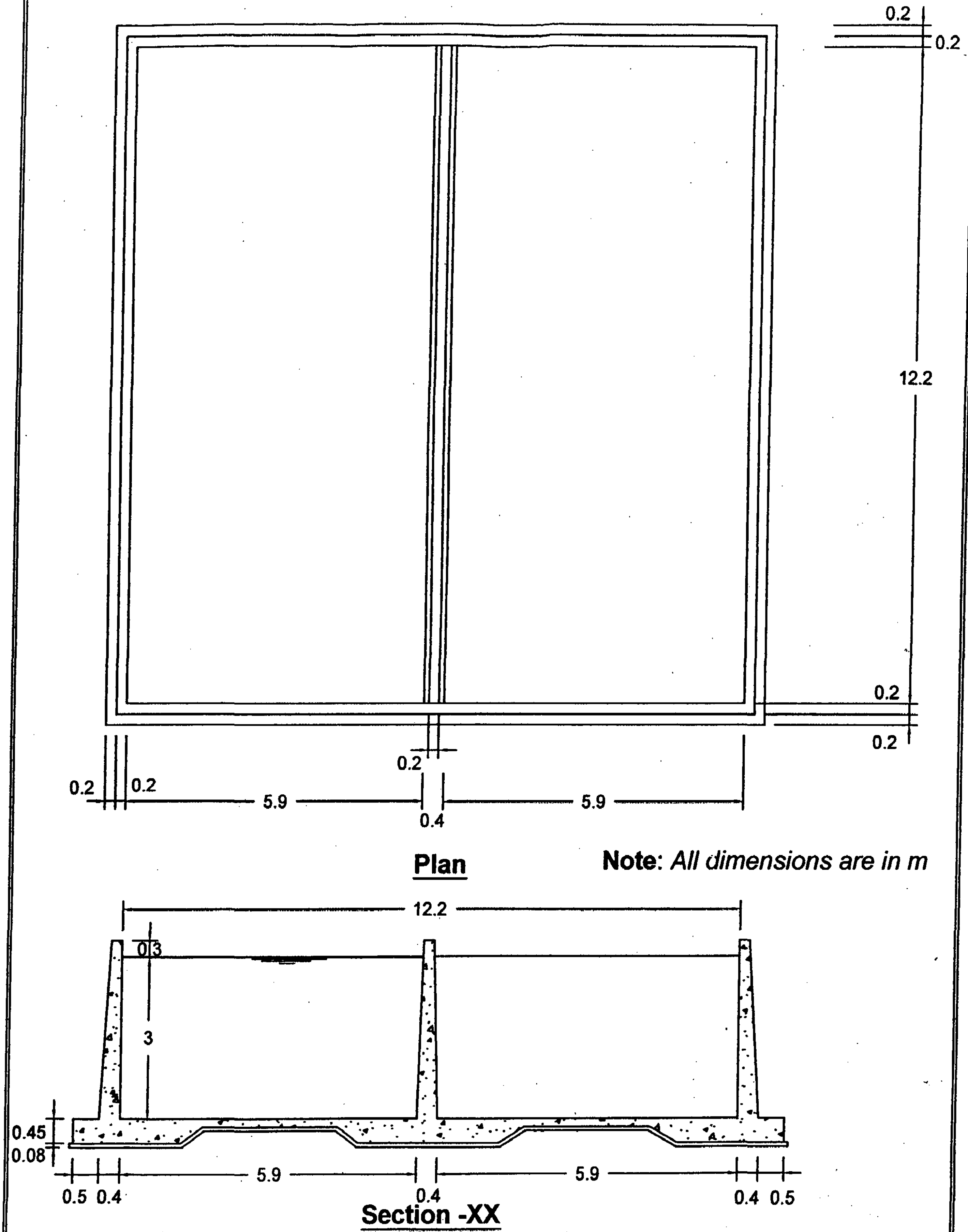


Fig.4.22 Details of Raw Water Storage Tank

3. Design average outflow = 19.57 / 0.98
= 19.97 mld
4. Design average outflow for 1 no = 19.97 /2 = 9.98 mld
= 416.0 m^3/hr
= 0.1156 m^3 /s
5. Detention period = 20 mins. ..... (10 to 40 mins)
6. Average value of vel.gradient , G = $40S^{-1}$ .... (20 to $75S^{-1}$)

Flocculator :

Dimensionless parameter, G.t = 40 x 20 x 60
= 4.8×10^4 within 2 to 6×10^4 for alum Coagulants.......... ....(O.K)

Volume of flocculator = 416.0x 20/60
= 138.67 m^3

Provide a water depth of 3.5 m (within 3 to 4.5 m)..............................(O.K)

Plan area of flocculator = 138.67/3.5
= 39.62 m^2

Let D = diameter of flocculator
Dp = diameter of inlet pipe (500 mm)

Then, $\pi/4\,(D^2 - Dp^2) = 39.62$

$\therefore$ D = 7.12 m

Provide a tank diameter of 7.2 m

Dimensions of Paddles:

Total power input to flocculator, $P = G^2 \mu$ (vol.)

i.e, $P = 40^2 \times (0.89 \times 10^{-3}) \times (\pi \times 7.2^2/4 \times 3.5)$
= 202.92 watts , say 203 watts.

Power input = $\frac{1}{2} . C_D . \rho . Ap\,(V - v)^3$

Where,

C_D = Newtons co-efficient of drag, 1.8 (0.8 to 1.9)
ρ = density of water at 25° C , 997 kg/m^3
V = velocity of tip of blades
= 0.4 m/s (0.3 to 0.4 m/s)
v = velocity of water at the tip of blades
= 0.25 x 0.4 (25% of V) = 0.1 m/s

∴	203	=	$\frac{1}{2} \cdot 1.8 \cdot 997 \cdot Ap \cdot (0.4 - 0.1)^3$
i.e,	Ap	=	8.38 m2

Ratio of area of paddle to cross sectional area of flocculator

= $Ap / \{\pi (D - Dp) h\}$

= $8.38 / \{\pi(7.2 - 0.5) x\ 3.5\}$

= 0.114 or 11.4 %...........within 10 to 25 %......... ..(O.K)

Assuming 4 paddles

Area of each paddle = 8.38 / 4

= 2.09 m^2

Assuming 3 m height of paddle

Width = 2.09 / 3

= 0.69 m

Provide 4 nos of paddles of 3 m height and 0.7 m width.

*Two shafts will support 4 paddles, each shaft supporting 2 paddles. The shaft will be at a distance of (7.2 – 0.5)/2 = 3.35 m from center line of clariflocculator. The paddles will rotate at the speed of 4 rpm.

Distance of paddle edge, r from centerline of vertical shaft is given by the equation.

	V	=	$(2\pi r n) / 60$
Where,	r	=	radius
	n	=	no. of revolution per minute
	V	=	velocity of tip of blade
i.e,	0.4	=	$(2\pi r 4) / 60$
∴	r	=	0.95 , say 1.0 m

Clarifier :

Assume a surface overflow rate of 40 $m^3/m^2/day$ ....(30 to 40 $m^3/m^2/day$)

Surface area of clarifier = 9.98 mld x 10^3 / 40

= 250 m^2

Diameter of clariflocculator, D_{cf} is given by

	$\pi/4\ [D^2_{cf} - 7.2^2]$	=	250
∴	D_{cf}	=	19.24 m , Say 19.25 m
	Length of weir	=	πD_{cf}
		=	π x 19.25
		=	60.47 m

Weir loading = 416.00 x 24 / 60.47

= 165.1 m^3/m/day <300 m^3/m/day............(O.K).

Assume detention period, t = 2 hrs (Ref. CPHEEO manual , pg.227)

Depth of tank = (416.0 x 2)/250

= 3.3 m , Say 3.4 m

Total depth at centre of the tank = 3.4 + 25% for sludge (0.85m) + (19.25-3)/2 x 1/12

= 4.93 m , say 5 m.

Launder/Collecting Channel (R.C.C):

Flow for one unit = 19.57 mld / 2 or 0.1133 m^3/s

Assuming velocity of flow in the channel as 0.3 m/sec. (maximum permissible being 0.4 m/s). Half of the portion of circular channel (launder) will collect half of this quantity.

Flow in half portion of channel = 0.1133/2

= 0.056 m^3/s

Assuming width of channel to be 0.7 m ,

Depth of water in the channel = 0.056/(0.3 x 0.7)

= 0.27 m

Providing a clear fall of 0.2 m in the channel around the clarifier, then

Total depth of channel = 0.27 + 0.2

= 0.47 say 0.5 m

Slope of the channel is such that it gives a velocity of 0.3 m/s;

For open channel ,

V = $1/0.013 \times R^{2/3} \times S^{1/2}$

Where , V = velocity (0.3 m/s)

R = hydraulic radius

= 0.189/1.24 = 0.15

∴ Slope, S = $(0.3^2 \times 0.013^2) / 0.15^{4/3}$

= 0.00019

Or 0.19 m per 1000 m

Or 1 in 5263

All mechanical scraper arrangements will be provided. Sludge collection pit of 3.0 m diameter and 0.5 m depth with 300 mm pipe and valve for sludge lead off are provided. Details of clariflocculator are shown in Fig.4.23.

4.7.9 Design of Chemical House

Let us select alum as coagulant, and design for store room for 3 months storage .

Dosage of alum = 50 mg/lt. (say)....(15 to 85 mg/lt, Garg/483)

Alum required per day = $50 \times 19.97 \times 10^6 / 10^6$

= 1000 kg

For 90 days (3 months) requirements

= 1000 x 90

= 90000 kg

= 90 M.Tons

Density of Alum is 1230 kg/m3

Volume of Alum required = 90000 / 1230

= 70.17 m^3

Assuming storage of 2 m height

Area of storage required = 73.17 / 2

= 36.6 m^2

With 50 % moving space

Total area required = 54.9 m^2

Provide **5m x 11m area** chemical house.

Alum storage in ground floor & solution tank in first floor.

Alum Solution Tanks :

Alum solution tank shall be housed at first floor of chemical house so that gravity feeding is possible.

Alum dosage : 50 mg/lt.

Daily requirement : 1000 kgs

At 5 % strength solution ....(Ref.GOI Manual ,194)

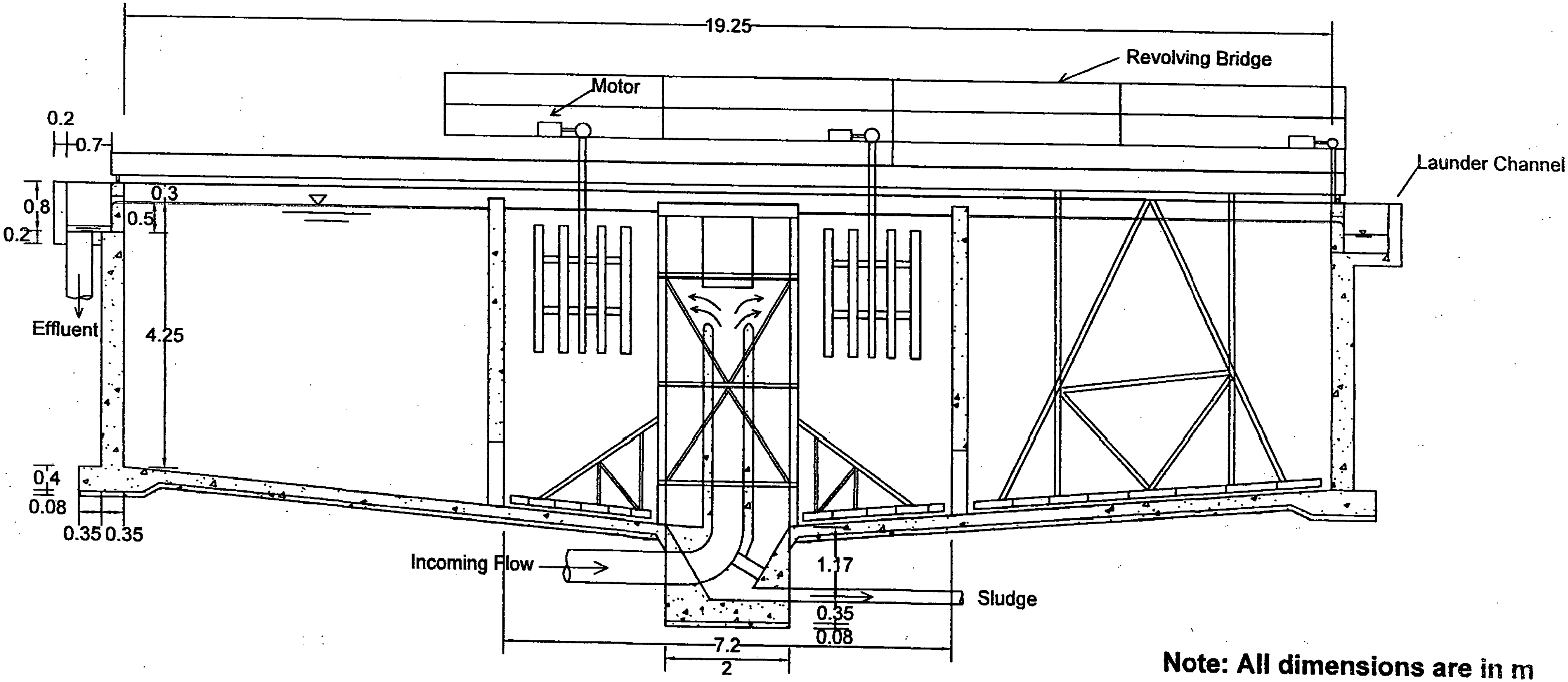
Capacity of tank for 8 hrs storage

= 100/5 x 1000 x 8/24

= 6666.67 lts.or 6.67 m^3

Provide 3 tanks of 2.6m x 2.6m x 1m + 0.3 m FB.

The solution tanks will be constructed in the FF of chemical house.



Note: All dimensions are in m

Fig.4.23. CLARIFLOCCULATOR WITH SLUDGE REMOVAL MECHANISM

Pipe Line From Flash Mixer to Clariflocculator

Assume a velocity of 0.8 m/s

From equation, $A \times V = Q$

i.e, $\Pi D^2/4 \times V = Q$

$\therefore$ $D = \sqrt{(0.116 \times 4)/(\Pi \times 0.8)}$

$= 0.43$ m

Provide 500 mm ϕ M.S. inlet pipe to Clariflocculator from flash mixer with a by- pass of same diameter to filters. Pipes are to be provided inside with epoxy paint.

4.8 FILTRATION

Filtration is a process for separating suspended and colloidal impurities from water by passage through a porous medium or porous media. Filtration, with or without pre-treatment, has been employed for treatment of water to effectively remove turbidity, color, microorganisms, precipitated hardness from chemically softened waters. Removal of turbidity is essential not only from the requirement of aesthetic acceptability but also for efficient disinfection which is difficult in the presence of suspended and colloidal impurities that serve as hideouts for the microorganisms.

Various types of filters which are in use are slow sand filter, rapid sand filter, pressure filter and upflow filter. Slow sand filter and rapid gravity filter are mostly used in municipal water supplies, though slow sand filters are mainly convenient for small systems and where large land is available. Pressure filters are used only on small purification plants due to high cost, and upflow type filters are rarely used in granular filters, but it is sometimes used in granular activated carbon bed.

4.8.1 Comparison Between Slow and Rapid Sand Filter

Comparisons between slow sand filter and rapid gravity filter are given in the following table.

Table 4.11 Comparison of Slow Sand and Rapid Gravity filters

S/N	Item	Slow Sand filter	Rapid Gravity filter
1	Pre-treatment requirements	Effluents either from plain sedimentation tanks or raw water without any treatment are generally fed into them;	Coagulation, flocculation and sedimentation is a must.

		coagulation is not at all required.	
2	Base material	Gravel base supports the sand. It varies from 3 to 65 mm in size and 0.2 to 0.3 m in depth.	Gravel base supports sand and also distributes the wash water uniformly on the surface of sand. It varies from 3 to 50 mm in size and 40 to 60 cm in depth
3	Filter sand	Effective size (D_{10}) is 0.25 - 0.35 mm and uniformity coefficient $\left(\frac{D_{60}}{D_{10}}\right)$ is 3 to 5. The grain size distribution is generally uniform throughout the depth of filter media. For best efficiency, filter bed should not be less than 0.4 to 0.5 m thick.	Effective size is 0.45 – 0.7 mm And uniformity coefficient 1.3 – 1.7. The sand is laid in layers with smallest grain size at top and coarsest at bottom. Usually, sand layer depth is 0.6 to 0.75 m.
4	Under drainage system	Laid in order to receive filtered water. Open jointed pipes or drains covered with blocks may be used	Laid in order to receive filtered water and also to pass water for backwashing at high rate. Perforated pipe laterals discharging into mains or diffuser plate bottom may be used.
5	Size of each unit	Large, area varying from 100 to 2000 sg.m or more	Small, 10 to 80 sq.m.
6	Rate of filtration	Small, such as 100 – 200 lt./hr. per sq. m of filter area	Large, such as 4800 – 6000 lt/hr. per sq.m of filter area.
7	Economy	High initial cost of both land and material, low cost of operation and maintenance	Low initial cost, higher cost of operation and maintenance.
8	Efficiency	Very efficiency in removing bacteria (98 to 99%) but less efficient in removing color. They cannot handle turbid waters of more than 20 NTU	Less efficient in removing bacteria (80 to 90 %) but very efficient in removing color. Can handle very turbid water with coagulation-sedimentation.

9	Flexibility	Not flexible for meeting variations in demand	Quite flexible for meeting seasonal variations in demand
10	Suitability and adaptability	For smaller village supplies or for individual industrial supplies	Widely and almost universally adopted for public supplies, especially at major cities and towns.
11	Post treatment required, if any	Almost pure water obtained. May be disinfected slightly to make it completely safe.	Disinfection a must, some other treatments may be given if needed.
12	Ease in construction	Simple	Complicated, as under drainage is to be properly designed.
13	Skilled supervision	Not required	Essential
14	Loss of head	Approximately 10 cm is initial loss, 0.8 – 1.2 m is final limit when cleaning is required	Approximately 0.3 m is initial loss, 1.8 to 2.0 m is final limit when cleaning is required.
15	Method of cleaning	(a) Scrapping and removing the top 2 to 3 cm thick layer and washing down by hoses. (b) laborious method	(a) agitating the sand grains and back washing with or without compressed air
16	Qty. of wash water required	Very small amount of wash water- 0.2 to 0.6 % of total water filtered.	Large amount – from 1 to 5 % of total water filtered.
17	Period of cleaning	At interval of 1 to 3 months	Frequently, at intervals of 1 to 3 days

4.8.2 Slow Sand Filter

Slow sand filters can provide a single step treatment for polluted surface waters of low turbidity (< 20 NTU). When raw water turbidity is high, simple pre treatment such as storage, sedimentation or primary filtration will be necessary to reduce it within desirable limits. Slow sand filter consists of an open box about 3 m deep rectangular or circular in shape and made of concrete or masonry (Fig.4.24).

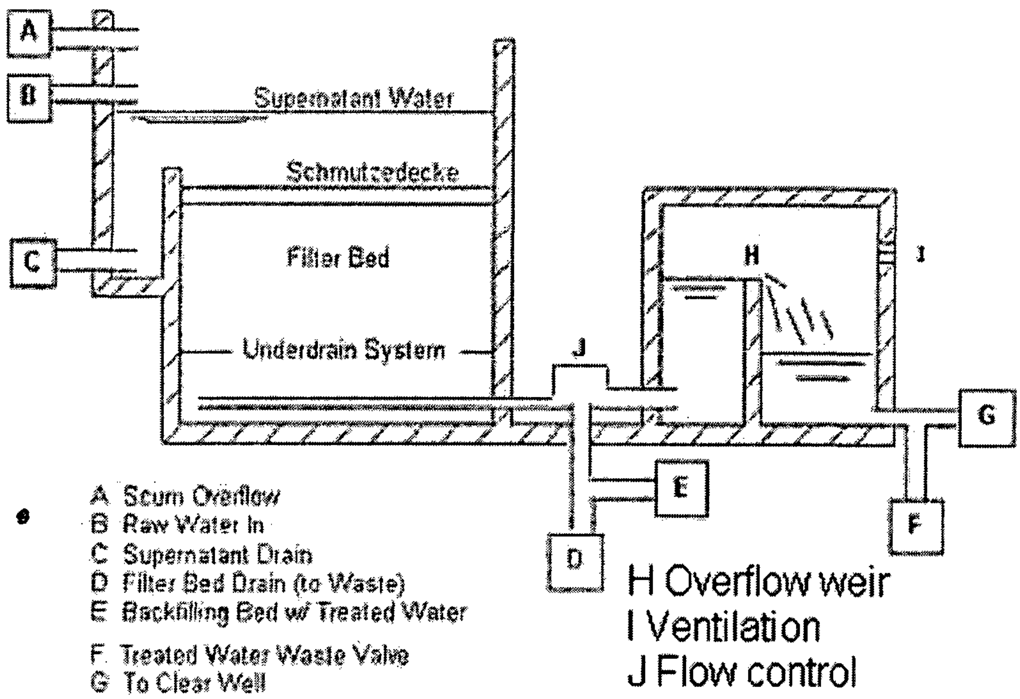


Fig.4.24. Basic Elements of Slow Sand Filter

4.8.3 Rapid Sand Filter

After the coagulation, flocculation, and sedimentation processes are complete, the raw water is much clearer, but is not yet potable. Some residual iron, manganese, clay, inert solids, bacteria, and other constituents are still present in the water. Even in a high quality unfiltered treated water having a total suspended solids concentration of 0.1 mg/l, approximately 200 million particles are present per liter. Therefore, filtration of the water is essential in further reducing the concentration of these constituents and this is done by rapid sand filtration.

Capacity of a Filter Unit. For flexibility of operation, a minimum of four units should be provided which could be reduced to two for smaller plants. Otherwise, the number of units at a filter plant may be roughly estimated using Morrell and Wallace's equation, i.e,

$$N = 1.22\sqrt{Q}$$

Where, N = no. of filter units

Q = plant capacity in million liters per day.

Dimensions of Filter Unit. Where filters are located on both sides of a pipe gallery, the ratio of length to width of a filter box is about 1.25 to 1.33. A minimum overall depth of 2.6m including a free board of 0.5 m may be adopted. A definition sketch of rapid sand filter is shown in Fig. 4.25.

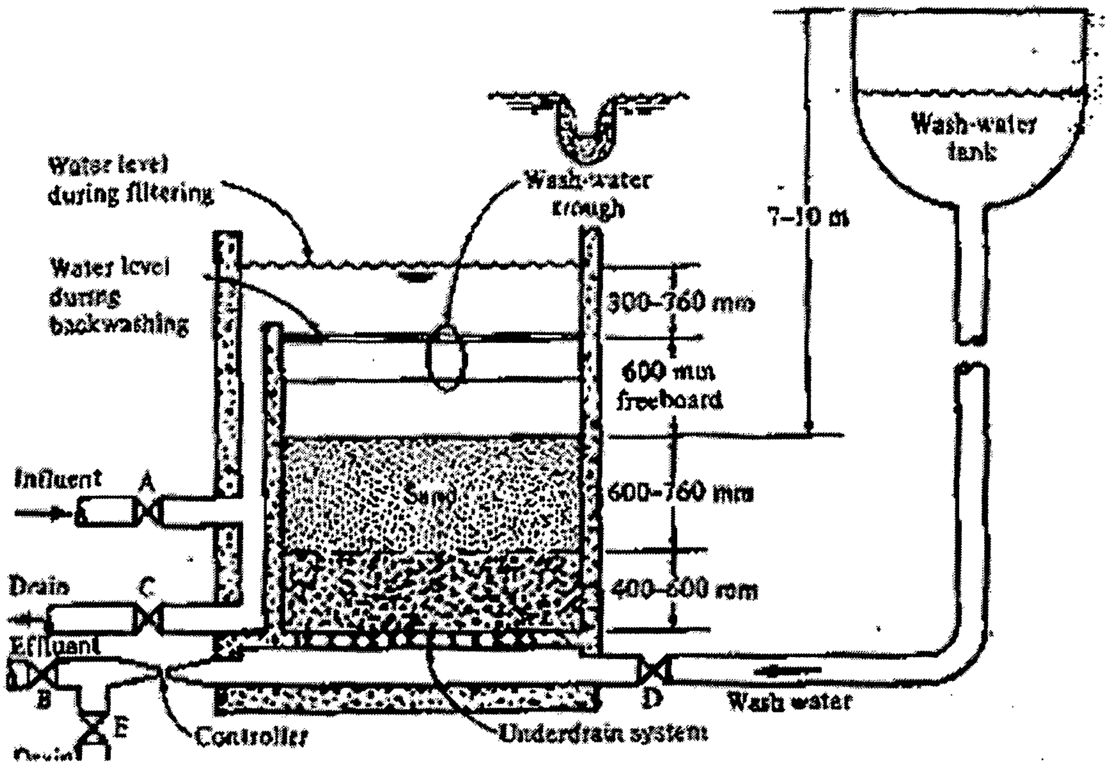


Fig. 4.25. Definition Sketch of Rapid Sand Filter.

Depth of Water. The standing depth of water over filter varies between 1 and 2m. Freeboard should be at least 0.5m so that when air binding are encountered, it will facilitate the additional level of 0.15 to 0.3m of water being provided to overcome the trouble.

Filter Bottom and Strainer System. The most common type of under-drain is central manifold with laterals either perforated on the bottom or having umbrella type strainers on top. The perforations vary from 5 to 12 mm in diameter and should be staggered at a slight angle from the vertical axis of the pipe. Ratio of total area of perforations to total cross sectional area of laterals should not exceed 0.5 for perforations of 12 mm and should decrease to 0.25 for perforations of 5 mm. Ratio of total area of perforations to the entire filter area may be 0.3%. The spacing of laterals shall be about 300 mm. The cross sectional area of manifold should be preferably 1.5 to 2 times the total area of the laterals to minimize frictional losses.

Filter Gravel Size of gravel and depth may vary according to the type of filter bottom and strainer system used.

(a) for strainer or wheeler type under drain system, gravel shall be of 2 mm minimum size, 50 mm maximum size and 0.3 to 0.5 m deep;

(b) for perforated pipe under drain system, gravel shall be 2 mm minimum size, 40 mm maximum size and 0.5 m, in depth.

Wash Water Gutters. Gutter may be square, V-shaped or semi-circular. They are generally spaced such that the horizontal distance traveled by wash water is kept between 0.6 to 1.0 m. The edge of the trough should be slightly above the highest elevation of the sand as expanded in washing. The bottom of the gutter should clear the top of the expanded sand by 50 mm or more. The troughs are designed as free falling weirs or spillways and the discharge capacity may be calculated from the formula

$$Q^{\bullet} = 1.376\, bh^{3/2}$$

Where, b is width of trough in m and h is depth of water in m.

High Rate Backwash .High rate back wash is applied normally where no other agitation is provided. Generally, wash water rate is 600 lpm/m^2 or 36m for a period of 10 minutes. For high rate wash, the pressure in the under drainage system should be 6 to 8 m with wash water requirement being 40-50 m/hr. for about 6 to 10 minutes.

Air Wash System. In the air wash system, compressed air is used to achieve scrubbing action with smaller volume of wash water. Air may be forced through the under drains at the rate of 600 to 900 lpm/m^2 @ 0.35 kg/cm^2 for a period of about 5 mins. before wash water is introduced at the rate of 400 to 600 lmp/m^2. Air may be also passed through a separate piping system placed between gravel and sand at the rate of 45 – 50 m/hr while wash water is applied at 12 – 15 m/hr.

4.8.4 Rapid Gravity Dual Media Filter

Most modern water treatment plants now use rapid dual-media filters following coagulation and sedimentation. A dual-media filter consists of a layer of anthracite coal above a layer of fine sand. The upper layer of coal traps most of the large floc, and the finer sand grains in the lower layer trap smaller impurities. This process is called *in-depth filtration,* as the impurities are not simply screened out or removed at the surface of the filter bed, as is the case in slow sand filters. Dual media filters behave very much like two single medium filters in series, each with a different grain size. Filtration rate of 16 m^3/m^2/hr could be recommended with filter run of at least 12 hrs. In general, dual media filters are operated at the rate of 7.5 to 12 $m^3/m^2/hr$. The backwash rates of 42 to 54 $m^3/m^2/hr$ have been employed to clean the filter.

4.8.5 Multimedia Filter

In order to enhance in-depth filtration, so-called mixed-media filters are used in some treatment plants. These have a third layer of a fine-grained, dense mineral called garnet at the bottom of the bed. Mixed-media filters are basically improved dual media filters with increased water quality. However, the cost is much higher than dual media filter, because garnet is quite expensive. Typical design values for dual and mixed media filters are shown in Table 5.9.

Table 4.12. Typical Media Design Values for Dual & Mixed Media Filters

Parameter	Dual medium filter	Mixed-media filter
Anthracite layer		
Effective size, mm	0.7 – 2.0	1.0 – 2.0
Uniformity coefficient	1.3 – 1.8	1.4 – 1.8
Depth, cm	30 – 60	50 – 130
Sand layer		
Effective size, mm	0.45 – 0.6	0.4 – 0.8
Uniformity co-eff.	1.2 – 1.7	1.2 – 1.7
Depth, cm	20 - 40	20 – 40
Garnet layer		
Effective size, mm		0.2 – 0.8
Uniformity co-eff.		1.5 – 1.8
Depth, cm		5 - 15

4.8.6 Design of Filter Units for Almora Water Treatment Plant

As practice in modern water treatment plants, rapid gravity filter is used to remove small flocs or precipitant particles not removed in the settling of coagulated waters. Moreover, the necessity of too much purification and maximum removal of bacteria (as is achieved by slow sand filters) is not felt necessary due to advancement of disinfection techniques. Slow sand filter cannot produce high filtration rate as compared to rapid sand filter, hence, it is not actually fit for use as municipal water treatment plant as it will require large areas. Hence, Rapid Gravity Filter is selected and designed as below.

General Provisions :

1. Required outflow	:	19 mld
2. Quantity of backwash water used	:	3 % of filter output
3. Time lost during backwashing	:	30 mins.
4. Design rate of filtration	:	4.8 $m^3/m^2/hr$ (Ref.GOI, manual)
5. Length to width ratio	:	1.25 – 1.33 :1
6. Under drainage system	:	central manifold with laterals
7. Size of perforations	:	9 mm

Design of Filter Bed Dimension:

Required filtered water per hour = 19 x 1000/24

= 791.67 m^3/hr

Design flow for filter after accounting for backwash water & time lost in back washing = 19 x 1.03 x 24/23.5

= 19.98 mld or 832.76 m^3/hr

Area of filter required = 832.76/4.8

= 173.492 m^2

The no. of units of filter plant may be roughly estimated as per Morell & Wallace as:

Number of filter beds, N = $1.22 \sqrt{Q}$, where Q is in mld

= $1.22 \sqrt{19}$

= 5.32 ; Say 6

Provide 6 filter beds, 4 being minimum no. for flexibility of operation for medium and large plants.

Area of each filter bed = 173.492/6

= 28.915 m^2

Assume L : W = 1.3 : 1

Width of filter, W = $(28.915/1.3)^{0.5}$

= 4.716 m

Length of filter = 1.3 x 4.716

= 6.13 m

Provide 6 filter units, each of **6.2 m x 4.75 m** size.

Sand Depth *:*

Assume a depth of **sand as 60 cm** and effective size of sand as **0.5 mm** (0.45 to 0.7 mm in manual). The depth can be checked against breakthrough of floc through the sand bed, by calculating minimum depth required by Hudson's formula in metric unit as:

$Qd^3h / L \quad = \quad B \times 29323$

where, $Q \quad = \quad 4.8 \times 2 = 9.6\ m^3/m^2/hr$ (assuming 100 % overloading of filter under emergency)

$d \quad = \quad$ mean dia. of sand in mm = 0.6 mm

$h \quad = \quad$ terminal headloss in m = 2.5 m

$L \quad = \quad$ depth of bed in m.

Assuming $B = 4 \times 10^{-4}$ for poor response to filtration and average degree of pretreatment. We get from the above equation.

$9.6 \times 0.6^3 \times 2.5 / L \quad = \quad (4 \times 10^{-4}) \times 29323$

$L \quad = \quad 0.44\ m$

Hence, assumed depth of 60 cm is adequate to avoid breakthrough of floc.

Estimation of Gravel and Size Gradation :

Assume a size gradation of 2 mm at top and to 50 mm at bottom. The requisite depth in cms of a component gravel layer of size d in mm can be computed from the empirical formula.

$L \quad = \quad 2.54\ K\ (\log d)$, where, $K = 10$ to 14

Let us assume, K = 12

The depth of various layers of gravel are,

Size, mm	=	*2*	*5*	*10*	*20*	*40*
Depth, cm	=	*9.2*	*21.3*	*30.5*	*40*	*49*
Increment, cm	=	*9.2*	*12.1*	*9.2*	*9.5*	*9*

Provide a gravel depth of **50 cm.**

Design of Under- Drainage System *:*

Plan area of each filter $= 6.2 \times 4.75$

$= 29.45\ m^2$

Total area of perforation $= 3 \times 10^{-3} \times$ Area of filter

$= 0.0883\ m^2$

$= 883.5\ cm^2$

Total no. of perforation of dia. 9 mm

= $883.5/(\pi x0.9^2/4)$

= 1338.77 , Say **1339 nos.**

Total cross sectional area of laterals

= 3 x area of perforations

= 3 x 883.5

= 2650.5 cm^2

Area of central manifold = 2 x area of laterals

= 2 x 2650.5

= 5301 cm^2

Diameter of central manifold= $\sqrt{(5301 \times 4/\pi)}$

= 82.15 cm

Provide a commercially available diameter of **900 mm**

Assume a spacing of 15 cm for laterals.

No. of laterals = (2 x 6.2 x 100)/15

= 82.67 ; Say 83 nos.

Cross sectional area of each lateral

= 2650.5/83

= 31.93 cm^2

Diameter of lateral = $\sqrt{(31.93 \times 4/\pi)}$

= 6.37 cm

Provide laterals of **diameter 80 mm .**

No of perforations per lateral = 1339/83

= 16.13 , Say 17

Length of lateral = ½(width of filter – dia. of manifold)

= ½(4.75 – 0.9)

= 1.925 m

Spacing of perforations = 1.925 x 1000/17

= 11.32 cm

Provide 17 perforations of 9 mm dia. @ 113 mm c/c.

Check :

Length of each lateral/dia. of lateral = (1.925 x 1000)/80

= 24.06 < 60 ....(O.K)

Now let us assume that the rate of washing of filter be 45 cm/min.

The wash water discharge = 0.45/60 x area of bed (29.45)

= $0.221\ m^3/s$

Velocity of flow in lateral for wash water

= $0.221/\{83\ (\pi/4 \times 0.08^2)\}$

= 0.53 m /s

Similarly, velocity of flow in manifold

= $0.221/(\pi/4 \times 0.9^2)$

= 0.35 m/ s ,which is less than 1.8 to 2.4 m/s (maximum permissible) (Ref. Garg/523)

Design of Wash Water Trough :

Assume a wash water rate of 36 $m^3/m^2/hr$, (24 to 26 $m^3/m^2/hr$)

Wash water discharge for 1 filter

= $36 \times 29.45\ m^3/hr$

= $1060.2\ m^3/hr$

= $0.294\ m^3/sec.$

Assuming a spacing of 1.6 m for wash water trough which will run parallel to the dimension of the filter unit.

No. of troughs = 4.75/1.6

= 2.96, Say 3 nos.

Discharge per unit trough = 0.294/3

= $0.098\ m^3/s$

For a width of 0.4 m, the water depth at upper end in the trough is given by

$$Q = 1.376\ bh^{3/2}$$

where, b = width of trough in m

h = water depth in m

Q = discharge capacity in m^3/s

$\therefore$, $0.098 = 1.376 \times 0.4 \times h^{3/2}$

i.e, h = 0.316 m, Say 0.35 m

Assume a freeboard of 0.1 m, provide a depth of 0.45 m

Provide **3 Troughs of 0.4 m wide x 0.45 m depth** in each filter.

Bottom of trough from sand bed = 0.05 m

Computation of Total Depth of Filter Box:

Depth of filter box = sum of depths for (i) underdrains, (ii) gravel, (iii) sand, (iv) water depth, (v) free board = 0.9 + 0.5 + 0.6 + 1.2 + 0.5

= **3.7 m**

Main Gutter:

Assume backwash rate as 60 cm rise/min. (15 to 90 cm/min, Garg/526)

Area of filter bed = 6.2 x 4.75

= 29.45 m^2

Flow per filter bed at 0.6 m rise/min

= 0.6 x 29.45

= 17.67 m^3/min

= 0.2945 m^3/s

From, $Q = 1.376\ b\ h^{3/2}$

We have, $0.2945 = 1.376 \times b \times 0.6^{3/2}$

b = 0.46 m , Say **500 mm**

Wash Water Tank :

Required flow for back wash is @ 0.5 m^3/m^2/min

Duration of backwash is 6 mins for 2 filter beds (Handbook by Bajwa)

Water requirement = to wash 2 filter beds at a time

Volume of tank = 2 x 0.5 x 6.2 x 4.75 x 6

= 176.7 m3 , Say 177 m^3

Assuming depth of water in the tank as 3 m,

Area of tank required = 177 / 3

= 59 m^2

Size of square tank provided= $59^{1/2}$

= 7.68 m , Say 7.7 m

Provide a square tank of **7.7 m x 7.7 m x 3 m + 0.5 m FB**

Wash Water Rising & Feeding Main :

The rising main shall take the water from the clean water reservoir to the wash water tank

Let velocity = 3 m/s (2.4 to 3.6 m/s, Ref. Bajwa, 465)

From the relation Q = A . V ; ....(i) where $A = \pi d^2/4$

Let us assume the wash tank to be filled in 2 hours, then

Discharge required/sec= 177/(2 x 3600)

= 0.0246 m^3/s

From (i), 0.0246 = $\pi d^2/4 \times 3$

∴ Diameter of pipe, d = 0.102 m ; Say 100 mm

Actual velocity in rising main= $0.0246/(\pi \times 0.1^2/4)$

= 3.1 m/s................ ....(O.K)

Now, Volume of water required to wash 2 filter beds at a time in 6 min = 176.7 m3

Discharge per second = 176.7/(6 x 60)

= 0.49 m^3/s

From (i) 0.49 = $\pi D^2/4 \times 3$

Where, D = diameter of feeding main

Or D = 0.456 m , Say 500 mm

Velocity check = $0.49/(\pi \times 0.5^2/4)$

= 2.5 m/s......................................(O.K)

Provide **100 mm ϕ** pumping main & **500 mm ϕ** feeding main.

Wash Water Pump :

Volume of water to be pumped in 2 hours= 177 m^3

Pumping head = 15 m

Rate of pumping = 177/(2 x 3600)

= 0.0246 m^3/s

Water power required = $\gamma_w . Q. H$

= 9.81 x 0.0246 x 15/ 0.65

= 5.57 KW or 7.58 HP

Provide **2 pumps of 8 HP each** with one of them as stand by.

Air Blowers/ Compressors :

600 to 900 lpm/m2 area of filter at 0.35 kg/cm^2 is to be supplied. Presuming the duration of air wash as 5 mins., the total quantity of air required for one bed @ 0.9 $m^3/m^2/min$. = 0.9 x 29.45 x 5

= 132.52 m^3

= 26.5 m^3/min

Provide 2 air blowers of 1600 m^3/hr capacity each (one working and one standby). This will be housed in filter house.

Volume of air tank @ at least double of air required to wash one filter bed

= 2 x 132.52

= 2.65 m^3

Effluent Pipe From Individual Filter :

Required outflow from each filter unit= 19.98 /6

= 3.33 mld or 0.0385 m^3/s

Assume velocity of flow= 1.2 m/s........ (0.9 to 1.8 m/s)

From $Q = A . V$

$0.0385 = \pi d^2/4 \times 1.2$

or d = 0.202 m ; say 200 mm

Actual velocity = $0.0385/ (\pi \times 0.2^2/4)$

= 1.22 m/s..............................O.K.

Provide **200 mm ϕ** pipe effluent conduit from single filter.

Common Effluent Pipe Leading To CLW Well :

Total effluent flow : 19.98 mld or 0.231 m^3/s

Assume velocity of flow : 1.2 m/s.........(0.9 to 1.8 m/s)

∴ Diameter of pipe = $\sqrt{((0.231 \times 4)/(\pi \times 1.2))}$

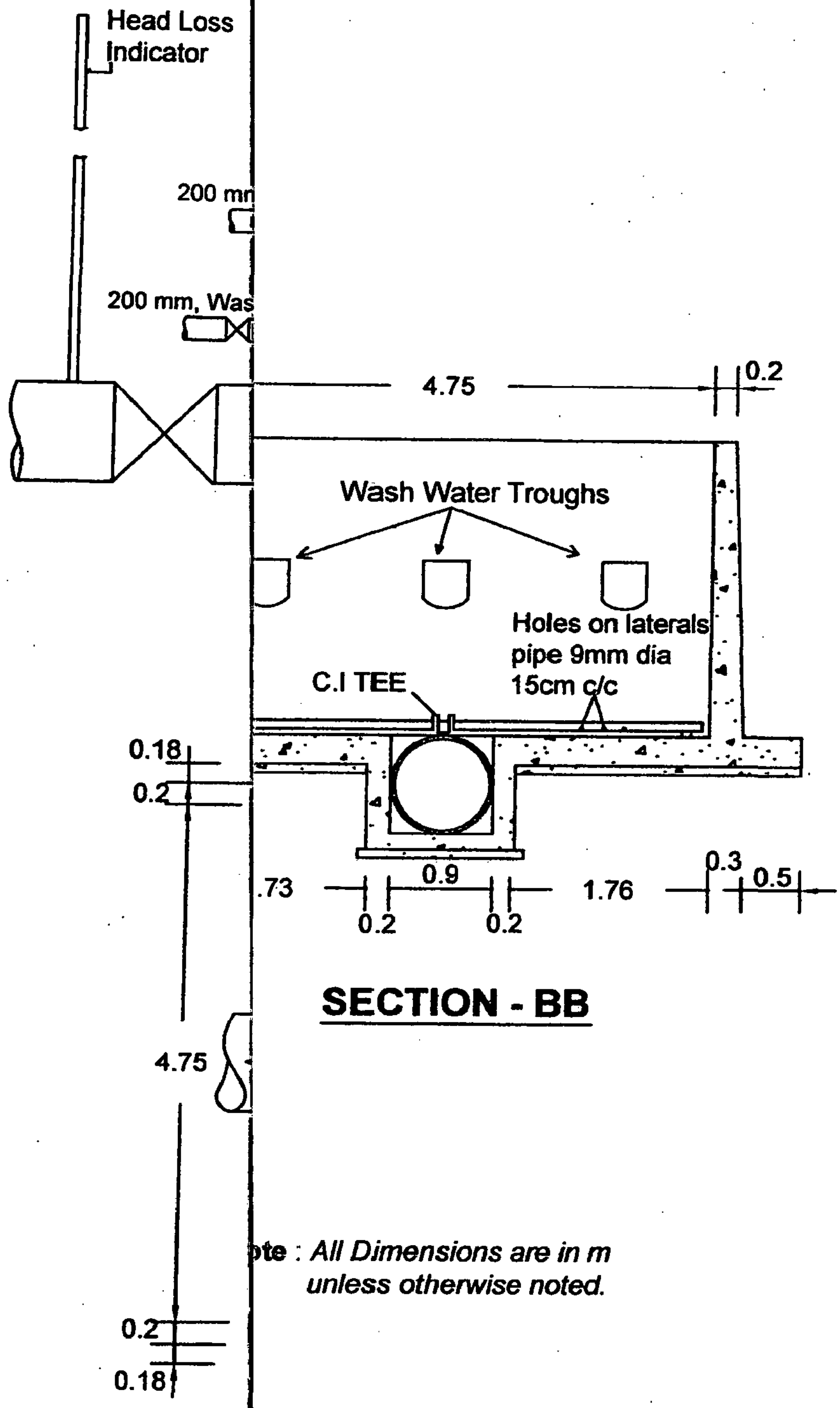
= 0.495 m ; say 500 mm

Actual velocity of flow= $0.231 / (\pi \times 0.5^2/4)$

= 1.17 m/s ......................O.K.

Provide 500 mm ϕ pipe as common effluent pipe.

Similarly, provide 200 mm ϕ & 500 mm ϕ pipe for single influent pipe and common influent pipe respectively. Details of RGF is given in Fig.4.26.



SECTION - BB

ote : *All Dimensions are in m unless otherwise noted.*

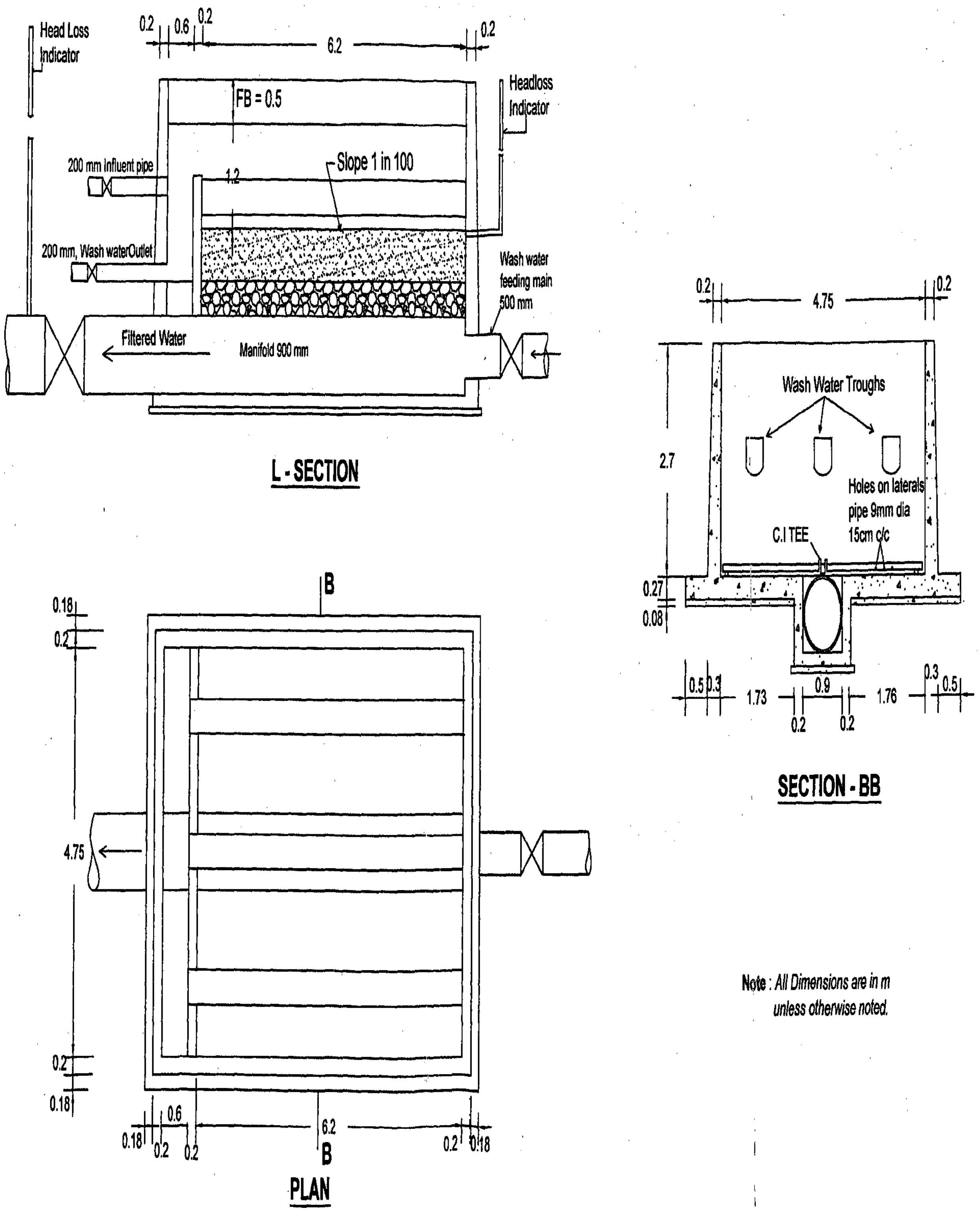


Fig. 4.26. DETAILS OF RAPID GRAVITY FILTER

4.9 DISINFECTION

The filters are unable to remove all the disease bacteria. They can remove only few types of bacteria. Therefore, the water which comes out from the filter may contain some disease causing bacteria in addition to useful bacteria and these bacteria have to be killed before the water is supplied to the public. The disinfection not only removes the existing bacteria from the water at the plant, but also ensures their immediate killing even afterwards, in the distribution system.

4.9.1 Methods of Disinfection

Among various disinfection methods, only chlorine and ozone are suitable for public water treatment [Gark,S.K,2006]. The disadvantage of using ozone is that it does not give residual effects. The use of ultraviolet rays and silver is very costly, and hence they are not recommended for public water supply. Treatment with potassium permanganate may be used for disinfecting well water supply in villages which are generally contaminated with lesser amounts of bacteria. Among disinfectants, **chlorine** in its various forms is invariably and almost universally adopted for disinfecting public water supplies. It is cheap, reliable, and capable of providing residual disinfecting effects for long periods. Only disadvantage is that when used in greater amounts, it imparts bitter and bad taste to the water.

4.9.2 Disinfection By Chlorine

When chlorine is added to water, it forms hypochlorous acids or hypochlorite ions, which have an immediate and disastrous effect on most forms of microorganisms.

$$Cl_2 + H_2O \xrightarrow{pH>5} \underset{\text{(hypochlorous acid)}}{HOCl} + HCl$$

$$HOCl \underset{pH<7}{\overset{pH>8}{\rightleftharpoons}} \underset{\text{hydrogen ions}}{H^+} + \underset{\text{hypochlorite ions}}{OCl^-}$$

The dissociation of hypochlorous acid into ions is more effective at high pH values and vice versa. In the pH range of below 5, chlorine does not react and remains as elemental chlorine. Among the three free available chlorine, hypochlorous acid is the most destructive, being about 80 times more effective than OCl^-. Hence, the pH value of water to be chlorinated is generally maintained slightly less than 7 to keep dissociation of HOCl minimum. Chlorine will also react immediately with ammonia present in water to form various chloramines. The mono-chloramine predominates at pH value > 7; di-chloramine at pH range of 5 – 6.5; and nitrogen trichloramine at pH < 4.4. The chloramines formed are called combined chorine and are stable and capable of disinfecting water but about 25 times less effective than free chlorines. In chlorination, the free chlorine will instantaneously kill pathogens, while chloramines will provide long term germicidal effect.

4.9.3 Chlorine Doses

In general, most of the water are satisfactorily disinfected if the free chlorine residual is about 0.2 mg/l, 10 min. after chlorine is applied. The effectiveness of chlorine depends upon the pH of water. Therefore, the optimum chlorine for given water is generally determined experimentally by adding varying amounts of chlorine to a given sample and observing the residual left after contact period of about 10 min. The dose which leaves a residual of about 0.2 mg/l is then selected.

4.9.4 Forms of Chlorine and Application

Chlorine is generally applied in the following forms:

- Liquid chlorine or chlorine gas, as free chlorine
- Hypochlorites or bleaching powder, as combined chlorine.
- Chlorine dioxide

The liquid form is generally preferred and almost exclusively used these days. Calcium and sodium hypochlorites and bleaching powder may be used for chlorinating small public supplies. Bleaching powder can be applied to the water either by using gravity feed chlorinators or injected in water pipe in solution form by means of dosing pump. Free chlorine is applied to the water to be treated, through an instrument called Chlorinators or chloronome. Solution feed chlorinators, using liquid chlorine are almost invariably used these days.

4.9.5 Design of Chlorinator for Almora Water Treatment Plant

Let us select chlorination for disinfection. Let us select liquid chlorine which is to be fed to water by chlorinator.

Dosage : 1 ppm for pre-chlorination (say)

1 ppm for post-chlorination (say)

Total requirement per day :

19.6 kg/day for pre-chlorination

19.0 kg/day for post-chlorination

Provide **2 nos. of 2 kg/hr chlorinators** (one working and one stand by).

4.10. CLEAR WATER WELL (CLW)

For pumping of 23 hours, considering loss due to tripping and other minor interruptions [GOI Manual on water supply,1999], let us design clear water reservoir for 1 hr. detention period to ensure 24 hrs. working capacity of treatment plant or to balance pumping rate. Also, storage required for chemical dilution & feed, pump operation, in-plant domestic water, filter backwash are also considered [Qasim, et al,2000].

(a) Storage required to balance pumping rate:

Design outflow from treatment plant	=	19 mld
∴ Volume required for 1 hr. detention	=	19 x 1000 x 1/24
	=	791.67 m^3

(b) Storage required for filter backwash:

Water required for one time backwash (2 filters)= 177 m^3

(c) Storage required for chemical dilution & feed :

(i) Water required for coagulant dilution :

Daily feed rate of alum	=	1000 kg
Water required @ 5 % strength solution	=	1000 x 95/5
	=	19000 lts/d or 19 m^3/d

(ii) Water demand for chlorine feeding :

Chlorine feed rate	=	38.6 kg/d
Injector water flow	=	10 lt/min. (from chart, Qasim,582)

Total capacity required = 1002.07 m^3, say 1000 m^3

Provide **2 nos. Clear well of each size 12.9 m x 12.9 m x 3 m depth + 0.3 m FB**

CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1 SUMMARY

- Selection of water treatment process mostly depends on the quality of raw water and on the desired quality of the finished water. It may be vary from simple system consisting only one unit operation or very complex system which may consists of combination of many unit operations.

- Pondage of water due to barrage and the site condition of proposed intake on the bank of Kosi river makes *Single Well type of a river intake or Jack well* to be most suitable. Hence, it is selected for intake of proposed Almora water supply scheme.

- The results of raw water analysis of Kosi river show the following undesirable occurrence:
  - presence of total coliform organisms in 100 ml water sample in two successive results which is undesirable.
  - presence of iron beyond the acceptable limit.
  - excess turbidity level to as much as 1500 mg/l during high flow season which is more than acceptable limit of 5 mg/l.
  - presence of slight taste & odor problems.
  - slight excess in chromium concentration in one analysis.

- The treatment plant for Almora should consists of state-of the-art conventional water treatment plant consisting Clariflocculator, Rapid gravity filter preceded by plain sedimentation and aeration to accomplish high turbidity removal, taste & odor control , Iron and chromium removal.

- The proposal of pre-sedimentation tank for reducing the turbidity level of raw water nearby the intake structure, lifting of raw water from pre-sedimentation tank to Matela treatment plant through a static head of about 58m makes it necessary to construct two pump stations, one low lift pump station at the intake, and another high lift pump station. Pump station at the intake is wet-pit pump station, and other should be dry-pit pump station.

- Cascade aerator is proposed to be provided to reduce the concentration of taste and odor and iron by volatilization or oxidation thus rendering them insoluble.

- The effective pH range for the use of alum coagulant is 6.5 to 8.5. Since the average pH value of kosi river falls within this limit, alum is proposed for use in Almora water treatment plant.

- Anionic polyacrylamides with very high molecular weights, which is regarded as the most effective coagulant aid is proposed to be used to improve and accelerate the process of coagulation and flocculation. Pre-chlorination should be done to control biological growths in raw water conduits, improve coagulation, prevent formation of mud ball and slime formation in filters, reduce taste& odor and color, and to minimize the post chlorination dosage. Filter aid (high molecular weight nonionic polymer) may also be used to enhance filtration during periods of poor settled-water quality.

- In public water treatment plant especially where the flow exceeds 300 m^3/hr, mechanical mixing also known as flash mixing, is desirable due to flexibility and better mixing. Design flow of Almora treatment plant in one of the two proposed rapid mixers will be 395 m^3/h. Hence, mechanical rapid mix with paddle type impeller is proposed.

- Clariflocculator effectively combine coagulation and sedimentation processes in a single unit, and proves to be compact and space saving. Moreover, clariflocculators have been widely and successfully used for treating river water having almost the same quality as that of Kosi river. Circular shaped unit which is even more compacted than rectangular shaped clariflocculator should be used for Almora water treatment plant.

- For flocculated water and water treatment plant for town and cities, use of rapid gravity filter is required. As per usual practice in India, single medium rapid sand filter is proposed for Almora water treatment plant.

- Disinfection is required for rendering any type of water potable to ensure inactivation of harmful bacteria. Ozone and chlorine have been used in large scale for public water treatment. Since ozone is not having residual effect to ensure safety up to the consumers' end, chlorine is proposed for use for disinfection of Almora water supplies. Chlorination is most effective at a pH value slightly less than 7 or near to it, it may not be required to alter the pH of Kosi water which is already having average pH value slightly more than 7.

5.2 CONCLUSIONS

The reliable water sources for public water supply of medium to large towns in hilly region are generally, surface water rivers. Since the towns in hilly areas are always located at much higher elevation from the rivers, multiple stage high head pumping of water inducing a very high cost of construction and maintenance is always required. In terms of treatment processes, it mainly depends on raw water quality and desired quality of treated water. Hillside construction may have an advantage in accommodating the headloss in the plant without excessive excavation. The topography of hilly terrain also can always be utilized for discharging sludges from the treatment units by hydrostatic force of water and gravity, thus escaping from the requirement of using pumps as in case of plain area.

5.3 SCOPE FOR FURTHER STUDY

1) Aeration alone without use of oxidizing agents such as PAC can remove taste & odors only to a certain extent. Up to what extend of taste & odor problems in the raw water, the aeration alone can prove to be useful, may be studied.

2) The possibility of other treatment combinations, say direct filtration train, i.e, coagulation-flocculation- rapid filtration (without sedimentation) may be studied with respect to Kosi river source.

3) The feasibility of use of roughing filter in place of pre plain sedimentation tank may be studied.

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Appendix I

Drinking Water Quality Standards

Physical – Chemical Quality

Components	WHO mg/l	USEPA mg/l	IS10500 mg/l
Colour (hazen scale)	15	15	5
Odour		3 (Threshold odor no.)	Unobjectionable
Taste			agreeable
Turbidity(NTU)	5		5
pH		6.5-8.5	6.5-8.5
Total hardness as CaCO3			300
Iron as Fe	0.3	0.3	0.3
Chlorides as Cl	250	250	250
Dissolved solids (total)	1000	500	500
Calcium as Ca			75
Copper as Cu	1.0	1.3	0.05
Manganese as Mn	0.5	0.05	0.1
Sulphate as SO4	250	250	200
Nitrate as NO3-	50	10 (measured as N)	45
Nitrite as NO2-	0.3 (acute) 0.2(chronic)	1.0 (measured as N)	
Flouride as F	1.5	4.0	1.0
Phenolic compounds as C6H5OH			0.001
Mercury as Hg	0.001(total)	0.002 (Inorganic)	0.001
Cadmium as Cd	0.003	0.005	0.01
Asbestos (filter >10 microns)		7 million fibre/litre	
Silver		0.10	
Foaming agent		0.5	
Selenium as Se	0.01	0.05	0.01
Arsenic as As	0.01	0.05	0.05
Cyanide as Cn	0.07	0.2(Free)	0.05
Lead as Pb	0.01	0.015	0.05
Zinc as Zn	3	5	5.0

Chromium as Cr6+	0.05	0.1(total)	0.05
Thallium		0.0005	
Mineral oil			0.01
Anionic detergents as (MBAS)			0.2
Boron	0.5		1.0
Aluminum (as Al)	0.2	0.2	0.3
Alkalinity			200
Antimony	0.005	0.006	
Barium	0.7	2.0	
Nickel	0.02		
Molybdenum	0.07		
Sodium	200		
Ammonia	1.5		
Beryllium		0.004	
Organic Chemicals	μg/l	mg/l	
Carbon tetrachloride	2	0.005	
Dichloromethane	20	0.005	
1,1Dichloroethylene		0.007	
1,2Dichloroethane	30	0.005	
1,1,1trichloroethane	2000		
Vinyl chloride	5	0-0.002	
Benzene	10	0-0.005	
Toluene	700	1.0	
Xylene	500	10	
Ethyl benzene	300	0.7	
Styrene	20	0.1	
Benzo (a)pyrene (PAH)	0.7	0.0002	Polyaromatic ydrogen..Nil (as AH)
Trichlorobenzene (total)	20		
1,2,4Dichlorobenzene		0.2	
Radionuclides			
Alpha particles	0.1(Bq/L)	15 (pCi/L)	0.1(Bq/L)
Beta particles	1.0(Bq/L)	4 milliremes per year	1.0(Bq/L)
Radium Ra 226 & Ra 228 9combined)		5 pCi/L	
Uranium	0.002 mg/l	30 μg/l	

Pesticides	WHO µg/l	USEPA mg/l	IS 10500
Alachlor	20	0.002	
Aldicarb	10		
Aldrin/dieldrin	0.03		
Atrazine	2	0.003	
Bentazone	300		
Carbofuran	7	0.04	
Chlordane	0.2	0.002	
Chlorotoluron	30		
Cynazine	0.6		
DDT	2		
1,2-Dibromo-3- chloropropane	1	0.0002	
1,2-Dibromomethane	0.4-15		
2,4-Dichlorophenoxyacetic acid	30		
1,2-Dichloropropane	40	0.005	
1,3-Dichloropropene	20		
Diquat	10	0.02	
Heptachlor and Heptachlor epoxide	0.03	0.0004 0.0002	Absent
Hexachlorobenzene	1	0.001	
Isoproturon	9		
Lindane	2	0.0002	
MCPA	2		
Methoxychlor	20	0.04	
Metolachlor	10		
Molinate	6		
Pendimethalin	20		
Pentachlorophenol	9	0.001	
Permethrin	20		
Propanil	20		
Pyridate	100		
Simazine	2	0.004	
Terbuthylazine	7		
Trifluralin	20		
Mecoprop	10		
Fenoprop	9		

Dichlorprop	100		
Disinfectant and disinfection by-products	**WHO µg/l**	**USEPA mg/l**	**IS 10500**
Bromate	25	0.010	
Chlorate			
Chlorite	200	0.8-1.0	
Chloramines as Cl2	3 (Monochloramines)	4	
Chlorine dioxide as ClO2		0.8	
Chlorine as Cl2	5.0	4.0	
Halo acetic acids as (HAA5)		0.06	
Total trihalomethane	Bromoform 100 Dibromochloromethane 100 Bromodichloromethane 60 Chloroform 200	0.080	
Cyanogen Chloride (as Cn)	70		

Microbilogical Quality

Microorganisms	**WHO**	**USEPA (mg/l)**	**IS 10500**
Cryptosporidium		0	
Giardia lambia		0	
Legionella		0	
Treated water in the WDS Total Coliforms (including faecal coliform and E.coli) E. coli	not detectable in any 100 ml sample or present only in < 5% of samples throughout the year in case of large supply. 3 in any 100ml sample ccasionally, but not in consecutive samples.	0 0	Total coliform ≤ 10/100 ml in random sample. Total coliform should not be present in two consecutive samples. Should not be found in 95% of samples throughout a year. not detectable in any100 ml sample
Viruses (enteric)		0	