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KINGDOM OF THAILAND
MINISTRY OF INTERIOR
PUBLIC WORKS DEPARTMENT

FEASIBILITY STUDY
ON
THE SANITARY DISTRICT WATER WORKS PROJECT
IN
THE NORTH-EASTERN REGION OF THAILAND

— DESIGN MANUAL
— OPERATION AND MAINTENANCE MANUAL
— RECOMMENDATION ON THE IMPROVEMENT OF EXISTING WATER WORKS

FEBRUARY 1986

JAPAN INTERNATIONAL COOPERATION AGENCY

KINGDOM OF THAILAND
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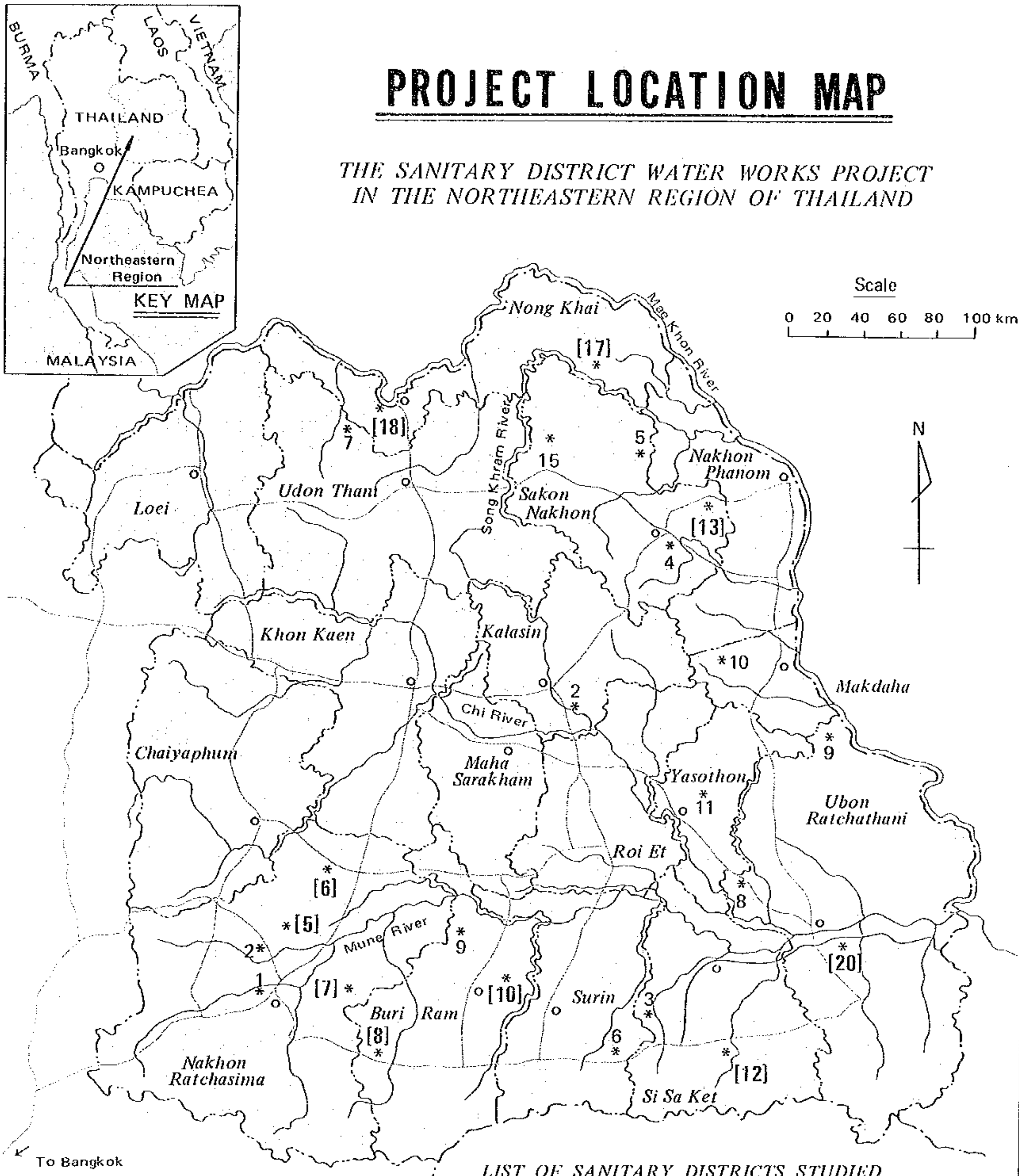
PROJECT LOCATION MAP

THE SANITARY DISTRICT WATER WORKS PROJECT IN THE NORTHEASTERN REGION OF THAILAND



LIST OF SANITARY DISTRICTS STUDIED

New Sanitary District Without Water Works (NSD)		*Existing Sanitary District With Water Works (ESD)*	
2	*Rong Kham*	1	*Cho Ho*
[5]	*Kham Sakae Sang*	2	*Non Thai*
[6]	*Nong Bua Lai*	3	*Prang Ku*
[7]	*Huai Thalaeng*	4	*Tha Rae*
[8]	*Nong Ki*	5	*Akat Ammuai*
9	*Hin Lek Fai*	6	*Sankha*
[10]	*Huai Rat*	7	*Ban Phu*
11	*Sai Mun*	8	*Khuang Nai*
[12]	*Khun Han*	9	*Chanuman*
[13]	*Kusuman*	10	*Khamcha-i*
15	*Don Khuang*		
[17]	*Phon Charoen*		
[18]	*Nong Song Hong*		
[20]	*Huai Kha Yung*		

LEGEND

	National Boundary
	Provincial (Changwat) Boundary
o	*Provincial Capital*
	National Highway
	River
* 2	*NSD Studied*
* [5]	*Proposed 10 NSDs*
* 1	*ESD Studied*

DESIGN MANUAL

FOR

THE SANITARY DISTRICT WATERWORKS

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1. Small Community Water Supplies
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CHAPTER I. INTRODUCTION

1.1. Purpose of the Design Manual

This design manual (hereinafter referred to as the Manual) is prepared to establish the design standards and to indicate the specific matters to be carefully studied for successful physical planning and designing of the Sanitation District water works in Thailand, which will ensure to supply quality and safe domestic water with consumers at the cost as low as possible.

In other respect, the Manual aims to be the technical guideline for the engineers and experts to be engaged in the water works as well as to be the standards for the appropriate design to the local conditions.

1.2. Conditions to Apply the Manual

The Manual shall cover the design standards for the water works with capacity to serve the population ranging from around 2,000 to 20,000, in taking into account the present conditions of the existing Sanitary District water works facilities in the Northeastern Region and the results of a series of discussions held among the officials in charge in the Public Works Department (PWD), the Provincial Water Works Authorities (PWA) and the study team of the Japan International Cooperation Agency (JICA). The Manual, therefore, should be appropriately utilized with full knowledge of various conditions mentioned below.

- The Manual shall be applied to construction of new Sanitary District (NSD) water works or expansion of existing Sanitary District (ESD) water works.

- The Manual discusses the basic concept of the physical planning and the design of the facilities for the water works in the SDs, and the comments or explanation of the detailed criteria of various hydraulic structures and the related theories shall be left intact. Such detailed items and theories should be studied with the reference books and publications and standards/criteria each specific field.

- The Manual is prepared in keeping the technical level for the university/college graduates in majoring water works engineering or those who are well-experienced equivalently to the above.

- A variety of figures and values found in the Manual are quoted on the average basis and can be adequately revised to those which meet the local requirements and the existing data and information available.

- The Manual is prepared in due consideration of the technology of the water works that can minimize the costs of construction/installation and O & M of the facilities and can be applied to the similar natured projects in the similar scale in Thailand. The Manual, therefore., might be revised appropriately in future as the technological innovation advances.

CHAPTER II. BASIC CONCEPTS ON PLANNING AND DESIGNING OF WATERWORKS

2.1. Objectives of Waterworks Projects

The objective of waterworks projects can be summarized as follows; and the persons or personnel concerned with the waterworks projects should always keep the objectives in mind for successful promotion of the projects.

1) Eradication of water-borne and water-related diseases.

2) Uplevelling of standard of living and promotion of development of industries and rural society.

3) Alleviation of burden by water fetching and increase in employment opportunities.

4) Ensuring to reserve the water for hydrants and to do domestic fire fighting.

2.2. Outline of Waterworks Projects

2.2.1. General

Successful planning and implementation of waterworks projects will require due consideration on local inhabitants' needs, project components and scale of the related regional development plan, time table of implementation as well as careful study on an optimum plan available in long-term financing and technology to be applied.

Following are those works which will be required prior to project implementation so as to ensure smooth promotion of the project.

1) Study of the basic plan and feasibility of the project.

2) Detailed design of the facilities and planning of construction.

3) Planning of operation and maintenance (O and M) of the facilities.

4) Financial planning.

2.2.2. Basic Plan of Project

The PWD or any alternative of it will prepare, as a rule, a basic plan of a project, when applications are made on execution of a waterworks project from the representative of the SD and the relevant beneficiaries. And the PWD or authorities concerned should establish a financial plan for carrying out the project. The aforesaid basic plan should cover the following matters:

1) Determination of the Project Area or Service Area.

2) Determination of the target year and projection of the population served by the Project.

3) Determination of the design water demand.

4) Selection of water sources by surface water or groundwater, in taking into consideration quantity and quality of the raw water as well as cost for investment and O and M works for facilities.

5) Establishment of optimum plans for those works of water intake, transmission, treatment and distribution.

6) Establishment of a plan of O and M of the facilities.

7) Estimation of the Project cost and establishment of financial plan.

8) Feasibility study of the Project and establishment of the necessary water tariff.

2.2.3. Preparation of Implementation Plan

1) General

When the basic plan of the Project, referred to in the previous section, is found to be feasible and required urgently, it is made to be a rule follow the procedures as below.

2) Detailed design of the facilities and preparation of the tender documents.

The major works required are as follows:

- Surveying, hydrological study and geological survey to be required for those facilities proposed in the basic plan.

- Review of preliminary design of the facilities.

- Detailed design of the facilities.

- Estimation of work volume and necessary construction costs.

- Preparation of the specifications and tender documents for construction works.

- Bidding, bid evaluation and conclusion of the contract with successful contractors.

3) Construction works

The construction works of the proposed facilities shall be carried out on the contract basis in accordance with the relevant contract and the project implementation plan.

4) Test operation and hand-over of the facilities

The facilities, when completed in construction, shall be successfully handed over to the employers from the contractors after careful inspection, trial operation and deliberated guidance to the employers' staffs on the O and M services.

2.2.4. O and M plan for Facilities

The detailed plan of O and M of the facilities should be prepared for successful O and M services, as the construction works made a progress. The plan shall cover the following works.

* Establishment of the O and M plan
* Training of O and M experts
* Employment of necessary staff
* Preparation of procurement plan for water treatment equipment and materials
* Preparation of O and M criteria
* Budgeting for O and M cost, etc.

2.2.5. Financial Plan

The financial plan of the Project shall be prepared according to the recapitulated budget proposed in the basic plan.

1) Allocation of the Project cost and ensuring the financial source for the cost to be allocated to the executing body.

2) Application of the Governmental subsidy.

3) Preparation of financing plan by loaning and its repayment plan.

4) Preparation of the O and M plan of the project facilities and budgeting.

5) Establishment of the tariff of the water charges.

6) Preparation of financial plan of the service construction cost to be allocated to the beneficiaries.

2.3. Outline of Waterworks Facilities

2.3.1. Conditions to be inevitably provided with Waterworks Projects

General considerations to be required for planning waterworks projects are given as follows:

1) Consensus and cooperation of beneficiaries

The requirements of the inhabitants in the Project Area shall be reflected in the Project as well as their good understanding and cooperation to the Project shall be confirmed. And the inhabitants will be required to take positive part in the project planning, construction, operation and maintenance of the project works.

2) Long-term plan of project

A long-term plan of the Project should be prepared in taking into account the harmonious relationship of the subject project facilities at the target year with any regional comprehensive projects or other water-related projects and the expansion/ improvement of the facilities provided by the subject project for a long term after the target year.

3) Physical planning in balance

The physical planning should be made so that the facilities as a whole can be in good balance with the related topography and regional environment as well as the plan can be economically and technically feasible.

4) Easiness in O and M of facilities

Every facility shall function well and the O and M works should be carried out safely and easily. And the facilities should be able to supply the water efficiently for effective water use.

5) Economic and financial feasibility of the Project

The proposed water charges are required to fully cover the project cost and the necessary O and M cost, and also should be within a range of the payment ability of beneficiaries. The Project shall be totally evaluation for sound operator of the Project.

2.3.2. General Descriptions of Waterworks Facilities

The facilities for the SD waterworks projects or those in similar scale should include the following items:

1) Water source facilities and intake facility

The water source facilities can be specified as follows by surface water or groundwater, and the optimum physical planning of the water source should be made in taking into account designed amount of water supplied, meteorological and hydrological conditions.

Surface water	Intake from rivers	Natural intake	Intake weirs, Intake facilities,
		Pumping intake	Pumping stations, Canal
	Intake from reservoirs	Natural intake	Const'n or rehab. of storage dams
		Pumping intake	Intake works, Pumping stations
Ground water	Intake from ground water	Pumping intake	Wells, Pumping stations

2) Transmission facilities

The transmission facilities will serve to convey the raw water from intake works to treatment plants through open canals or pipelines. The transmission facilities should be planned in the optimum scale and dimensions from the viewpoint of economic feasibility, design discharge, topographical condition and effective O and M services.

3) Water treatment plant

According to the result of quality analysis of the proposed water source, the optimum physical plan of the water treatment plant should be determined among the following combination, in due consideration of the present treatment technology available, topographical conditions and economic feasibility of the plant.

- Facilities for surface water intake receiving wells, chemical dosing, rapid mixing basin, flocculation basin, sedimentation basin, rapid (slow) sand filter, chlorination wells, and other appurtenant facilities.

- Facilities for groundwater intake receiving wells, chlorination well, aeration facilities, sand filter and other appurtenant facilities.

4) Distribution facilities

The distribution facilities will serve to convey the treated water to consumers safely and stably from time to time, consisting of the following structures and equipment.

Distribution reservoir, distribution pump station, elevated tank, distribution pipeline networks, service pipes/taps to be provided by beneficiaries.

CHAPTER III. PROJECT PLANNING

3.1. General

Importance and meaning of the basic plan formulation of the proposed Project are detailed in the previous chapter. This chapter discusses the basic concept and methodology of the design works of the waterworks projects, centering around the technical/engineering fields. The financial and economy of the Project will be handled in the other guidelines concerned.

3.2. Study of Proposed Dimensions

3.2.1. Determination of Proposed Service Area

The service area at the target year of the Project should be determined in due consideration of the following conditions:

1) The service area shall extend within the sanitary district in principle.

2) A study shall be made on the relationship of the subject Project with urban development projects and/or land use plans.

3) Economy of the facilities shall be studied in taking into account distribution and density of public utilities/facilities, commercial/industrial districts and residential areas.

4) The service areas shall be determined with prior consensus of the inhabitants (opinion of the SD Board).

3.2.2. Establishment of Target Year

The target year of the Project, in principle, shall be set the year by a decade (10 years) after the initiation of the subject Project. When, however, local conditions would not allow to do so, it could be set by more than 10 years after the initiation. In such case, a staged development plans shall be made in view of effective investment.

3.2.3. Projection of Population Served

According to the existing statistic population data available, the estimated population served within the proposed service area can be expressed as follows by the following equation in taking into account the division of the service area, target year and the total project plan.

$$Y = Yo\ (1 + X)^n$$

Where,

Y	:	Population projection at the target year
Yo	:	Present population
X	:	Average population growth rate per annum
n	:	Number of the year up to target year

When the aforesaid equation cannot be applied in considering a trend of urban development in future and current status of the regional development, any other method available can be used. (Refer to Appendix-A). And if there are any data available in population projection by any development projects in the subject Project area, the said value can be employed through a thorough review can be estimated based on the projected population at the target year.

The population to be served by the subject Project can be estimated from the prospected total population and the water supply rate at the same year. Overestimation of the water supply rate will result in decrease in benefit in respect to collection of water charges although the higher water supply will bring the investment to be more economically feasible. Under the circumstances, the design water supply rate shall be arranged around 60 through 70 percent.

3.2.4. Determination of Water Amount to be Supplied

1) Definition of Terms Used

The water amount to be supplied is specified into two as follows in the Manual.

Water Consumption

Water consumption referred to in the Manual is defined as the water amount to be consumed by beneficiary inhabitants for drinking and other miscellaneous domestic use, by public facilities, by industrial and commercial organization and by any other purposes, and can be expressed by per capita for the beneficiary inhabitants.

Water Demand

The water demand referred to in the Manual is defined as the total water amount to be consumed as treated water and can be expressed by per capita amount to be supplied for the total amount of the water consumption and losses in the course of distribution/uneffective discharges. (leakage/wastage losses).

The water demand can be specified into average daily demand, maximum daily demand, maximum hourly demand, etc. in its expression.

2) Calculation of water demand

a) The average daily water demand in the Manual shall be quoted in a range from 100 to 120 lcd in principle. In this case, the ratio of leakage/wastage losses to the water consumption shall be taken by 0.20-0.25 (as target value) for calculation.

When the water demand cannot be calculated with the aforesaid target value, the calculation shall be made by accumulation of water demand by respective consumers. For such calculation, the reference values of water consumption by various consumers are tabulated in Appendix B.

b) Maximum daily water demand shall be set by 1.20 to 1.50 times as much as the average daily water demand.

c) Maximum hourly water demand shall be set by 1.50 to 2.0 times as much as maximum daily water demand.

3.3. Study of Water Sources for Waterworks

3.3.1. Prerequisites of Water Sources for Waterworks

The water sources for waterworks should be selected with careful study of availability of the water source at the target year to the subject Project area in keeping it in mind how to stably supply the safe and low cost water all the year-round.

In general, the water source for the waterworks in the scale of the existing sanitary district, where waterworks facilities are existed, can be selected among those mentioned below.

Surface Water Sources:	River water, lakes, ponds, reservoirs
Groundwater Sources :	Springs, groundwater

The surface water sources show characteristic features to have seasonal fluctuation in runoff discharge by natural environment and meteorological conditions prevailing in the related catchment area. Furthermore, the water quality will change depending upon soil conditions and prevailing land use in the catchment area. The surface water sources, however, will be the promising sources in Thailand with clearly divided two seasons, the dry and the wet, if the reservoir facilities can well control the stored water in harmoniously with runoff discharge and water consumption.

On the other hand, the groundwater sources are superior in water quality to the surface water in many cases, and lower in construction cost of the facilities than that of the surface water sources.

The groundwter sources, however, cannot be expected much in the yield to meet the demand and cannot play solely a role as the water source in this kind of waterworks project.

Table 3-1. Main Difference between Surface and Ground Water

Characteristic	Surface Water	Ground Water
Temperature	Varies with season	Relatively constant
Turbidity, suspended solids	Level variable, sometimes high	Low or nil
Mineral content	Varies with soil, rainfall, effluents etc.	Largely constant, generally appreciably higher than in surface water from the same area.
Divalent iron and manganese (in solution)	Usually none, except at the bottom of lakes or ponds	Usually present
Aggressive carbon dioxide	Usually none	Often present in large quantities
Dissolved oxygen	Often near saturation level	Usually none at all
Ammonia	Found only in polluted water	Often found, without systematically indicating pollution
Hydrogen sulphide	None	Often present
Silica	Moderate proportions	Level often high
Nitrates	Level generally low	Level sometimes high; risk of methemoglobinaemia
Living organisms	Bacteria (some pathogenic), viruses, plankton	Ferrobacteria frequently found

Source: Water Supply Engineering, 1984, JICA

3.3.2. Surface Water Sources

1) Data collection

A successful study of the surface water sources will require collection/analysis of related data available, field investigation physical planning for the water sources, necessary water balance computation, etc.

The data required for the study, although different in kinds by locally specific features of the Project area, commonly include the following:

- Topographical maps of the Project area.
- Maps showing existing dams and rivers.
- Meteorological data covering the Project area.
- Hydrological data covering the Project area.
- Present status of water utilization by means of existing water source facilities and water storage facilities and inventory books for existing facilities.
- Demand of drinking water and miscellaneous domestic water, and other demand for irrigation, industries and miscellaneous uses.
- Any other data and information related to the water source development planning.

It is desirable to collect the aforesaid data/records as many in kind wide in coverage as possible. In particular, the hydrological data of rivers with small catchment areas can be found rarely with the observation values, and consequently it is desirable to collect the necessary data at the relevant RID Regional Offices for the subject Project area and its peripheral areas.

2) Field investigation

According to the results of data analysis made along the guideline mentioned in the previous paragraph, a proposed water source shall be selected in view of water quality and quantity available and a thorough study shall be made on the present status of the subject Project area through field investigation. At the same time, supplemental data collection shall be carried out together with surveys by two times on the related river discharge or reservoir operation in the dry and the rainy season.

3) Meteorological and hydrological analysis

a) Determination of standard drought year

The waterworks project is generally designed based on hydrological condition of a standard drought year, which has a re-occurrence interval of once in ten years.

b) Runoff discharge

The storage water of reservoir is supplied by runoff discharge from catchment area, which is affected by many factors such as topographic condition, vegetation of catchment area and rainfall intensity. As a general rule, if measured data are available, the river runoff is estimated by the mathematical model such as "Tank Model Method". However, due to lack of measured data for subject water sources, monthly total runoff discharge from catchment area is computed by using the RUNOFF ESTIMATION CHART prepared by Project Planning Section, RID and/or another manners. (Refer to Fig. 3-1.)

c) Water losses

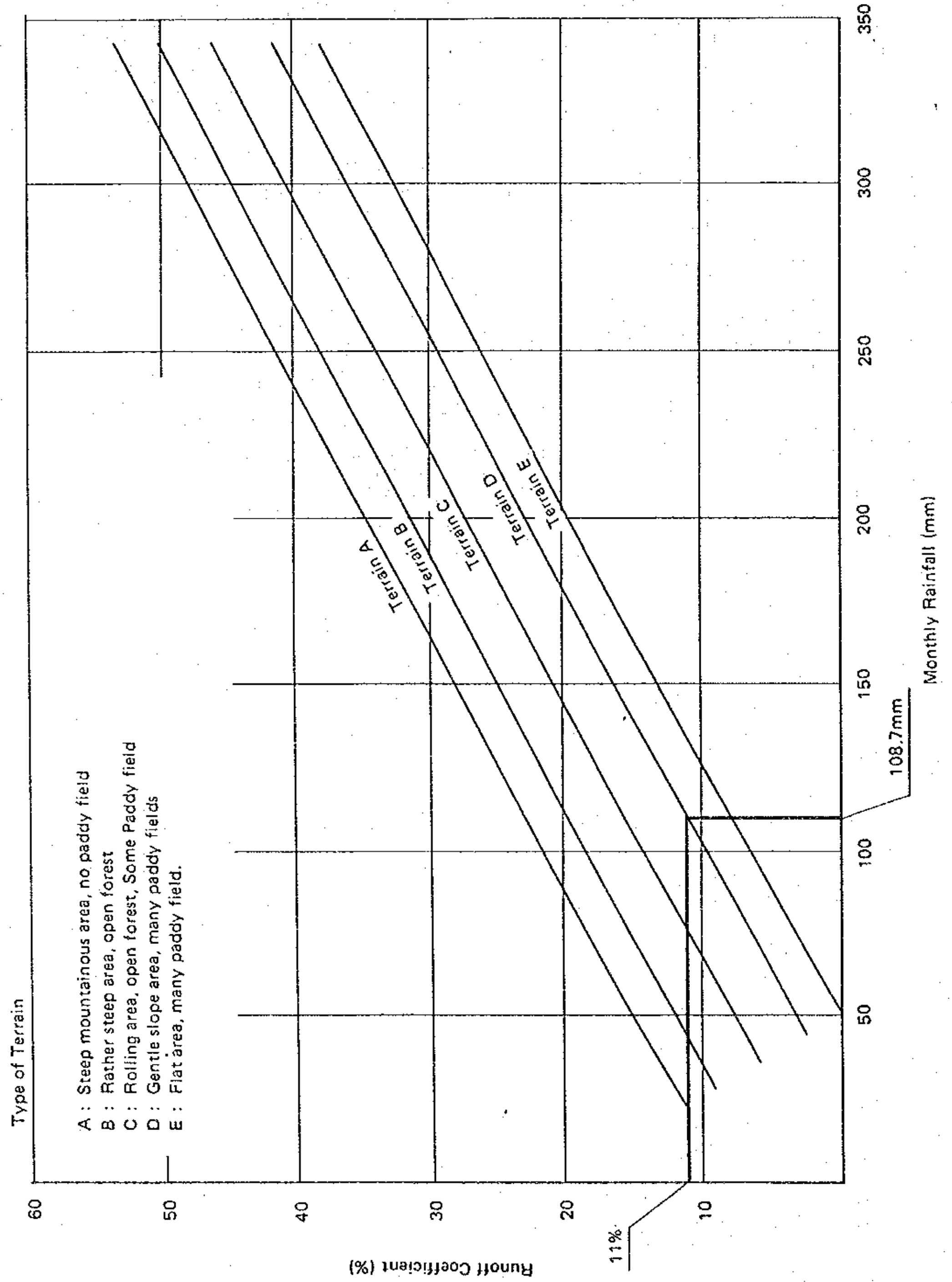
Major water loss from reservoir is evaporation loss from water surface. The loss is estimated on a monthly basis in multiplying 70 percent by the observed figures from Class A-pan evaporation device throughout the year.

d) Water demand or requirement from various sectors

Drinking water: The monthly water demand shall be estimated in using water demand obtained in para. 3.2.4, population served and coefficient of monthly water consumption.

Irrigation water: The monthly water requirements can be estimated by deducting the effective rainfall from unit water requirement obtained based on present cropping acreage, cropping pattern.

Fig. 3-1. Runoff Estimation Chart



Source: Project Planning Section RID.

3.3.3. Groundwater Source

1) Data Collection

The survey works of the groundwater sources include collection/review of data of existing wells, related topographical maps, geological maps, and field investigation, electric prospecting together with test well drilling, pumping test and water quality test, etc.

Prior to field investigation of the subject Project area the related topographical maps and geological maps should be collected for review and analysis, and as for the existing wells provided by PWD, DH, DMR, etc., geological logs, and the data of water quality tests, yield tests, etc. should be collected and reviewed.

2) Field investigation and test well drilling

a) General

The field investigation works are composed mainly of four steps a geomorphological reconnaissance survey, a geoelectric prospecting, a well drilling and a pumping test.

First of all, each district will be surveyed geomorphologically and hydrogeologically, then a suitable geoelectric prospecting station will be determined so as to make a sufficient investigation on hydrogeological condition of the district.

However, it is difficult to estimate availability of ground water by those indirect prospecting.

The availability and the field potential of the aquifer should be carefully studied by such direct means as well as drilling and pumping test.

b) Geoelectric prospecting

i) Purpose

Geoelectric prospecting shall be conducted to grasp an aquifer structure and to get a basic data available drilling wells and planning pumping test.

ii) Method

- Selecting a target area

  A target area and prospecting points are selected by field reconnaissance and review of collected data.

- Geoelectric prospecting

  Kind of prospecting ... Vertical prospecting by "Wenners or Schlumberger arrangement"

  Target depth ... 100 m as a rule

  Measuring interval ... see Appendix C, Data sheet.

  Prospecting points .. 20 points/ district at least.

iii) Analysis

Rough analysis ... by "Direct reading method" and "Standard curve method"

Final analysis ... by "Curve fitting method" using computer

Results ... (i) ρ -a curve with analysis log.
(ii) structural profile

c) Well drilling plan

i) Drilling method

Any kind of drilling method (a percussion type or rotary type) is available if it has an ability to drill through a rock formation more than 100 m in depth with 300 mm drilling diameter.

ii) Drilling diameter and depth

Drilling diameter should be 250 mm or more, and the drilling depth should be 60 m for all districts.

iii) Geological log

A geological log shall be taken as exactly as possible for test well drilling, and consequently, a percussion drill is more recommendable than a rotary drill.

iv) Electric logging

Electric logging test should be made in each well soonest after drilling to the designed depth.

d) Pump test plan

i) Preliminary pumping test

The preliminary pumping test shall be carried out for keeping records on pumping rates, which can serve the following drawdown test, and also the preliminary test aims to confirm the settlement of all the set of the test equipment prior to the full scale main test.

ii) Drawdown test

The pumping tests at each step shall be carried out based on the information obtained by the preliminary test. The rates should be generally estimated as to course drawdowns of 5, 10, 15, 20 and 25 m or 30 m when the water level is high.

iii) Main test

After estimating safety yield of the well the main pumping test (continuous pumping test) should be conducted by the rate of safety yield or 50 percent up of the safety yield but should never exceed a limit yield.

3.3.4. Water Quality

1) Selection of water source and treatment process

The treatment method shall be selected in the assumption that the raw water would be in the most critical condition in quality. Tables 5-1 and 5-2 show the basic criteria for the selection.

There are two ways in water quality analysis: field test by portable equipment and laboratory test.

In the field tests quality is examined on pH, turbidity, taste and odour for many proposed water source. In the laboratory tests, detailed analysis is carried out with sampled water based on the Thai Standards of water analysis.

2) Quality control of groundwater

a) The field tests of groundwater available shall be made on the following items:
pH, taste and odour, turbidity, color, chloride, iron

b) Laboratory tests
The major test items are:
pH, turbidity, color, iron, manganese, hardness, hydrogen sulfide, carbonate acid.

3) Quality control of surface water

a) The field tests of surface water available in existing reservoirs, ponds and rivers are carried out on the following items:
pH, turbidity, color, chloride, iron, taste and odor.

b) Laboratory tests
The major test items are:
pH, turbidity, color, chloride, iron, manganese, alkality, $KMnO_4$ residence.

4) Drinking water quality standards in Thailand

The water quality standard in Thailand is shown in Table 3-2.

Table 3-2. Water Quality Standard in Thailand

Physical Condition

Item	Highest Desirable Level	Maximum Permissible Level
colour	5	15
taste	Unobjectionable	Unobjectionable
odour	Unobjectionable	Unobjectionable
turbidity	5	20
PH range	6.5 to 8.5	Under 9.2

Chemical Condition

Item	Highest Desirable Level (ppm)	Maximum Permissible Level (ppm)
total solids	500	1,500
Fe	0.5	1.0
Mn	0.1	0.3
Fe + Mn	0.5	1.0
Cu	1.0	1.5
Zu	5.0	15
Ca	75	200
Mg	50	150
So	200	250
Cl	250	600
F	0.7	1.0
NO_3	45	45
alkylbenzyl sulfonates, ABS	0.5	1.0
Phenolic-substances, as phenol	0.001	0.002

Remarks : Total Hardness (as Calcium Carbonate) less than 300 ppm is defined to a good water as standard.

Toxin

Item	Highest Desirable Level (ppm)
Hg	0.001
Pb	0.05
As	0.05
Se	0.01
Cr Hexavalent	0.05
CN	0.2
Cd	0.01
Ba	1.0

Bacteriological Condition

Item	Highest Desirable Level
Standard Plate count (Colonies/cm^3)	500
MPN (Coliform Organism/100 cm^3)	less than 2.2
E. coli	none

*/ : Data Source PWD.

The water quality standard, WHO and Japan can be referred from Appendix D.

3.3.5. Water Balance Simulation and Evaluation

1) General

For every water source to be proposed, the water balance computation shall be carried out according to the following conditions so that safety yields/runoff discharge available can be estimated and the necessary construction/improvement of the facilities can be proposed appropriately.

- To confirm the vested water right of the surface water sources and the priority in the said sources.

- To reconfirm the water demand of not only water works but other uses.

- To make an improvement plan for the water source facilities of the multi-use.

- To estimate safety yields of groundwater and to make a proper layout of new wells.

- To make plan of combination use of groundwater and surface water as sources for short supply by groundwater only.

2) Simulation

The water balance computation of a reservoir can be made by the following equation:

$$V = I - W - IR - L$$

V : Storage volume of the reservoir
I : Inflow into reservoir

W : Water demand for the water supply

IR : Irrigation water requirement (if any)

L : Water losses from reservoir

In case of taking the existing ponds and/or reservoirs as sources of water supply, priority should be given to water uses when depending upon multipurpose water sources.

There are two types of reservoirs for water supply systems: one functions mainly as a source of domestic water supply with excess water, if available, used for irrigation purpose, and the other functions for irrigation purpose only. The reservoirs constructed under Small-Scale Irrigation Program (SSIP), originally function to supply irrigation water, can be used for domestic water supply in priority to irrigation if the water demand for domestic use should arise much in the downstream therefrom. It is natural that there should be excess water available in SSIP-reservoirs, such water can be used for any other purposes alike industrial or commercial. The first priority, however, shall be given to irrigation, when the water is used under those projects of Medium Scale Irrigation Program (MSIP), King's Projects and Tank Projects by RID.

The reviews and studies will be made on availability of the existing water sources along with the aforesaid criteria.

3) Evaluation

Based upon the results of the water balance computation several cases should be proposed as the references for the alternative studies mentioned in the next paragraph.

[Example 3.1]

Water Balance Simulation and Evaluation

SD Name	:	Kusuman
Water Source	:	Huai Daeng Reservoir
Storage Capacity	:	1,150,000 cu.m
Catchment Area	:	10.5 sq.km
Rain Gauge Station	:	Sakon Nakhon
Irrigable Area	:	160 Ha

1) Runoff analysis

The computation procedure is shown as follows:

- Determination of "Terrain Type"..Type "D"
- Reading the runoff coefficient in Fig. 3-1
- Computation of runoff discharge:

Runoff Depth = Monthly Rainfall x Runoff Coefficient
= 108.7 x 0.111
= 12.1 mm

Monthly Inflow = Depth of Runoff x Catchment Area
= 12.1 x 10.5
= 127.1 x 10^3 cu.m

The following table is the result of runoff calculation.

Monthly Inflow at Huai Daeng Reservoir

Month	Rainfall (mm)	Runoff Coefficient (%)	Runoff Depth (mm)	Inflow (1,000 cu.m)
Apr.	108.7	11.1	12.1	127.1
May	214.9	24.6	52.9	555.5
Jun.	148.1	16.1	23.8	249.9
Jul.	210.7	24.1	50.8	533.4
Aug.	383.1	46.0	176.2	1,850.1
Sep.	108.3	11.1	12.0	126.0
Oct.	78.7	7.3	5.7	59.9
Nov.	0	0	0	0
Dec.	14.4	0	0	0
Jan.	0	0	0	0
Feb.	0.8	0	0	0
Mar.	0.2	0	0	0
Total	1,267.9	26.3	333.5	3,501.9

2) Computation of Water Balance

The water balance computation of a reservoir is expressed by the following equation.

New (1) = (1) + (1') - (2) - (3) - (4)

If the value of (1) is larger than the storage capacity, the value of (1) is changed to storage capacity.

Water Balance Simulation of Huai Daeng Reservoir

	Volume(1)	Inflow(1)	Water Works(2)	Irrigation (3)	Loss(4)
Nov.	1,150.0	0	24.0	0	102.5
Dec.	1,023.5	0	25.0	0	93.6
Jan.	904.9	0	23.0	0	112.1
Feb.	769.8	0	27.0	0	95.1
Mar.	647.7	0	28.0	0	119.4
Apr.	500.3	127.1	31.0	0	61.0
May	535.4	555.5	27.0	0	65.7
Jun.	998.2	249.9	29.0	156.0	65.7
Jul.	997.4	533.4	22.0	182.1	38.1
Aug.	1,150.0	1,850.1	23.0	7.0	49.0
Sep.	1,150.0	126.0	21.0	391.8	62.6
Oct.	800.6	59.9	20.0	475.8	54.4
Total		3,501.9	300.0	1,212.7	919.2

3.4. Preliminary Design of Facilities and Project Cost

3.4.1. Preliminary Design of Facilities

The objectives of preliminary design of the facilities in the basic planning are summarized as follows:

1) The study shall be made on the Project area, population served and proposed water source available so as to establish several alternative plans and to clarify the technical feasibility of the Project.

2) The alternative plans technically feasible shall be studied on their construction costs and O and M costs so as to select the physical plan that is most feasible economically.

3) Based on the aforesaid results of the alternative studies, the preliminary design shall be carried out with the preciseness that will not require any change or deviation in basic points for implementation.

3.4.2. Selection of Alternative Plans and Optimum Plan

1) Alternative Plan

For the waterworks depending upon the surface water sources, in general, optimum planning and designing of facilities for water sources, intake, transmission in view of both technology and economy will give considerably large effect on work volumes for construction and O and M of facilities. In the case that the surface water is used as a single source, it will be sufficient to give considerations on the above-mentioned facilities for alternative plans.

When taking the groundwater as a single source, the locations of the proposed new wells will give considerable effect, and alternative plan formulation should be studied on the method of raw water collection, transmission of treated water, etc. In other words, one way is to treat the raw water at one treatment plant in concentration after conveyed from several wells or the other is that each well shall have its own treatment plant.

In case of combination sources of surface water and groundwater, optimum plans for independent sources and their combined plans shall be comparatively studied.

2) Optimum plan

The selected alternative plans that are found technically feasible shall be thoroughly studied to decide an optimum plan by means of comparison of costs for construction and O and M, including costs for necessary energy, materials, laborers' wages,s and repairing.

3.4.3. Project Cost

The Project cost shall include those costs for survey, construction, engineering, administration, land acquisition, and contingency for successful physical planning, designing and implementation. The Project cost shall be the essential factor for establishing the financial plan, the tariff of water charges.

3.5. Implementation Program and O and M Plan

3.5.1. Implementation Program

An implementation program shall be established according to those factors of the scale and financial plan of the project. The major items to be contained in the program are survey, design, bidding procedures, implementation, trail operation of facilities, training of O and M staffs, etc.

3.5.2. O and M Plan

The O and M Plan shall be prepared to establish the O and M criteria of the Project facilities, establish an appropriate O and M organization, employ the persons in charge of O and M works, procure the necessary equipment and materials, collect water charges, so that the stable supply of high quality water can be secured through the proposed waterworks. And the Plan shall be submitted to the higher authorities concerned for their approval.

3.6. Financial Plan and Project Evaluation

3.6.1. Financial Plan

A study of the Project from a financial viewpoint is as essential as a study of the Project implementation.

In the waterworks projects, the necessary project cost in principle shall be covered by the water charges to be collected, although sometimes the government subsidy might be given to them. And the major items to be considered in the financial planning are as follows:

- Government subsidy
- Fund to be available by SD itself
- Loan from financing agencies
- Contributions and others

3.6.2. Project Evaluation

According to the financial plan prepared based on the previous paragraph, the project financial balance shall be studied in taking into account the yearly allocation of the Project cost, repayment conditions of the loan, O and M cost, etc., as one hand and the revenue of water charges as the other.

The water charges should be decided within an adequate range of payment ability of the beneficiaries, and a special consideration should be given to the financial balance so as to be stabilized in the early stage after Project completion.

CHAPTER IV. INTAKE AND WATER TRANSMISSION WORKS

4.1. General

4.1.1. Water Sources

A careful review shall be made on the basic plan proposed to confirm the suitability of such items to the physical plan as water amount available and water quality of the proposed source. In other respect, the detailed survey shall be carried out on the topography and geology of the proposed facilities construction sites.

When the improvement works are required on the existing water source facilities, the closest cooperation shall be made with the O and M office of the relevant facilities or the related agencies. The technical guideline for the improvement works to be required shall be prepared along with the instructions by the said O and M office or the results of the consultation with that office.

4.1.2. Design Intake Discharge

Design discharge at the intake point is to be 110 percent of the maximum daily water demand. The volumes of 10 percent of the maximum daily water demand is the amount of water required for washing filtration plant, and water leakage/wastage losses between intake facilities and water treatment plant.

4.2. Intake Method

4.2.1. Surface Water

There are two ways of water intake from the surface water sources; one is to take river runoff water directly and the other the stored water in the reservoirs.

The water intake shall be, in principle, by gravitational intake method so far as conditions can permit, although may be changed in the lowest water level at the intake point, location of the treatment plant, topography along the proposed channel routes.

There are two methods of gravitational intake and pumping intake in the direct water intake from the river runoff.

In employing either method of the two, the necessary facilities shall be planned so as to correspond as effectively as possible to considerably large fluctuation in the water level between flooding and drought seasons.

In case the water is taken from the storage facilities like reservoirs, lakes/ponds, etc., there are two ways to provide the intake facilities within a reservoir area or to utilize the existing intake facilities for water intake.

In conveying water to a treatment plant after taking water by gravitational method, the intake facilities shall be composed of·intake controlling sluice gates, etc. there are hydraulically advantageous in due consideration of preventing sand and silt inflow.

For pumping intake, the physical planning should be made in taking into consideration the safety intake, cavitation of pumps, moisture-free electric parts, etc.

4.2.2. Groundwater

Submerged pumps are used for groundwater intake, in principle, and Fig. 4-1 illustrates the typical structure of deep well.

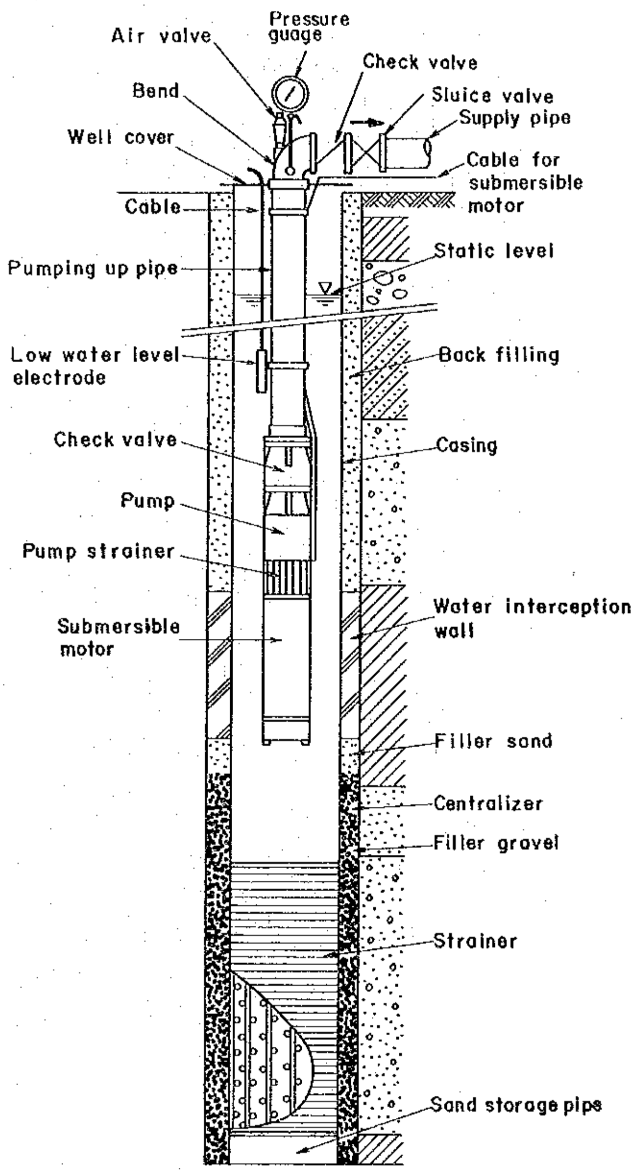


Fig. 4-1. Structure of Deep Well (Ref. Book No.5)

4.3. Diversion Weir

The diversion weir proposed for the subject Project shall be planned to divert the river water in the flooding season so as to supplement the reservoir water. The diversion weir should be planned not only for safety intake of designed water but also with hydraulically safe and sound as river structures to meet the requirements stipulated in the related design standards or criteria. The design standards prepared by RID can be referred to for designing of the said weirs.

4.4. Intake Pump

4.4.1. Classification of Pumps

The pumps to be used for intake from surface water sources and groundwater sources can be classified by local conditions and purposes.

Table 4-1. Classification of Pump

Kind	Charateristics	Classification	Purpose
Centrifugal pump	*More excellent than the other type of pump from operation, principle, structural point of view *Widely used	*Volute *Turbine *Submersible *Single suction *Double " *Single stage *Multi stage	*Intake *Distribution *Booster *Back wash *Surface "
Axial flow pump	*Low head *Large discharge		*Low head intake
Mixed flow pump	*Large discharge		*Intake *Drain

For surface water intake, the volute type single suction horizontal axis pumps shall be used in principle in taking into consideration the designed discharge, water sources, water conveyance, etc.

If the said type pumps cannot be employed due to the fact that the pumps cannot meet the local conditions, those of vertical or inclined shaft type pumps shall be adopted.

The pumps for groundwater intake shall be of submersible multi-staged centrifugal type.

4.4.2. Number of Pumps to be Required

For surface water source, two units of the pumps in the same kind and type shall be provided for one site. One of the two shall be diesel powered and used as stand-by unit. The operation for peak intake shall be considered to last 24 hours.

For groundwater intake, one unit of the pump shall be provided for each well but no stand-by unit provided.

The operation for the peak intake should be considered to last 20 hours at maximum for daily maintenance services, and the specifications of the pumps shall be determined to meet such requirements.

4.4.3. Preliminary Study on Specifications of Pump and Motor

The relationship between bore diameters of suction mouth and the discharges can be shown in Table 4-2. The output of the prime mover can be estimated by the following equation.

$$\text{Output of Prime Mover} = \frac{0.163 \times Q \times H}{\eta_t \times \eta_p} \times (1 + \alpha) \times \gamma \text{(KW)}$$

Q : Discharge (m^3/min)
H : Total lift (m)
α : Margin rate of capacity, Motor 0.15, Diesel engine 0.20
η_t : Transmission efficiency 0.90 - 1.00
η_p : Pump efficiency (Ref. to Fig. 4-2)
γ : Specific weight of water (1.00)

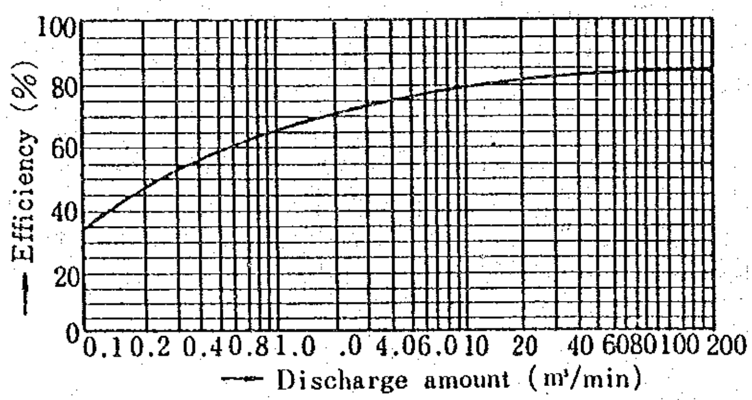


Fig. 4-2. Standard Efficiency of General Use Pump (Ref. Book No. 5)

The specifications of the proposed pumps and motors can be roughly grasped from Fig. 4-3.

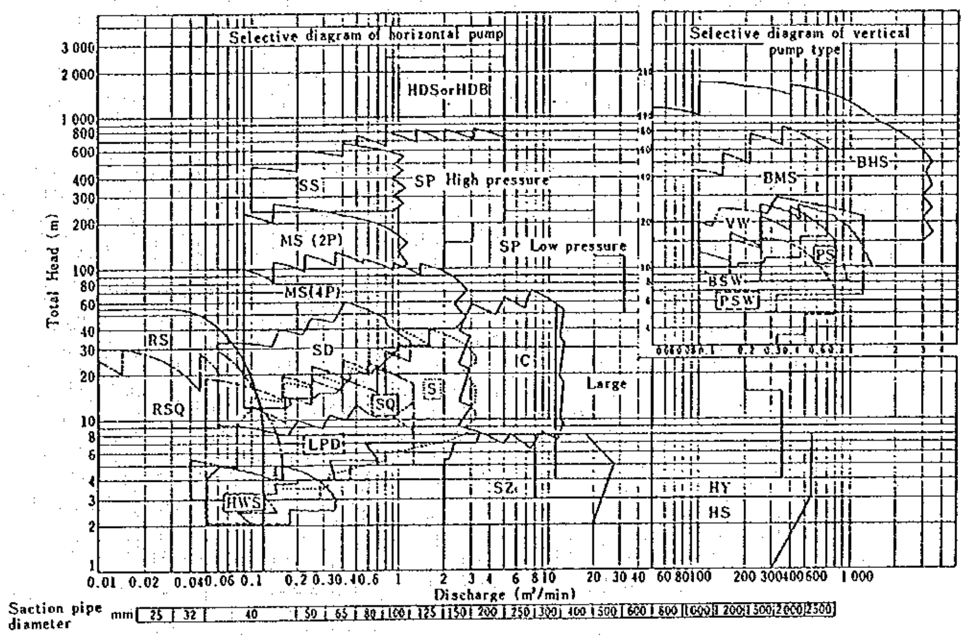
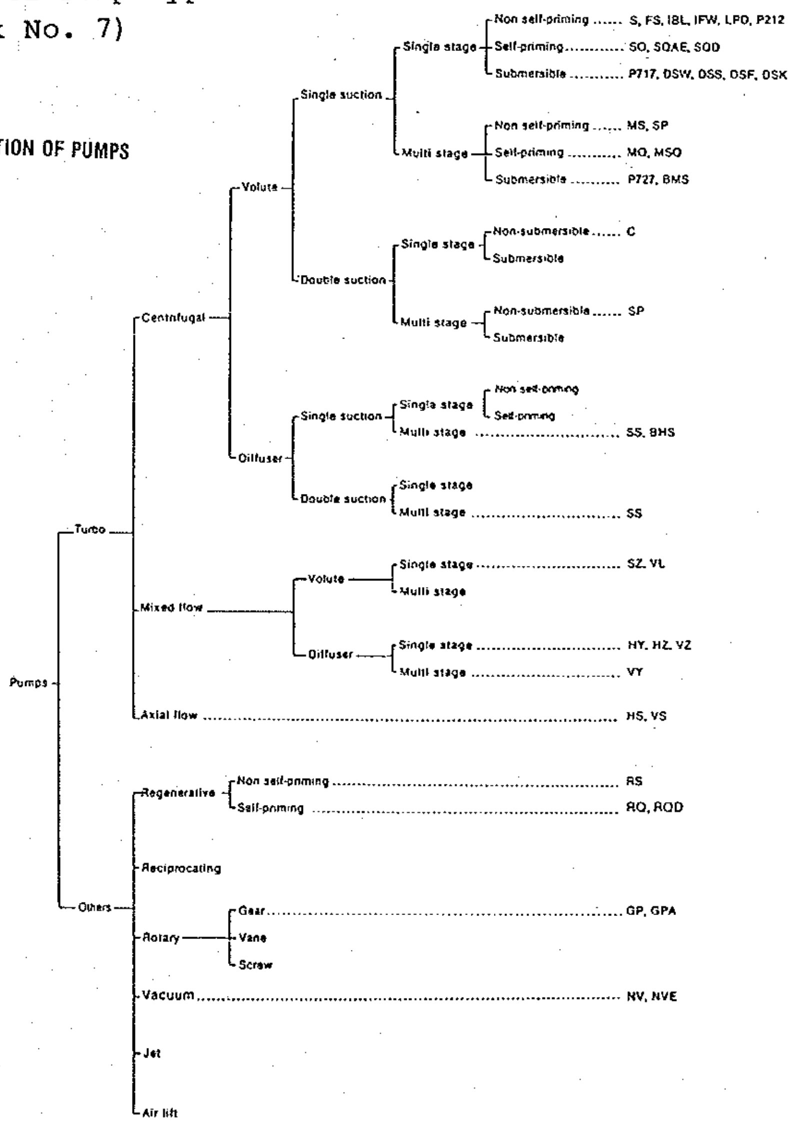


Fig. 4-3. Selection of Pump Type 50Hz
(Ref. Book No. 7)

CLASSIFICATION OF PUMPS



4.4.4. Design of Suction Tank

Successful works for design of a suction tank and installation of pumps will require the deep and sufficient knowledge on the kind and type of pumps, their operability, conditions of water level, etc., and the layout of these equipment shall be made in due consideration of easiness of O and M services. The suction pipes shall be as short as possible for preventing pumps from cavitation and keeping suction capability high. For installation of pumps and determination of the shape of suction tank, the following values can be referred to as standards. (Ref. to Table 4-2 and Fig. 4-4.)

Table 4-2. Suction Size and Discharge Capacity of Pumps

Suction Size	Discharge Capacity
40 mm	0.10 - 0.20 m^3/min.
50	0.16 - 0.32
65	0.25 - 0.50
80	0.40 - 0.80
100	0.63 - 1.25
125	1.00 - 2.00
150	1.60 - 3.15
200	2.5 - 5.0
250	4.0 - 8.0
300	6.3 - 12.5

Source: Japanese Industrial Standard (JIS)

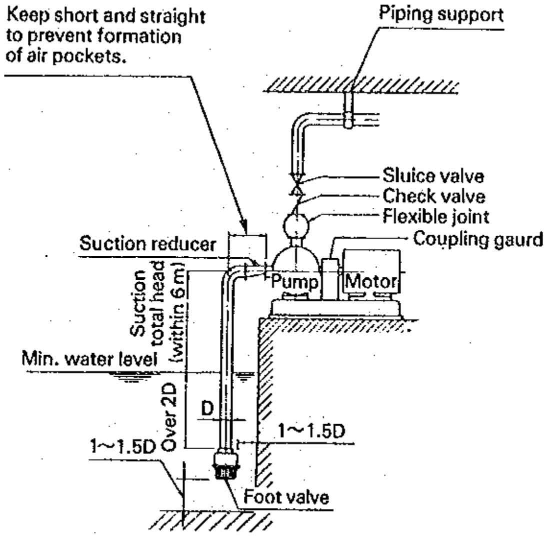


Fig. 4-4. Location of Pump and Suction Tank Layout (Ref. Book No.8)

4.4.5. Operationability of Pumps in Parallel

When two pumps with the same characteristics are operated in parallel, the discharge flow rate theoretically becomes double that in single pump operation for the same head point.

In fact, however, the discharge does not double because the pump is operated at the intersection between the parallel characteristic curve and the resistance curve. The effectiveness in parallel operation increases as the scope of the resistance curve decreases. (Ref. to Fig. 4-5.)

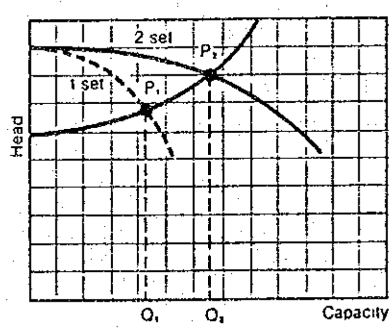


Fig. 4-5. Characteristic of Parallel Pump Operation (Ref. Book No. 8)

4.5. Water Transmission Works

4.5.1. Routing of Transmission Channels

1) Pipelines

The routing of the pipelines shall be carried out in taking into account the following points:

- to be as short as possible
- to lay pipes under roads or side walks in consideration of installation works and maintenance works
- to transmit the water by gravity system
- to provide O and M roads together with the pipelines in case of laying the pipes through farm land or forest/mountain area.

2) Open channels

In principle, it is desirable that the open transmission channels should be avoided for waterworks systems so far as conditions permit, so that the water quality can be well maintained by being free from containing foreign materials in the courses. And all the same considerations should be given to routing works of the open channels as those to the pipelines, where the open channels will be inevitably employed.

The open channels will be suitable for transmitting a large amount of water like compared with the pipelines system.

4.5.2. Laying Depth of Pipes and Foundation Treatment

The allowably shallowest laying of pipes is as follows:

For road : 1.00 m between pipe top and ground surface

For farm land : 0.80 m between pipe top and ground surface

Sand bed is required to avoid the pipes supported by point, wherever the pipe are laid.

4.5.3. Selection of Pipes by Kind and Type

Ductile cast iron, steel, asbestos cement, hard polyvinyle chloride, prestressed concrete are commonly used as materials of the pipes for transmission pipelines.

Section of pipes among such varieties shall be made in taking into account the necessary diameters, design discharges, economy, easiness in installation, etc.

4.5.4. Hydraulic Calculation

In hydraulic calculation, Williams-Hazen's formula is adopted for pipeline, while Manning's formula for open channel.

(a) Williams-Hazen's formula

$$V = 0.35464 \times C.D^{0.63} \times I^{0.54}$$
$$Q = 0.27853.C.D^{2.63} \times I^{0.54}$$
$$I = 10.666 \times C^{-1.85} \times D^{-4.87} \times Q^{1.85}$$

Where,

V = average velocity (m/s)
Q = discharge (m^3/s)
D = pipe diameter (m)
I = hydraulic gradient
C = velocity coefficient

The velocity coefficient of C will generally have a value by 100 for old pipes with rusty inner surface. And the C values in various kinds of pipes are shown in Table 4-3. The diagrams of Williams-Hazen's formula are shown in Appendix E.

Table 4-3. C-value of Pipes (Ref. No. 9)

Conduit Material	Age New	Age Uncertain
Cast-iron pipe, coated (inside and outside)	130	100
Cast-iron pipe, lined with cement or bituminous enamel	130*	130
Steel, riveted joints, coated	110	90
Steel, welded joints, coated	140	100
Steel, welded joints, lined with cement or bituminous enamel	140*	130*
Concrete	140	130
Wood stave	130	130
Cement-asbestos and plastic pipe	140	130

* For use with the nominal diameter, i.e., diameter of unlined pipe.

In hydraulic calculation, various head losses in addition to friction loss should be taken into account. And in hydraulic estimation, 10 percent of the aforesaid friction loss shall be added to the other head losses to make a total loss for the pipeline.

For pressurized pipeline systems, the optimum pipe diameters should be decided through estimation of the costs for pumping facilities and pipelines plus operation cost of pressurizing pumps. And the optimum pipe diameters, therefore, should be confirmed by various estimations.

(b) Manning's formula

$$V = \frac{1}{n} \times R^{2/3} \times I^{1/2}, \qquad Q = A.V.$$

Where,

V : Average velocity (m/s)
R : Hydraulic mean depth (m)
I : Hydraulic gradient
n : Roughness
Q : Discharge (cu.m/s)
A : Inner area of pipe (sq.m)

"n" values:
Earthern canal 0.030
Concrete lining 0.015
Concrete block 0.020

(c) Various kinds of head loss

$$h = f_i \times \frac{V^2}{2g}$$

Where, f_i : coefficient of head loss (Refer to Table 4-4).

Influent	f=0.5	0.25	0.1	0.01~0.05	1.0						
Effluent	1										
Expansion	D_1/D_2	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	f	1.00	0.98	0.92	0.82	0.70	0.56	0.41	0.26	0.13	0.04
Reduce	D_2/D_1	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	f	0.50	0.50	0.49	0.49	0.46	0.43	0.38	0.29	0.18	0.07
Bend	ρ/D	1			2			3			
	θ	22.5	45	90	22.5	45	90	22.5	45	90	
	f	0.10	0.15	0.20	0.08	0.11	0.15	0.05	0.07	0.10	
Sluice valve	S/D	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	
	f	300	110	25	10	5.5	3.2	1.8	1.0	0.6	
Reverse stop valve	0.6~1.5										

Table 4-4. Coefficient of Head Loss (Ref. Book No. 7)

4.5.5. Appurtenant Structures

The appurtenant structures shall be planned in paying attention to the following points:

- For piping, special attention should be paid to the position of pipes so that they are located longitudinally below the hydraulic grade line to prevent negative pressure in the pipe.

- Sluice valves should be provided at 1 to 3.0 km intervals to stop the water flow and to adjust the flow rate.

- Air valves should be provided on the convexes of the piping.

- Drainage facilities should be provided between the sluice valves, if possible.

CHAPTER V. WATER TREATMENT WORKS

5.1. General Description

5.1.1. Objective of Water Treatment

The water treatment can be defined as technical operation which fulfills any quality gaps existed between raw water and treated water by combination of such treatment systems such as chlorination, rapid sand filtration, slow sand filtration and aeration system.

These systems are made up of some unit operation, suspend matter sand and silt, dissolved matters, and colloidal matter which are removed from the raw water through individual physical, chemical and biological processes.

5.1.2. Preliminary Selection of Water Treatment Process

Water treatment process can be selected by the following criteria after sample raw water taken at the proposed intake sites is analyzed.

Table 5-1. Selection of Water Treatment Process

Water Source	Raw Water Quality (Maximum Annum)				Treatment Process
	PH	Turb.	Color	Fe (mg/l)	
Ground Water (A)	6.5-8.5	<5	<5	<1.0	Chlorination
Ground Water (B)	6.5-9.2	<30	<15	<3.0	Aeration + Rapid Sand Filter
Surface Water (A)	6.5-9.2	<15	<15	<1.0	Slow Sand Filter
Surface Water (B)	5.0-9.2	<1,000	<30	<5.0	Rapid Sand Filter

Note: 1) Table 5-1 is generally applied to select an adequate process from water source to treatment plant, and the table are rather preliminary standard.

In case that the results of water quality analysis are nearly equal or over than the indicated figures, by carrying out analysis such as jar test, it is necessary to judge whether the water can be treated for drinkable or not, its treatment process is economical or not in respect to chemical dosage and O/M is possible or not.

2) Rapid sand filter process requires pre-chlorination in some case or not require it in other case. Its selection depends on the raw water quality analysis and estimation of change of the water quality.

3) Selection of slow sand filtration process is based on the water quality analysis over a year in due consideration of contamination of the water.

Effectiveness of water treatment by respective methods is summarized in Table 5-2.

Table 5-2. Effectiveness of Water Treatment
(Ref. Book No. 1)

	Treatment Process					
Water Quality Parameter	Aeration	Chemical Coagulation and Floc.	Sedimentation	Rapid Filtration	Slow Sand Filtration	Chlorination
Dissolved Oxygen Content	*	o	o	-	--	*
Carbon Dioxide Removal	-	o	o	*	**	*
Turbidity Reduction	o	***	*	***	****	o
Colour Reduction	o	**	*	*	**	**
Tast and Odour Removal	**	*	*	**	**	*
Bacteria Removal	o	*	**	**	****	****
Iron and Manganese Removal	**	*	*	****	****	o
Organic Matter Removal	*	*	**	***	****	***

Remark : *** etc. = increasing positive effect
o = no effect
- = negative effect

Classification of typical water treatment process on both surface water and groundwater is illustrated in Figure 5-1.

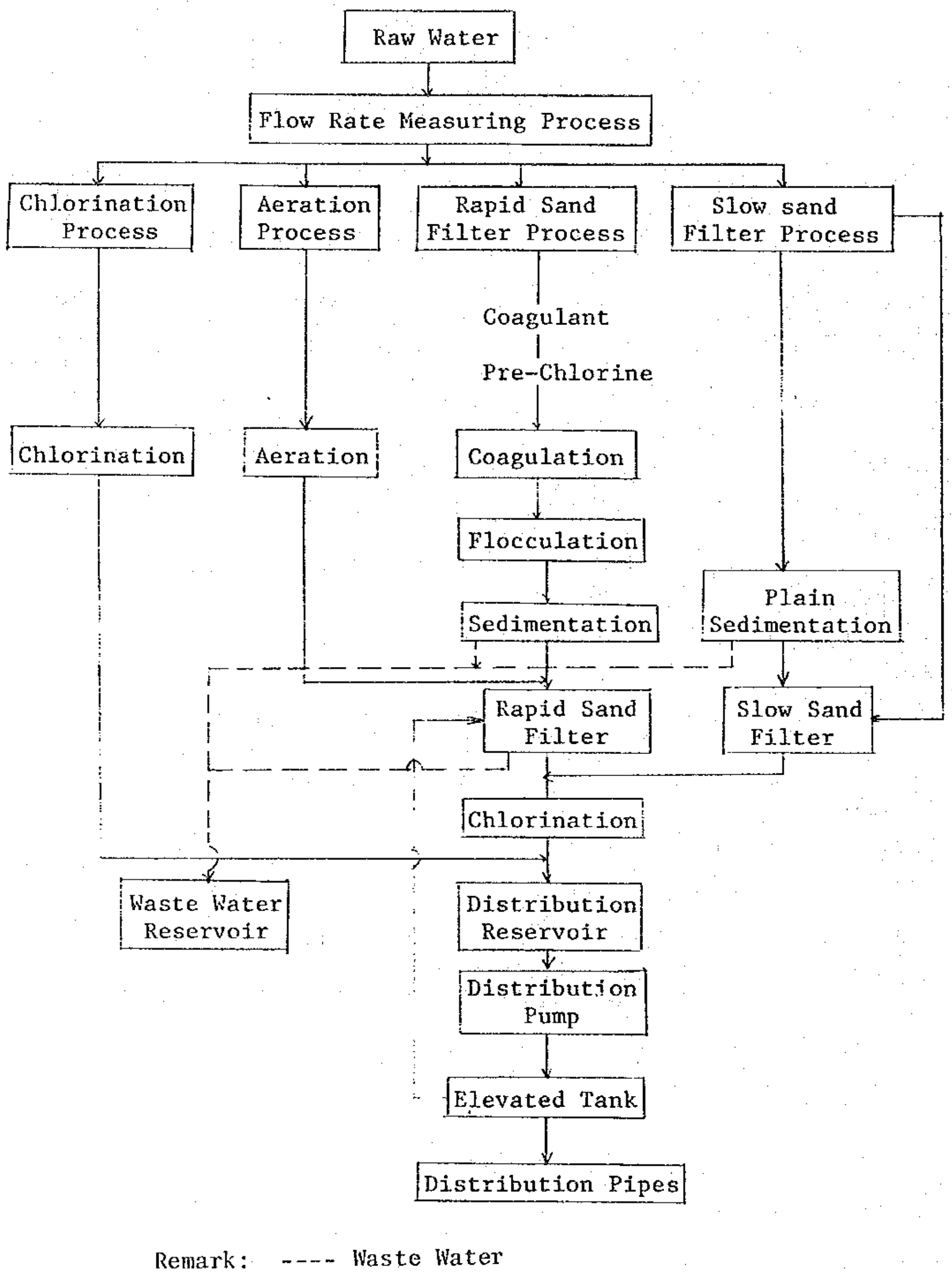


Fig. 5-1. Classification of Water Treatment Process

In general, unpolluted surface water with low turbidity may be purified by slow sand filtration as single treatment process or by rapid filtration followed by chlorination only. Slow sand filtration have great advantage particularly for small community. When the turbidity of water to be treated is high or when algae are present, a pre-treatment will be needed with sedimentation, rapid filtration or both process in combination.

For water from rivers, lakes and reservoirs, which is of a very wide variety in composition, the treatment processes of such water shall be studied carefully.

On the other hand, groundwater, if properly withdrawn, will be free from turbidity and pathogenic organisms. When the groundwater originates from a clean sand aquifer, other hazardous or objectionable matters will also be absent. In these cases, a direct use of the water for drinking may be permitted without any treatment.

Sometimes, groundwater contains organic matters or excessive amounts of iron, manganese and ammonia, and even such groundwater sources are abstracted and treated with aeration, chemical coagulation, flocculation or filtration to render them fit for drinking and domestic use.

5.2. Measuring Devices

5.2.1. Objectives of Flow Meters

Flow meters shall be installed to check the inflow amount of raw water to treatment plant and to render successful O and M services of the facilities as well as to serve for determination of the dosing amount of chemicals.

5.2.2. Kind of Flow Meters and Adaptability

Following shows the flow meters by kinds and type:

1) Over flow type weir
   * Triangular weir
   * Rectangular weir (Reducer type)
   * Rectangular weir (Flat type)
2) Parshall flume
3) Venturi meter
4) Orifice
5) Area flow meter
6) Magnetic flow meter
7) Inferential flow meter

The specific features of the respective flow meters and calculation method of flow can be referred to in Appendix F.

In general, the flow meters shall function both for checking the inflow and serving to estimate the amount of coagulant to dose. Coagulant dosing and stirring shall be made to secure even distribution of chemicals. The weir, parshall flume type meter is recommended as the most suitable type to the requirements.

5.3. Rapid Sand Filtration

5.3.1. General

Filtration shall commonly be conducted by either of the following ways as rapid sand filtration, slow sand filtration, magnetic filtration, etc., among which the rapid sand filtration has been most widely used for the purpose.

The rapid sand filtration has the mechanism that coagulant is dosed in raw water to form easily-settling suspension solids (floc), the water can be filtered effectively with flocs settled rapidly. This plant consists of the following unit process. (Refer to Fig. 5-2.)

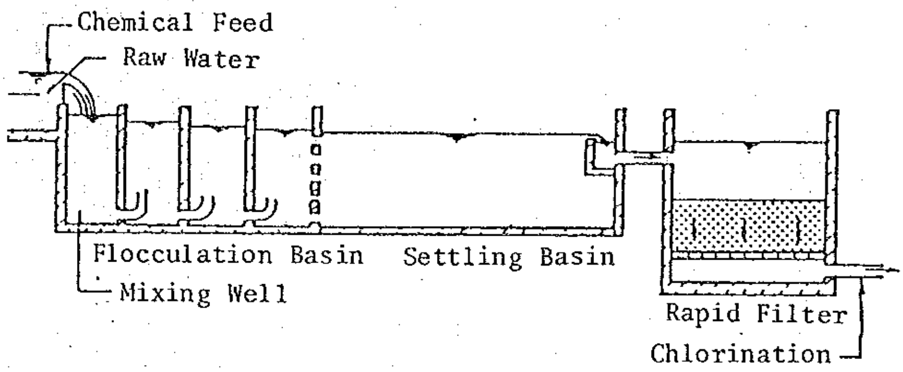


Fig. 5-2. Flow Diagram of Rapid Sand Filtration (Ref. Book No. 1)

5.3.2. Coagulation and Flocculation

1) General

The major purpose of coagulation and flocculation process in a treatment system is to agglomerate suspended/colloidal fine matters in the water to form flocs for removal.

Colloidal particles (colloids) are suspended by electrostatic repulsion and hydration. Electrostatic repulsion occurs because colloids usually have a surface charge due to the presence of a double layer of irons around each particle.

The colloid has an electric charge mostly a negative one. Hydration is the reaction of particles at their surface with the surrounding water. The resulting particle water agglomerates have a specific gravity which differs little from that of water itself.

The electrostatic repulsion between colloidal particles effectively cancels out the mass attraction forces (Van der Waal's forces) the colloidal particles.

They check the electrostatic repulsion, and thus enable the particles to flocculate to form flocs. These flocs can grow to a sufficient size and specific weight to allow their removal by setting or filtration.

2) Rapid Mixing

Rapid mixing aims at the immediate dispersal of the entire dose of chemicals throughout the mass of the raw water. For even distribution of chemicals, it is necessary to agitate the water thoroughly and to inject the chemicals in the most turbulent zone in order to ensure its uniform and rapid dispersal. The mixing should be made rapidly because the hydrolysis of the coagulant takes place almost instantaneously (within a few seconds).

There are two types of mixing devices, hydraulic rapid mixing and mechanical rapid mixing. The hydraulic rapid mixing type further can be subdivided into weir and parshall flume.

Mechanical rapid mixers are not so suitable for small scale treatment plants as hydraulic ones, because they require a reliable and continuous supply of power.

The weir type and/or the parshall flume type are recommendable for the Project taking into account safe operation and dual purposes of rapid mixing and discharge measuring. Figure 5-3 shows the weir type device.

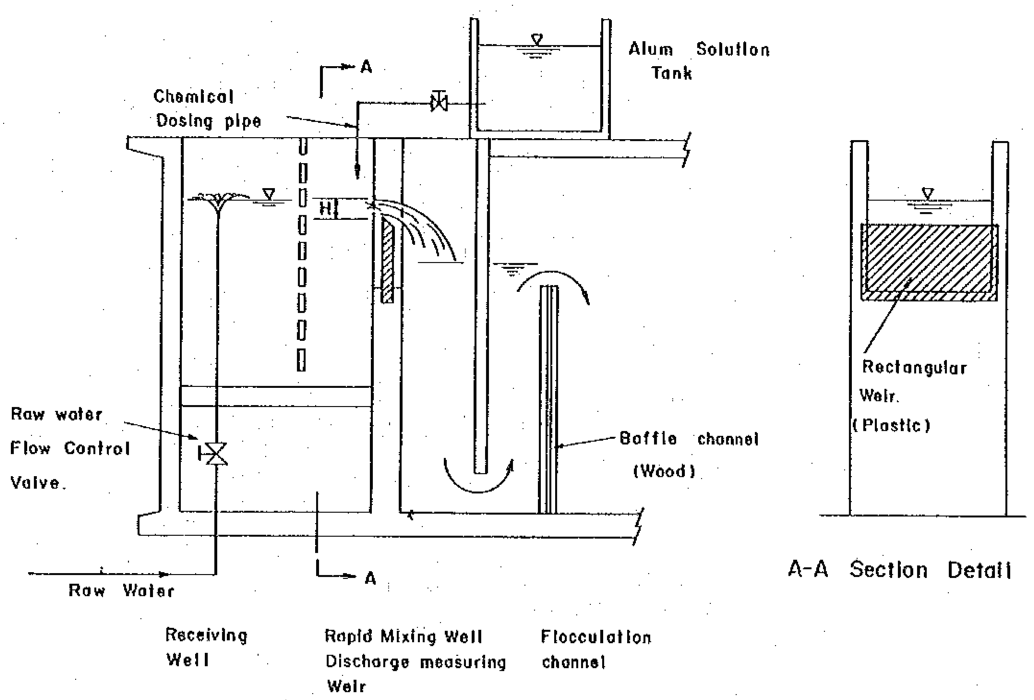


Fig. 5-3. Rapid Mixing Weir

3) Flocculation

Flocculation is the process of gentle and continuous stirring up the water for the forming flocs through aggregation of the minute particles present in the water.

The flocculation efficiency is largely affected by the number of collisions between the minute coagulated particles per unit of time.

The velocity gradient (G) and the detention time (t) are major factors in the designing for flocculator installation.

The product G and t gives a measure for the number of particle collisions.

The formula for computing the velocity gradient is:

$$G = \sqrt{\frac{\rho \cdot Q \cdot H \cdot g}{\mu \cdot V}}$$

where,

G = velocity gradient (sec^{-1})
ρ = density of water (kg/m^3)
Q = flow (m^3/s)
H = head loss (m)
g = gravitational constant (9.8 m/s^2)
μ = dynamic viscosity (kg/m/s)
V = volume of unit (m^3)

Variations of the Specific Gravity (Density) and Viscosity of Water with Temperature

Temperature °C		Dynamic Viscosity (μ) (kg/m/s)
0	999.9	0.00179
5	1000	0.00152
10	999.7	0.00131
15	999.1	0.00114
20	998.2	0.00101
25	997.1	0.00089
30	995.7	0.00080

Flocculators are classified into two types as follows:

a) Mechanical flocculators
The water is stirred up with devices of paddles, paddle reels or rakes.

b) Hydraulic flocculators
The flow of the water is so influenced by small hydraulic structures that a stirring action results. Generally, hydraulic flocculations are used for small scale treatment system. There are three types of hydraulic flocculators as follows:

i) Horizontal type baffled channel flocculators (refer to Fig. 5-4).

- Water flow velocity : 0.10 - 0.2 m/s

- Detention time: 15 - 20 minutes
- Velocity gradient: 40 - 90 sec^{-1}

ii) Vertical type baffled channel flocculators (refer to Fig. 5-5). This type is most recommendable because baffles are adjustable to changes in water flow and water quality.

- Velocity : 0.1 - 0.2 m/s
- Detention : 15 - 20 minutes
- Velocity gradient: 40 - 90 sec^{-1}

iii) Hydraulic jet mixer (refer to Fig. 5-6). In a jet the coagulant is injected in the raw water using a special orifice device.

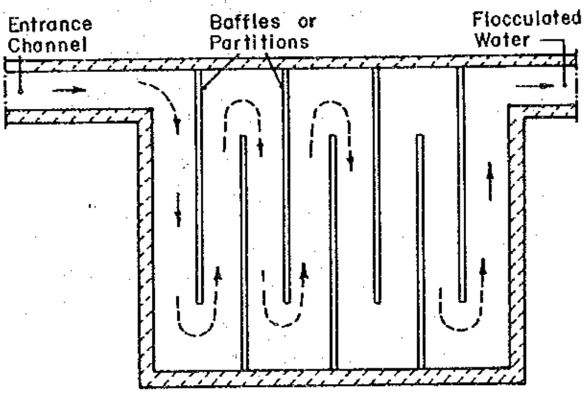


Fig. 5-4. Horizontal Type Flocculator
(Ref. Book No. 1)

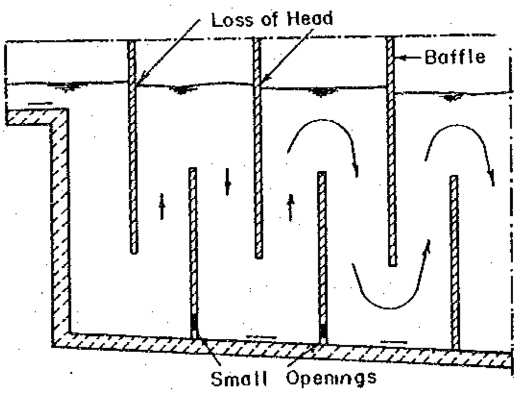


Fig. 5-5. Vertical Type Flocculator
(Ref. Book No. 1)

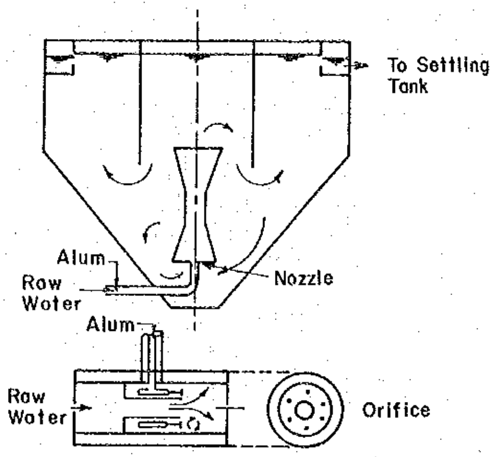


Fig. 5-6. Hydraulic Jet Mixer and Flocculator
(Ref. Book No. 1)

[Example 5-1)

Design of the receiving well, measuring flow, rapid mixer and vertical type baffled channel flocculators (refer to Fig. 5-7).

1) Design data

Treatment capacity : Q = 50 (m^3/hr)
= 0.83 (m^3/min)
= 0.014 (m^3/sec)

2) Calculation

a) Receiving well

Dimension : 0.5 W x 1.5 L x 1.5 H
Volume : 1.12 m^3
Detention Time: 1.12/0.83 = 1.3(min.)

b) Measuring weir: triangle weir (Θ = 90)

$$Q = 60\ (1.334 + \frac{0.0205}{\sqrt{H}})\ H^{5/2}$$ ------ Sticland's formula

From Fig. F-1, H can be read as:

H = 159 (mm)

Calculations table is shown in Appendix F-1.

c) Flocculation basin

Dimension: $0.4^{W} \times \{(5.25 - 1.5) + 5.25 \times 4\}^{L} \times 1.2^{H}$

Volume: 11.9 (m^3)

Detention Time: 11.9/0.83 ≒ 15 (min.)

Velocity: 0.014/0.4 x 0.25 = 0.14 (m/sec)

Head Loss: H

Bend loss: h_1

$$h_1 = f_1 \frac{V^2}{2g} \cdot N_1 \quad , \quad f_1 = 3.5, \quad N_1 = 38$$

$$= 3.5 \times \frac{0.14^2}{2 \times 9.8} \times 38$$

$$= 0.133$$

Weir loss: h_2

$$h_2 = f_2 \frac{V^2}{2g} \cdot N_2 \quad , \quad f_2 = 1.0, \quad N_2 = 37$$

$$= 1.0 \times \frac{0.14^2}{2 \times 9.8} \times 37$$

$$= 0.037$$

Bottom and wall loss: h_3

$$h_3 = \frac{L}{C^2 R} V^2 \quad \text{-------} \quad \text{Chazy formula}$$

$C = 31$

$$= \frac{(24.75 + 75 \times 1.2) \times 0.14^2}{31^2 \times (0.4 \times 0.25)/2 \times (0.4 + 0.25)}$$

$$= 0.029$$

Total head loss: $H = h_1 + h_2 + h_3 = 0.20$ (m)

G-value

$$G = \sqrt{\frac{P.Q.H.g}{\mu\ .V}}$$

$$= \sqrt{\frac{10^3 \times 0.014 \times 0.20 \times 9.8}{10^{-3} \times 13.6}}$$

$$= 44.5\ (\mathrm{sec}^{-1})$$

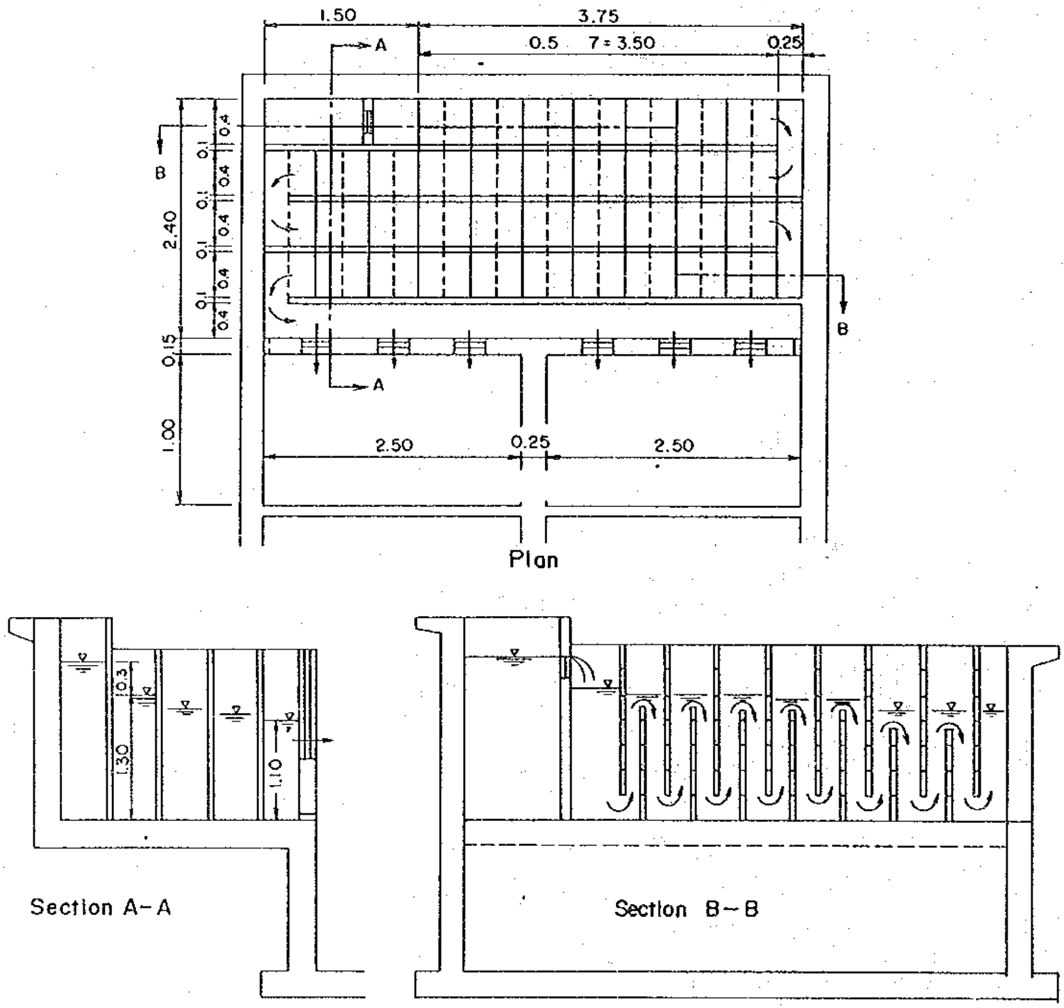


Fig. 5-7. Rapid Mixer and Flocculator Arrangement

5.3.3. Sedimentation

1) General

Sedimentation aims to settle and remove suspended particles (forms in flocs) that are made when water is still in, or flows slowly through a basin, and thus clear water of sedimentation can be reduced on the filtration loading.

Most of the existing sedimentation basins used in small water treatment plants are of the horizontal-flow type.

The rectangular horizontal-flow sedimentation basin consists of the influent zone, settling zone, effluent zone and sludge zone. (Refer to Fig. 5-8.)

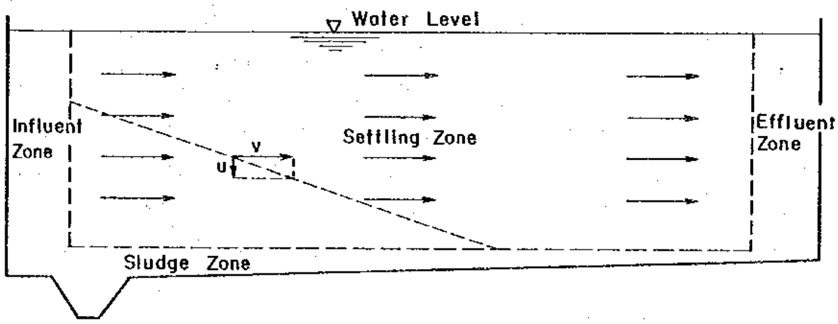


Fig. 5-8. Rectangular Horizontal Flow Sedimentation Basin (Ref. Book No. 3)

2) Design consideration

The basic formula of sedimentation basin design are:

$$t = V/Q$$
$$U = H/t$$
$$= Q/W.L$$

Where,

t : detention time (hr)

V : basin volume (m^3)

$V = H \times W \times L$

H, W, L : depth, width and length of basin (m)

Q : influent flow rate (m^3/hr)

U : surface loading rate or settling velocity ($m^3/hr/m^2 = m/hr$)

The above formula shows that the settling velocity basically only depends on the influent flow rate and surface area of the basin. Optional settling velocity (U)is determined by settling test of settable particles in the raw water. The method of setting test is referred in Reference Book Nos. 1, 3, 7, 9, and 10.

3) Design Criteria

The design of sedimentation basins is made based on the following basic criteria, such as (1) the quantity of water to be treated, (2) the selected detention time, and (3) the selected surface loading rate.

Most recommendable dimensions and criteria of the design are as follows:

- Number of sedimentation basin appropriate 2 basins with the capacity of more than 50 m^3/hr and 1 basin with the capacity of less than 40 m^3/hr.
- Detention time ... 2 - 4 hours at the maximum daily demand
- Flow velocity ... less than 40 cm/min
- Effective depth ... 3.0 - 4.0 m
- Width; length ... 1 ; 3 to 5
- Surface loading rate ... 5 - 15 mm/min.
- Over flow rate of outlet weir ... less than 500 $m^3/day/m$

- Influent zone
  For a uniform distribution of the influent these openings should be spaced as shown in Fig. 5-10.

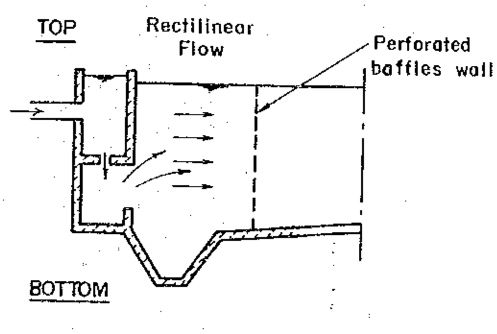


Fig. 5-9. Influent Arrangement
(Ref. Book No. 1)

- Settling zone
  For a density current of a rectilinear flow in the settling zone, the perforated battle wall will be set up. (Refer to Fig. 5-10.)

- <u>Effluent zone</u>
  For reducing approach velocity, outlet channel or triangular notched are provided on the effluent zone. (Refer to Figs. 5-10 and 5-11 below.)

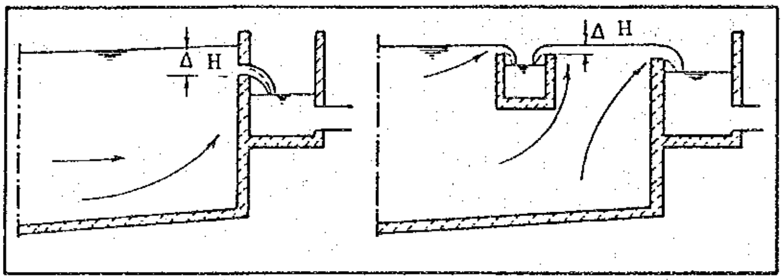


Fig. 5-10. Effluent Arrangement
(Ref. Book No. 1)

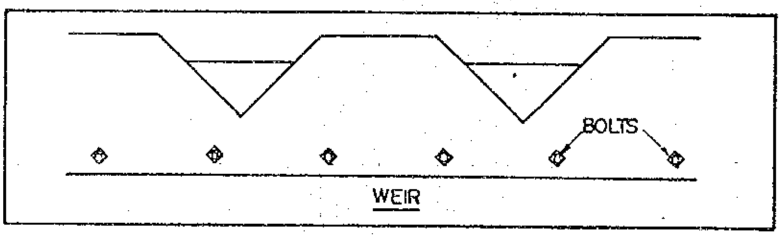


Fig. 5-11. Triangular Notches
(Ref. Book No. 1)

[Example 5.2] Design of Sedimentation

(1) Design data

a) Treatment capacity ... $Q = 50$ (m^3/hr)

b) Raw water turbidity... $Tr = 50$ (mg/l)

c) Treated water turbidity... $Tr_1 = 5$ (mg/l)

d) Alum feeding rate ... 20 (mg/l)

e) Sludge content ... 10 (kg/m^3)

(2) Calculation

a) Dimension of sedimentation basin

(i) Detention time $T = 3.0$ (hr)

(ii) Volume $V = 50 \times 3 = 150$ (m^3)

(iii) Dimension $\underset{(W)}{2.8} \times \underset{(L)}{9.0} \times \underset{(H)}{3.0} \times 2 \doteqdot 150m^3$

b) Flow velocity

$$V = \frac{50}{60 \times 2.8 \times 3.0 \times 2} = 0.05 < 0.4 \text{ (m/min)}$$

c) Overflow rate

$$FR = \frac{50}{60 \times 2.8 \times 9.0 \times 2} = 15 \leqq 15 \text{ (mm/min)}$$

d) Baffle flow wall

Total hole area = flow area 6%

$= 2.8 \times 3.0 \times 0.06 = 0.5$ (m)

ϕ100 mm .... $A = 0.067$ (m^2), $N = 0.5/0.007 = 71$

e) Overflow rate of outlet weir

(i) flow rate ..... 500 (m^3/day/m)

weir length $L = 50 \times 24/500 = 2.4$ (m)

f) Sludge removal

Total sludge volume

$$Tds = \frac{(50 - 5) + 0.234 \times 20}{10} \times 50 \times 24 \times 10^{-3} = \frac{59.5}{10} = 6.0 \ (m^3/d)$$

Drain time ; 5 min/day

Pipe diameter ; ϕ50 mm

Number of drain pipe ; 6

$$Q = \frac{6.0}{5.0 \times 6} = 0.2 \ (m^3/min) = 0.003 \ (m^3/sec)$$

$$\text{Head loss} = \lambda \frac{1}{d} \frac{V^2}{2g} + \Sigma f \frac{V^2}{2g}$$

$$Q = \frac{\pi d^2}{4} \sqrt{\frac{2gh}{\lambda \ell / d + f_1}} = \frac{\pi d^2}{4} \sqrt{\frac{2 \times 9.8 \times 3.0}{0.03 \times 3/d + 4.0}}$$

$$= \frac{\pi d^2}{4} \sqrt{\frac{58.8}{0.09/d + 4.0}}$$

$d = 0.05m$

$$Q = \frac{\pi \times 0.05^2}{4} \sqrt{\frac{58.8}{0.09/0.05 + 4.0}} = 0.002 \times \sqrt{\frac{58.8}{5.8}}$$

$$= 0.02 \times 3.17 = 0.06 \qquad (m^3/s)$$

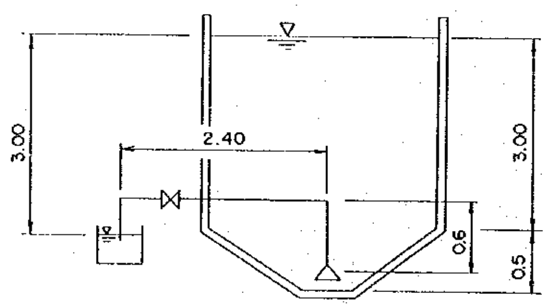


Fig. 5-12. Sludge Drain Device

5.3.4. Rapid Sand Filter

1) General

The rapid sand filter aims to purify raw waters into clean water by filtering at high velocity through chemical settling basin with coarse sand filter layer.

For rapid filtration, sand is commonly used as the filter medium but the process is quite different from slow sand filtration. This is so because much coarser sand is used with an effective grain size in the range 0.4-1.2 mm, and the filtration rate is much higher, generally between 5 and 15 m^3/m^2/hour (120-360 m^3/m^2/day). Due to the coarse sand used, the pores of the filter bed will be relatively large and the impurities contained in the raw water will penetrate deep by into the filter bed. Thus the capacity of the filter bed to store deposited impurities is much more effectively utilized and even very turbid river water can be treated with rapid filtration. For cleaning a rapid filter bed, it is not sufficient to scrape off the top layer. Cleaning of rapid filters is effected by back washing. This is directing a high-rate flow of water back through the filter bed whereby it expands and is scoured. The back-wash water carries the deposited cloggings out of the filter. The cleaning of a rapid filter can be carried out quickly; it need not take more than about one and half hour. It can be done as frequently as required, if necessary each day.

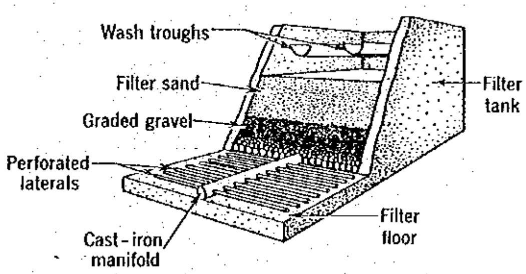


Fig. 5-13. Rapid Sand Filter (Ref. Book No. 9)

For effective filtration, the pretreatment should produce floc particles that are small enough to penetrate the bed. Removal of floc within a bed is accomplished primarily the grains, or floc already deposited, and adherence thereto.

2) Type of rapid sand filters

Rapid filters are mostly built open with the water passing down the filter bed by gravity.

For certain operating conditions, other rapid filters than the open gravity-type are better suited. Other types are pressure filters, and multiple-media filters.

a) Pressure filters

Pressure filters (Fig. 5-14) are of the same construction as gravity-type filters but the filter bed together with the filter bottom is enclosed in a water-tight steel pressure vessel. The driving force for the filtration process is the water pressure applied on the filter bed which can be so high that almost any desired length of filter run is obtainable.

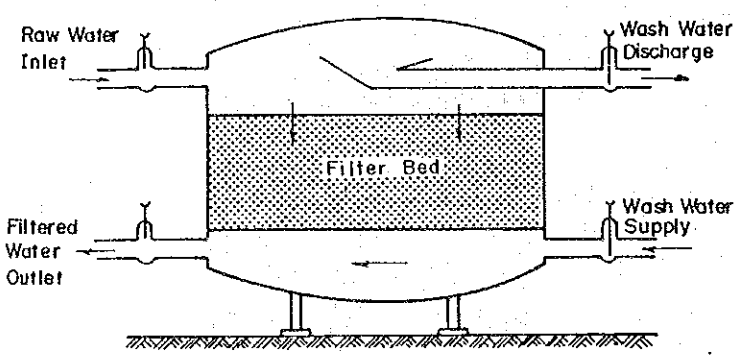
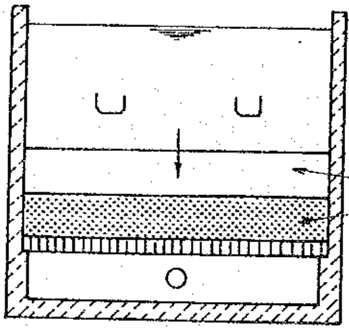


Fig. 5-14. Pressure Filter (Ref. Book No. 1)

b) Dual-media filters (Fig. 5-15) are of gravity-type, downflow filter with the filter bed composed of several different materials which are placed coarse-to-fine in the direction of flow.



Filter Bed Material

Material	Depth	Diameter	Specific Weight
Authracite	0.6m	1.6	1.5
Sand	0.4m	0.8	2.6

Fig. 5-15. Dual-Media Filter (Ref. Book No. 1)

3) Design consideration

a) Filter control

There are three types of filtration-rate controllers. (Refer to Fig. 5-16.) The type b) is probably simplest as there are no moving parts at all. In this type the raw water enters the filter over a weir. For all filters the weir crest is at the same level. Thus, the over-flow rate at each weir will be the same, and the raw water feed to the filter units will be equally split.

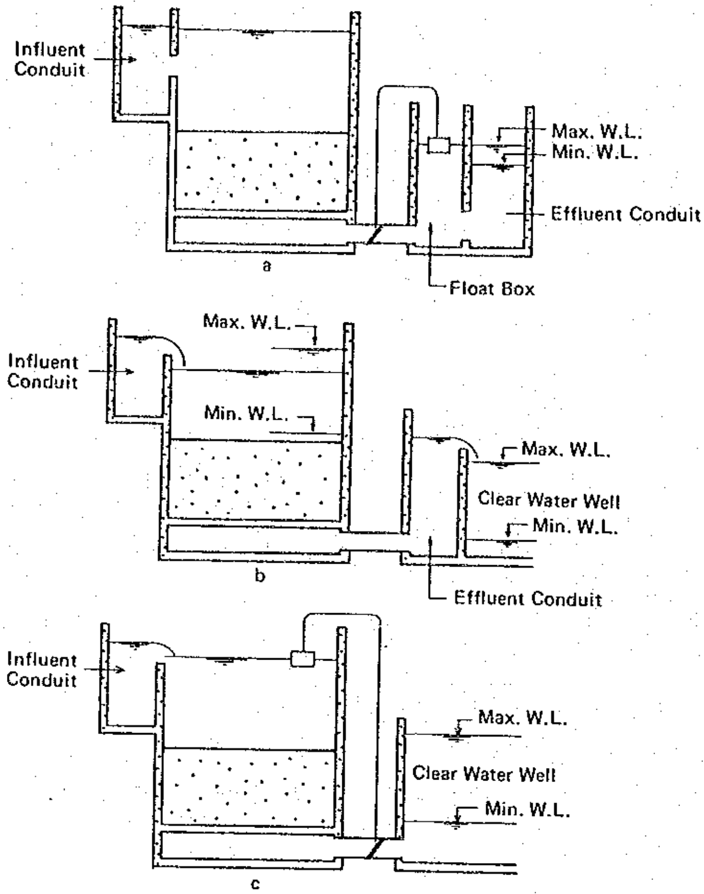


Fig. 5-16. Filter Control System
(Ref. Book No. 1)

Therefore, the type b) is the most recommendable because any mechanical equipment is not used.

b) Backwash arrangement

A rapid filter is cleaned by backwashing, that is directing a flow of clean water upward through the filter bed for a period of a few minutes. Filtered water accumulated by pumping in an elevated tank can be used, or the effluent from the other (operating) filter units of the filtration plant directly ("self-wash arrangements"). The velocity of the upward water flow should be high enough to produce an expansion of the filter bed so that the accumulated cloggings can be loosened and carried away with the washwater (refer to Fig. 5-17.)

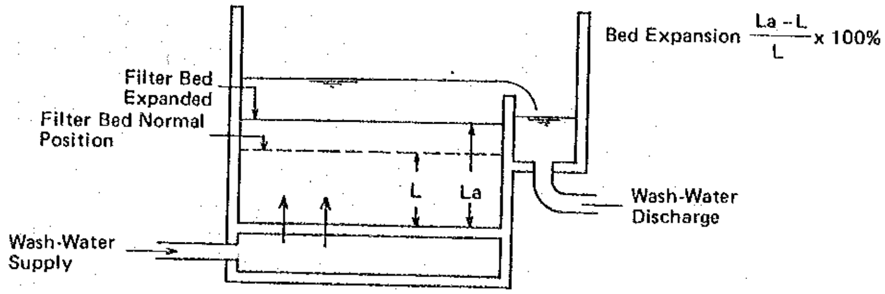


Fig. 5-17. Backwash Arrangement (Ref. Book No. 1)

For a filter bed of sands (specific weight: 2.65 g/cm^3) typical backwash rates giving about 20 percent expansion are listed in Table 5-3.

d mm	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
t	V ; BACKWASH RATE (m^3/m^2/hour)								
10°C	12	17	22	28	34	40	47	54	62
20	14	20	26	33	40	48	56	64	75
30	16	23	30	38	47	56	65	75	86

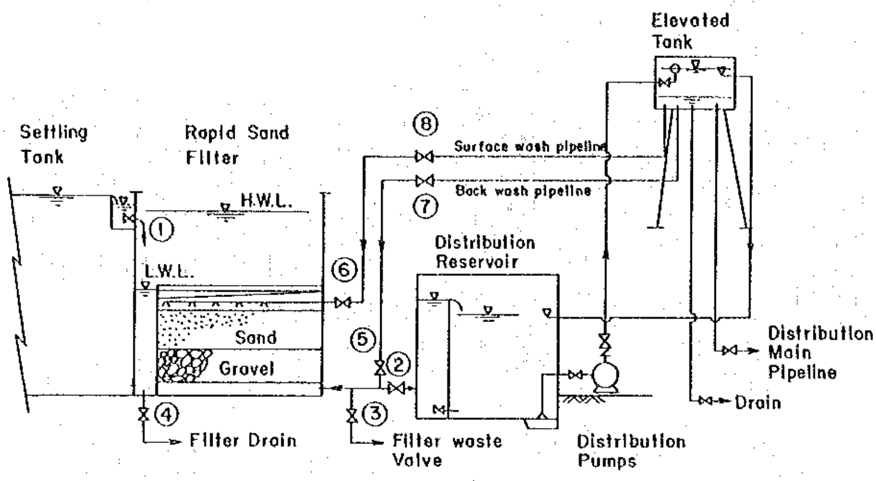
d = average grain size of filter sand (mm)

t = back-wash water temperature (°C)

v = back-wash rate ($m^{3/m2}$/hour)

Table 5-3. Typical Backwash Rates (Ref. Book No. 1)

The reservoir generally should have a capacity between 3 and 6 m^3 per square meter of filter bed area and it should be placed about 4-6 meters above the water level in the filter.



① Influent Valve or Influent Flow Splitting Weir
② Effluent Valve
③ Filter waste Valve
④ Filter Drain Valve
⑤ Back wash Water Valve
⑥ Surface wash Water Valve
⑦ Back wash Water Control Valve
⑧ Surface wash Water Control Valve

Fig. 5-18. Wash-Water Tank Arrangement

A simpler solution is to increase the depth of the water standing over the filter bed and it limits the maximum filter resistance. The filtered water will then be available at a head of some 1.5 to 2 m above the filter bed which should be sufficient. The operating units of the filtration plant must supply enough water for the required backwash rate.

The wash water is admitted at the underside of the filter bed through the underdrain system ("filter bottom"). To divide the wash water evenly over the entire filter bed area, the underdrain system should provide a sufficient resistance against the passage of wash water (generally 0.6-1.0 m head of water).

Backwash velocity and backwash pressure are obtained from the following formula: (Refer to Example 5.3)

$$h_b = \frac{L_o (1 - E_o) \times (\rho_s - \rho_f)}{\rho_f}$$

where,

- U : most reasonable backwash velocity (m/min)
- h_b : backwash pressure (m)
- L_o : sand bed depth (m)
- E_o : porosity of unexpended bed
- ρ_s : density of sand (kg/m^3)
- ρ_f : density of water (kg/m^3)

4) Surface water arrangement

Particularly when fine sand is used, with grain size less than about 0.8 mm, the scouring force of the rising wash water may be inadequate to keep the filter grains clean in the long run. After some time they could become covered with a sticky layer of organic matter. This may cause problems such as mud balls and filter cracks (refer to Fig. 5-19).

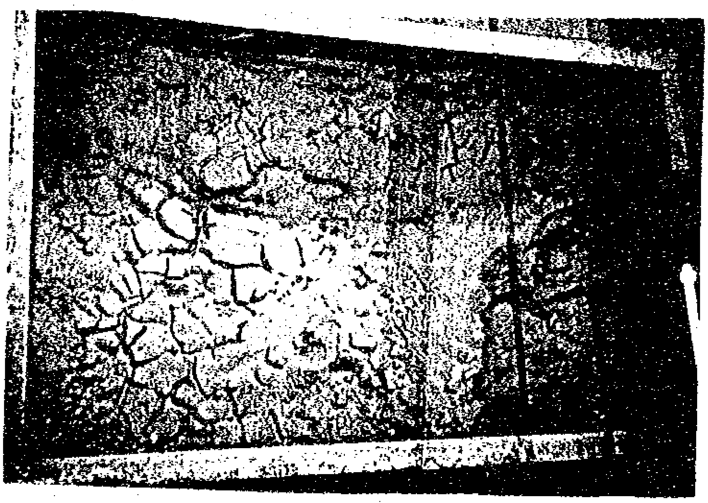


Fig. 5-19. Sticky Layer of Organic Matter

5) Major items of design criteria

a) The filtration rate shall be commonly take in a range from 120 to 200 m^3/day.

b) There shall be two basins to be provided, including one stand-by basin.

c) The maximum filtration head shall be taken by less than 1.50 m.

d) The washing rate shall be as follows:

i) Backwash --- 0.5-0.7 m/min in velocity at 1.5 m actual head.

ii) Surfacewash --- 0.15-0.2 m/min in velocity at 15-20 m actual head.

e) Materials of filter layers

Materials	Effective Size (mm)	Thickness (m)
Sand	0.45 - 0.7	0.6 - 0.7
Gravel	2 - 5	0.1
"	5 - 10	0.1
"	10 - 15	0.1
"	15 - 30	0.1

Note: Coefficient of uniformity of sand shall be 1.3 to 1.7.

f) Filter bottom : Underdrain pipe or porous blocks.

There can be prevented by providing an additional scour through surface water wash. Filter cleaning now starts by back washing usually combined with a surface water wash at a 10-15 m/hour rate. (Refer to 5-20.)

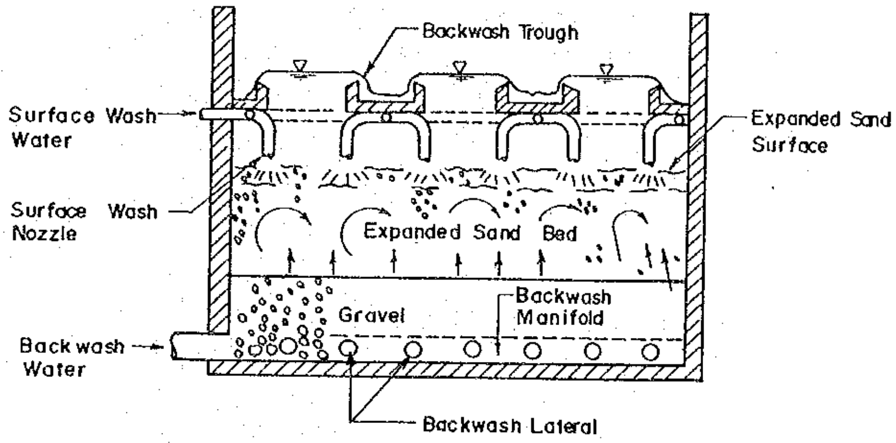


Fig. 5-20. Backwashing with Surface Wash Water (Ref. Book No. 4)

Table 5-4. Washing Method of Rapid Sand Filter (Ref. Book No. 4)

Washing Method / items	Surface and Backwash Fixed Type Surface-Wash
Pressure of Surface-Wash Water (kg/cm^2)	1.5 to 2.0
Flow Rate of Surface-Wash Water (m^3/m^2/min.)	0.15 to 0.20
Duration Time of Surface-Wash (min.)	4 to 6
Pressure of Back-Wash Water (kg/cm^2)	0.25 to 0.50
Flow Rate of Back-Wash Water (m^3/m^2/min.)	0.5 to 0.7
Duration Time of Back-Wash (min.)	4 to 6

Note: Pressure of back-wash water and surface-wash water are values at discharge points(Orifice of jet) of Underdrain system and surface-washing system, respectively.

[Example 5.3]

Head losses of the backwash water. (Refer to Fig. 5-21)

1) Design

a) Water treatment capacity: $Q = 50\ (m^3/hr)$
b) Filtration rate: $V = 120\ (m/d)$
c) Filter area: $A = Q/V = 10\ (m^2)$
d) Number of basin: $N = 2$
e) Dimension : 2.30 (m) x 2.20 (m) = 5.0 (m^2)
f) Filter bed

i) Sand; effective size: 0.65 (mm)
space coefficient: 0.48
depth: 0.60 (m)
density: 2.630 (kg/m^3)

ii) Gravel; 1st layer: effective dia. 2.0 (mm)
2nd layer: effective dia. 4.0 (mm)
space coefficient: 0.4
depth: 0.2 (m) each layer
form coefficient: 0.7

g) Filter bottom
Underdrain Type
Open area rate: 1.0 (%)
Velocity coefficient: 0.62

h) Back-wash velocity: 0.65 (m/min) = 0.0108 (m/sec)
Back-wash flow: 5.0 x 0.65 = 3.25 (m^3/min)
= 0.054 (m^3/sec)

2) Calculation

(1) Head loss at sand bed: h_1

$$h_1 = \frac{L_o}{\rho t}(1 - E_o) \times (\rho s - \rho f)$$

$$= \frac{0.6}{10^3}(1 - 0.48)(2,630 - 1,000)$$

$$= 0.51\ (m)$$

(2) Head loss at gravel layer; h_2

$$h_2 = \frac{200 \times L_o \times U \times \mu}{\rho f \times g \times \phi^2 \times D^2} \times \frac{(1 - E_o)^2}{E_o{}^3}$$

$$= \frac{200 \times 0.0108 \times 10^{-3}}{10^3 \times 9.8 \times 0.7^2} \times \left(\frac{0.2}{(2\text{x}10^{-3})^2} + \frac{0.2}{(4\text{x}10^{-3})^2}\right) \times \frac{(1 - 0.4)^2}{0.4^3}$$

$$= 0.32 \text{ (m)}$$

μ : Kinematic Viscosity of Water

1 (CP) = 10^{-3} (kg/m.s) 20^oC

(3) Head loss at filter bottom;

$$h_3 = \frac{1}{2g} \left(\frac{U}{\alpha\ \beta}\right)^2$$

$$= \frac{1}{2 \times 9.8} \left(\frac{0.0108}{0.62 \times 0.01}\right)^2$$

$$= 0.16 \text{ (m)}$$

(4) The total head losses at the filter bottom influent

$$h_o = h_1 + h_2 + h_3$$

$$= 0.51 + 0.32 + 0.16$$

$$= 0.99 \fallingdotseq 1.00 \text{ (m)}$$

(5) Head loss of transmission pipe; h_4

$$h_4 = f \times \frac{L \times V^2}{D \times 2g} + f_1 \frac{V^2}{2g}$$

$$= (0.03 \times \frac{2.0}{0.15} + 4) \times \frac{3.17^2}{2 \times 9.8}$$

$$= 4.1 \text{ (m)}$$

$f = 0.03$

$V = \frac{Q}{a} = \frac{0.054}{0.017}$

$= 3.17$ (m/s)

$D = 150$ (mm)

$a = \frac{\pi D^2}{4} = 0.017$ (m^2)

$L = 20$ (m)

$f_1 = 1.0 \times 4$

Therefore, the total head of excludes valve A control loss head before value A: h_5

$$h_5 = 18.0 - (ha + ho + h_4)$$

$$= 18.0 - (1.7 + 1.0 + 4.1)$$

$$= 11.2 \text{ (m)}$$

(6) Open rate of valve A loss coefficient; f_u

$$f_u = h_5 \Big/ \frac{V^2}{2g}$$

$$= 11.2 \Big/ \frac{3.17^2}{2 \times 9.8} = 21.8$$

Accordingly, referred from Table 4-4, Valve A should be opened at about 1/5, Valve B is fully opened.

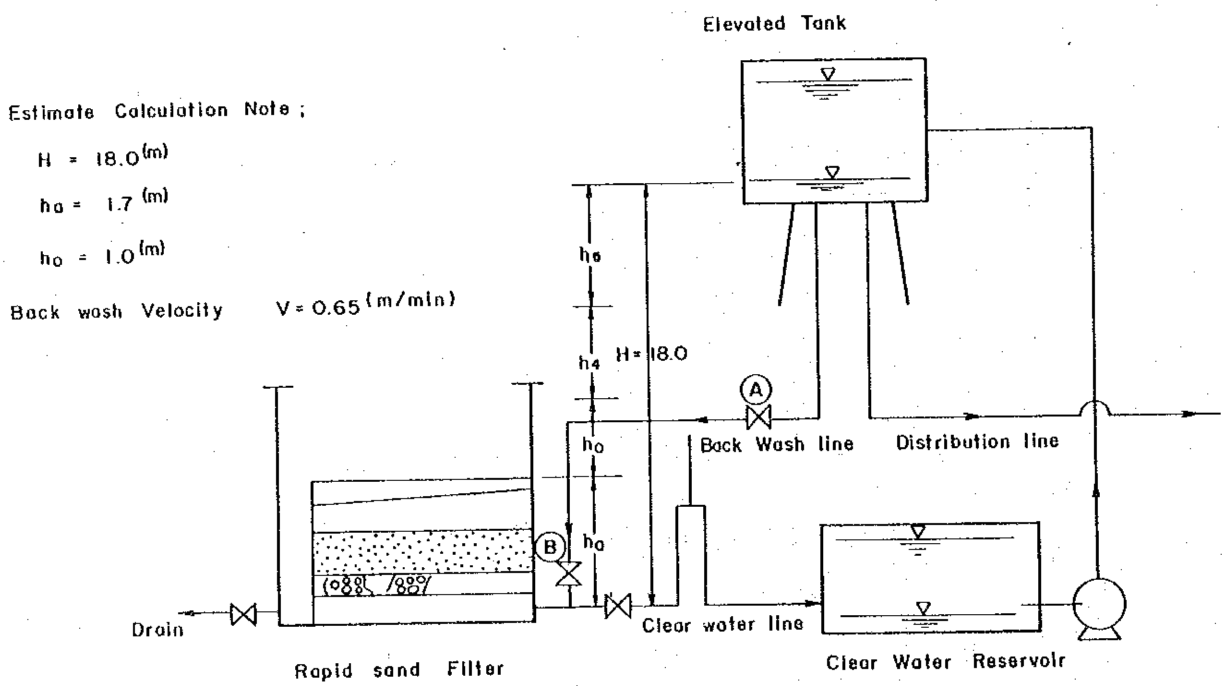


Fig. 5-21. Back-wash Flow Diagram

5.3.5. Chemicals Dosing

1) General

For successful coagulation, an optimum amount of coagulant shall be dosed into the raw water and the water shall be stirred up properly. The optimum amount of coagulation for proper dosing varies depending upon the nature and the composition of the water as a whole. A laboratory experiment called "Jar Test" is commonly employed for regular checking of the water to keep the adequate dosing.

The standards of coagulant application in Japan are shown in Appendix G.

2) Design consideration

Aluminum sulphate solid as coagulant and soda ash as alkalinity are most widely used for small-scaled treatment plants in Thailand.

a) Alum solution (usually 3 to 7%, normal 5%) is prepared in special tanks with a holding capacity of 10 or more hours coagulant feeding requirements. (Refer to Fig. 5-22.)

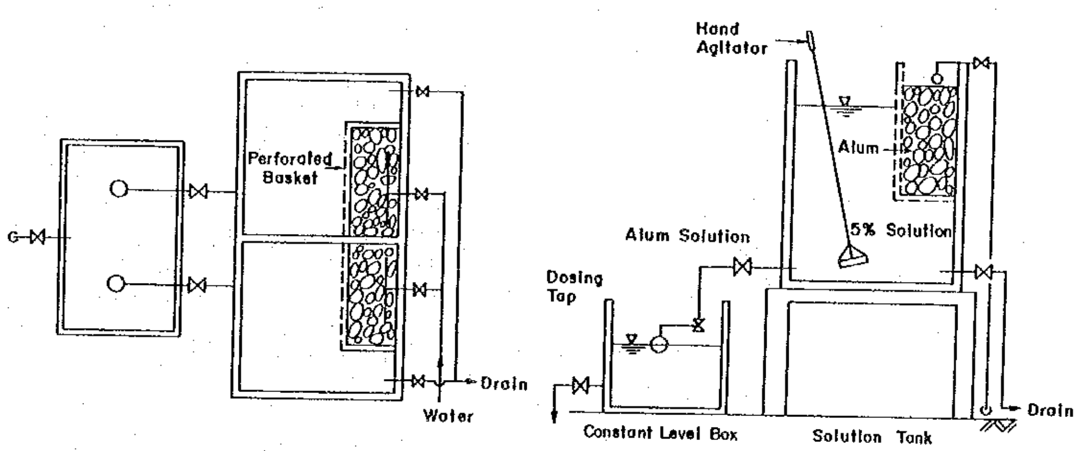


Fig. 5-22. Chemical Feed Arrangement for Alum

b) Two tanks are required; one for operation and the other for solution.

c) Alum is strong acid, so that feeding facilities was made up anti-erosive materials.

d) For coagulation with alum, the average pH value is shown at 6.0 which remains below the optimum range. Alkaline should be additionally dose so as for alkalinity to reach at least 20 degrees, if alkalinity of raw water is insufficient for coagulation. (Refer to reduction of alkalinity in Table 5-5.)

Table 5-5. Reduction of Alkalinity by Coagulant 1 ppm injection (Ref. Book No. 5)

Classification	Reduction of alkalinity (ppm)
Aluminum sulfate (solidity) (Al_2O_3, 15%)	0.45
Aluminum sulfate (liquid) (Al_2O_3, 8%)	0.24
Poly aluminum chloride (Al_2O_3, 10%, basicity 50%)	0.15

5.4. Slow Sand Filtration

5.4.1. General

The main purpose of slow sand filter is to remove suspended matters and pathogenic organisms in the raw water, in particular the bacteria and viruses responsible for spreading the water-related diseases.

The slow sand filter is also very effective in removing suspended matters from the raw water. However, trouble-free operation is only possible when the average turbidity of the raw water is less than 15 N.T.U. with peak values below 30 N.T.U. and without continuous occurring more than three to four days.

Generally, slow sand filtrator is carried out in combination with sedimentation process by basins or reservoirs as shown in Fig. 5-23.

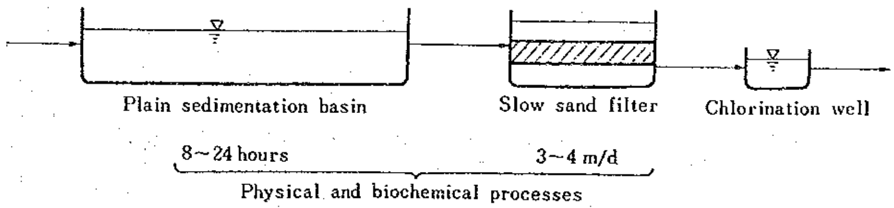


Fig. 5-23. Typical Process of Slow Sand Filter (Ref. Book No. 4)

Most important operations of the above system is to remove impurities from the raw water which are brought by biochemical and microbial actions.

In storage reservoirs, suspended particles will be removed by settling but quick blocking of the slow sand filter bed would still take place when algae grow considerably thick. Pre-treatment will then be necessary.

The purification processes start in the supernatant water but the major part of impurities are removed from the water and the microbial and biochemical action take place in the top layer of the filter bed. This filter bed is called 'Schmutzdecke', which is the 'filter skin' or layer of deposited materials that is formed at the top of a slow sand filter.

Slow sand filters have many advantages in using for small community water supply. They produce clear water that is free from suspended impurities and hygienically safe. Slow sand filters do not require coagulating chemicals. Chlorine, however, should frequently be dosed.

5.4.2. Design of System

The major design factors of slow sand filter are filtration rate, number of filter units, thickness of filter sand bed, grain size distribution and depth of supernatant water. Outline of the slow sand filter is roughly illustrated in Fig. 5-24.

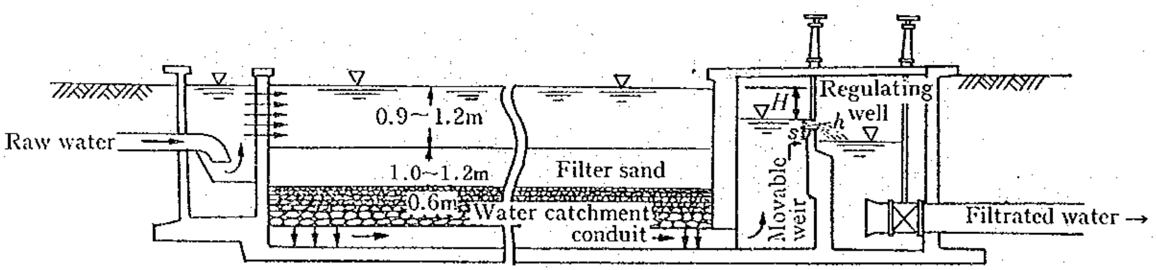


Fig. 5-24. Outline of Slow Sand Filter
(Ref. Book No. 5)

1) The filtration rate is recommended by 4-5 m/day.

2) At least two units of filters should be provided for one system, and three units, if possible. And the filtration rate should not exceed 0.2 m/hr, even if only one filter is available due to filter cleaning or any emergency of other units.

3) The thickness of the filter sand bed should be in a range from 1.0 m to 1.2 m.

4) The grain size distribution of the sand locally available should be analysed. The effective size and coefficient of uniformity can be determined from the following figure and the standard values are about 0.20 mm as effective size and less than 3 as a coefficient of uniformity. If such sand is not available, coefficient of uniformity up to 5 and an effective size of the sand ranging from 0.15 to 0.35 mm may be acceptable (refer to Fig. 5-25.)

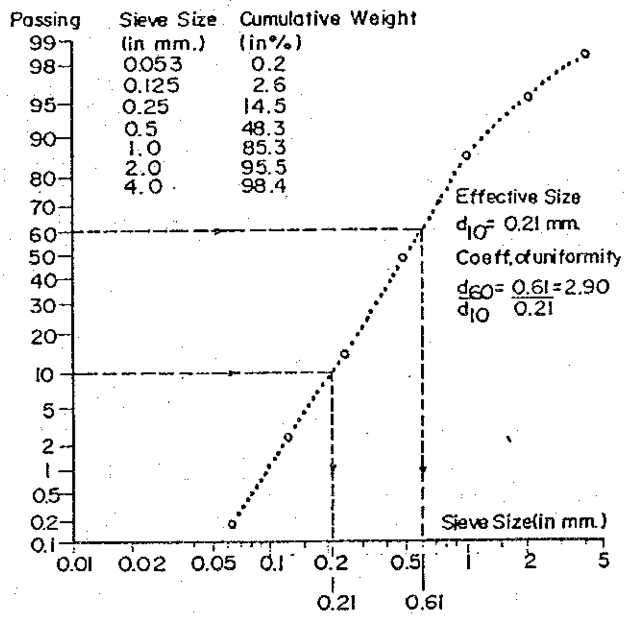


Fig. 5-25. Grain Size Distribution of Filter Sand (Ref. Book No. 1)

5) The depth of supernatant water should be about 0.9-1.2 m.

5.4.3. Cleaning

The time-proven method of cleaning a slow sand filter is by scraping off the sand surface with hand shovels to remove the top layer of dirty sand over a depth of 1.5-2.0 cm. The scraped-off mixture of sand and impurities are piled in ridges or in heaps which are carried or carted to the edge of the filter using barrows or hand-carts wheeled over wooden planks (refer to Figs. 5-26 & 5-27).

The dirty sand is sometimes discarded (can be used for landfills) but in other cases it is cleaned by washing (refer to Fig. 5-28), if cheaper than buying new sand. To prevent purification the sand should be washed immediately after taken out from the filter.

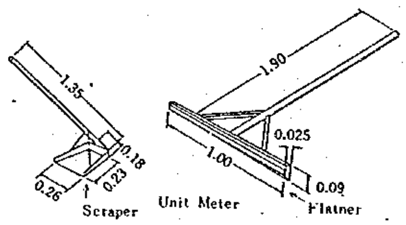


Fig. 5-26. Scraper and Flatner
(Ref. Book No.6)

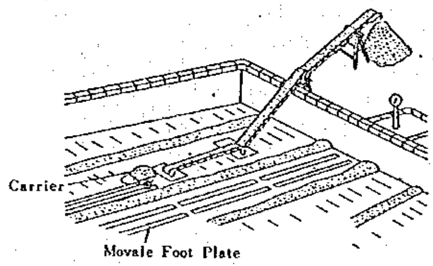


Fig. 5-27. Removal of Filter Sand
(Ref. Book No. 6)

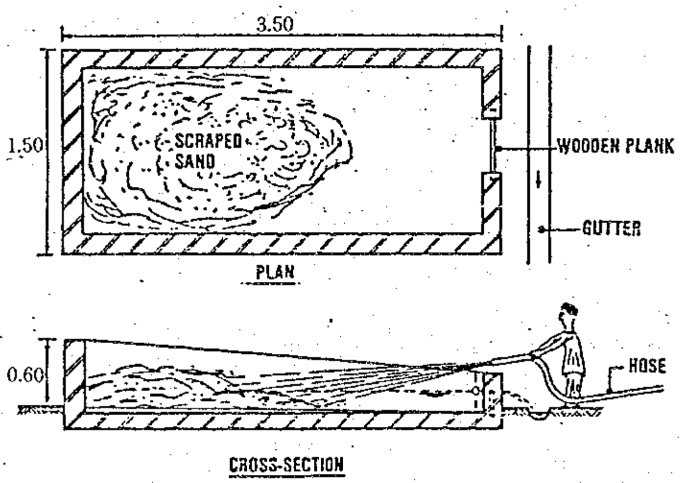


Fig. 5-28. Cleaning of Filter Sand
(Ref. Book No. 1)

5.5. Aeration Process

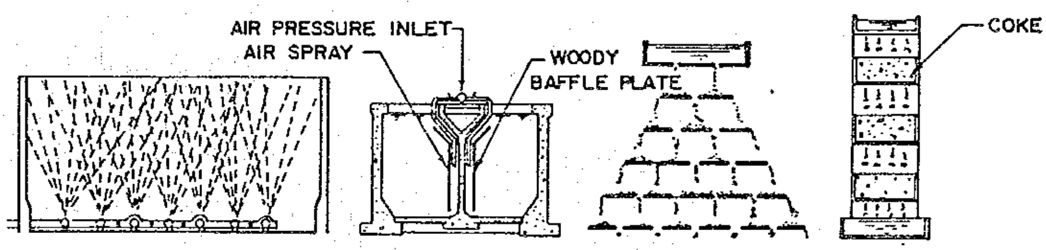
5.5.1. General

Aeration is the treatment process whereby water is brought into intimate contact with air for the purpose of increasing the oxygen content, reducing the carbon dioxide content, and removing hydrogen sulfide, methane and various volatile organic compounds responsible for taste and odor.

Aeration is widely used for the treatment of groundwater having too high iron and manganese content. For the treatment of surface water, aeration would only be useful when the water has a high content of organic matters.

5.5.2. Aeration Methods

Structures or equipment for aeration may be classified into 1) waterfall aerators, spray nozzles, multiple trays, 2) diffusion or bubble aerators, bubbles of compressed air passed through the water, and 3) mechanical aerators. The water fall type accomplishes aeration by causing the water to break into drops or thin films, thereby increasing the area of water exposed per unit of volume. The diffusion type produces a similar effect in discharging bubbles of air into water by means of air injection devices. Mechanical aerators employ motordriven impellers or in combination with air injection devices and find the greatest application in treatment of waste water. (Refer to Fig. 5-29.)



Spray Nozzle Type | Water Fall Type | Diffusion Type | Multiple-Tray Type

Fig. 5-29. Aerator Types (Ref. Book No. 5)

5.5.3. Design of System

Generally the spray type and the multiple tray type are used for aeration of the small water supplies.

1) Spray nozzles type

The device of this type shall be designed in close reference to the existing nozzles available for studying. The general ideas for designing are described as follows, although the nozzles have variety in kind.

* Diameter	2.5 - 4.0 cm
* Discharge	250 - 500 l/min at approximately 0.7 kg/cm^2 pressure
* Nozzle spacing	0.5 - 3.0 m
* Allocated area	3.0 - 9.0 $m^2/m^3/hr$

2) Multiple tray types

Coarse media such as stones or ceramic balls ranging from 5.0 cm to 15 cm in diameter shall be placed in the trays to improve the efficiency of gas exchange and distribution.

* Number of trays	3 - 9
* Tray space	30 - 75 cm
* Required area	80 - 240 $m^2/m^3/hr$
* Water application rate	50 - 75 $m^3/hr/m^2$

5.5.4. Arrangement of Aerator

In the treatment of groundwater, rapid filtration is carried out removing iron and manganese. To assist the filtration process, aeration is commonly provided as pretreatment to form insoluble compounds of iron and manganese (refer to Figs. 5-30 and 5-31).

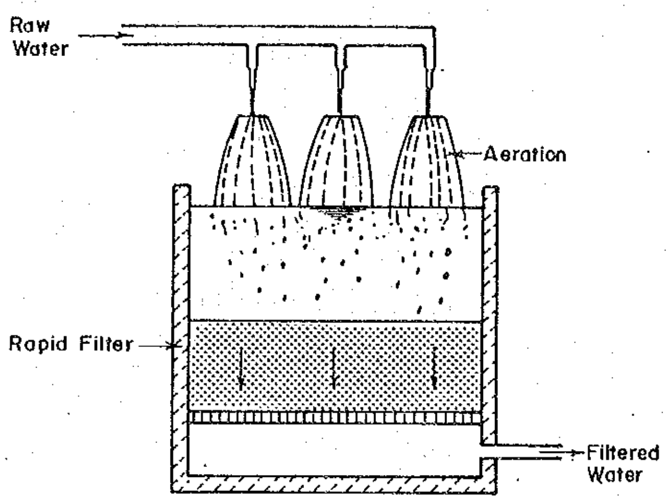


Fig. 5-30. Aeration and Rapid Filter
(Ref. Book No. 1)

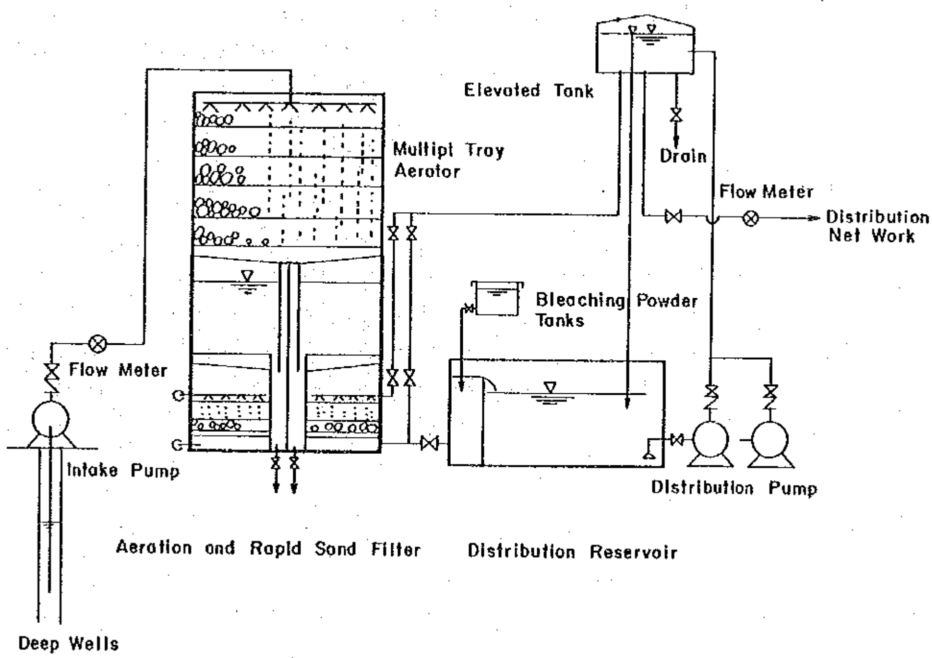


Fig. 5-31. Aeration and Rapid Sand Filter Process
Flow Diagram

5.6. Disinfection

5.6.1. General

Disinfection aims to provide water in distinction of bacteria or at least in complete inactivation of harmful micro-organisms. There are two ways in disinfection, physical and chemical, the former is to boil water and the latter is to use chlorine for the purpose.

The single and most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease of illness to the consumers. Processes such as storage, sedimentation, coagulation and flocculation, and rapid filtration reduce to varying degrees the bacterial content of water. However, these processes cannot assure that the water which they produce is bacteriologically safe. Final disinfection will commonly be needed. In cases where no other methods of treatment are available, disinfection may be resorted to as a single treatment against bacterial contamination of drinking water.

1) Physical disinfection

Boiling is a safe and time-honoured practice in destroying pathogenic micro-organisms such as viruses, bacteria, cercariae, cysts and ova. While it is effective as a household treatment it is not a feasible method for community water supplies. However, in emergency, boiling of water may be a temporary but useful measure.

Under the conditions usually associated with the boiling of drinking water, it requires about 1.0 kg of wood to boil 1.0 litre of water.

2) Chemical disinfection

The chemicals that have been successfully used for disinfection are chlorine, chlorine compounds and ozone.

The chlorination for community water supplies is extremely important.

a) Chlorine is a greenish yellow topic gas found in nature only in the combined state, chiefly with sodium as the common salt. Chlorine having a characteristic of penetrating and irritating odour, is heavier than air and can be compressed to form a clear amber-coloured liquid, which is heavier than water. It vapourises in atmosphere temperature and ordinary pressure. Commercially, chlorine is manufactured by the electrolysis of brine with caustic soda and hydrogen as by-products.

b) Chlorinated Lime ("Bleaching Powder"): Before the advent of liquid chlorine, chlorination was mostly accomplished by the use of chlorinated lime. It is a loose combination of slaked lime and chlorine gas, with the approximate composition $CaCl_2.Ca(HO)_2.H_2O + Ca(OCl)_2.2Ca(OH)_2$. By adding water, it decomposes to give hypochlorous acid, HOCl. When fresh, chlorinated lime has a chlorine content of 33 to 37 percent. Chlorinated lime is unstable and exposure to air, light and moisture makes chlorine content fall rapidly. The compound should be stored in dark, cool and dry place in closed corrosion-resistant containers.

5.6.2. Design of Chlorination System

1) Chlorination practices

Chlorination practices may be grouped into two categories depending upon the desired level of residual chlorine and the point of application.

When it is required to provide a residual and the times of contact is limited, it is a common practice to provide free residual chlorination available. If combined available residual chlorine is used, chlorine should be further applied to water to produce, with natural or added ammonia, a combined residual effect is shown in Fig. 5-32.

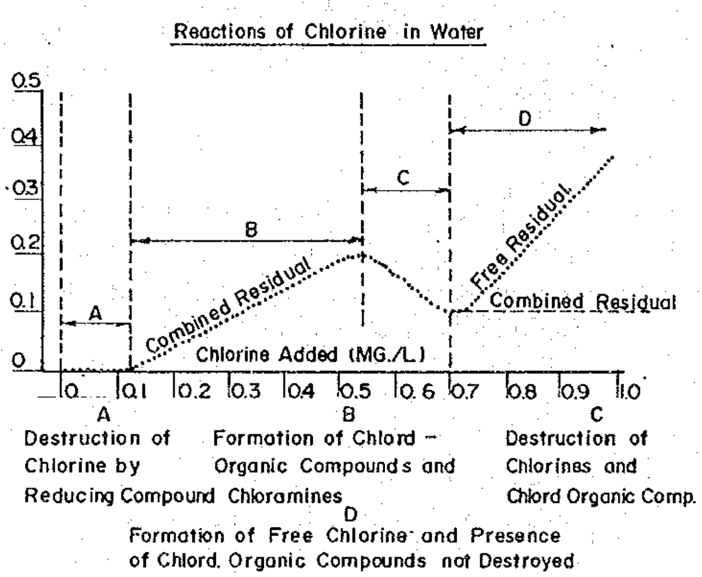


Fig. 5-32. Reactions of Chlorine in Water (Ref. Book No. 1)

Pre-chlorination is an application of chlorine prior to any other treatment. This is commonly used for controlling algae, taste and odour. Post-chlorination refers to the application of chlorine after other treatment processes are over, particularly filtration.

2) Residual chlorine

Following are the methods available to measure residual chlorine in water,

a) Diethyl-para-phenylenediamine
b) Orthotolidine Method

is ordinarily recommendable.

3) Chlorination technology

In the rural areas, dosage should be made densely; but residual by 0.5 ppm (mg/litre) after 30 minutes contact period will be sufficient to achieve ordinary disinfection.

The batch method of mixing is most commonly used. This method consists of density mixing a predetermined volume to a predetermined strength and applying it to water by means of some gravity system. The strength of the batch should not be more than 0.65% of chlorine by weight, as this is about the limit of solubility of chlorine at ordinary temperatures. As an example, 10 g of ordinary bleaching powder of 25 percent density is dissolved in 5 litres of water, giving a stock solution of 500 ppm. For disinfection of drinking water, one part of the stock solution may be added tentatively to 100 parts of the water to be treated. The initial dosage would make the water by 5 ppm. If the chlorine residual after 30 minutes' contact were found more than 0.5 ppm the initial dosage could be reduced in future. Figs. 5-33 and 5-34 show two types of devices which will be used to stock solutions.

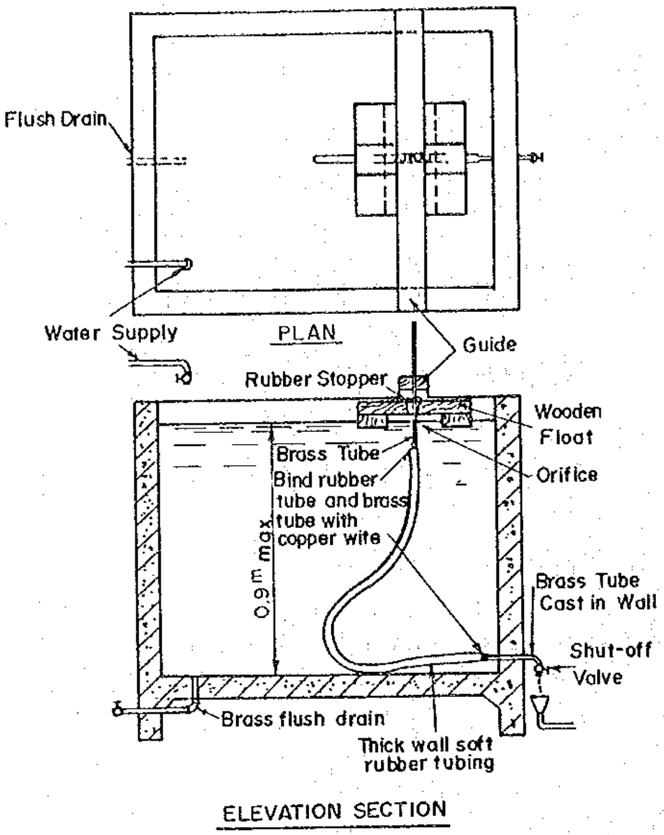


Fig. 5-33. Chlorine Feeding Arrangement(a)
(Ref. Book No. 2)

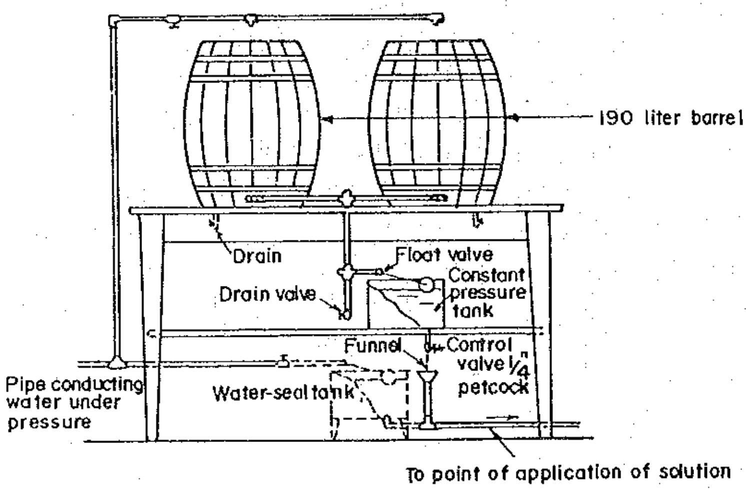


Fig. 5-34. Chlorine Feeding Arrangement(b)
(Ref. Book No. 2)

CHAPTER VI. WATER DISTRIBUTION WORKS

6.1. General Description

Water distribution facilities aim at delivering hygienically safe, readily available water after treated to the beneficiaries. Their capacity should be rendered it possible to meet the hourly maximum water demand, but also supply the amount of water for fire fighting. The distribution facilities comprise of distribution reservoir, pump, elevated tank, pipeline and service pipe.

The water demand at a district varies considerably during a day. Water consumption is highest during the hours when food preparation and washing of clothes are done. During the night the water use will be lowest. (Refer to Fig. 6-1.)

Distribution reservoir serves to accumulate and store water during the night so that water can be supplied during the daytime to meet the maximum water demand.

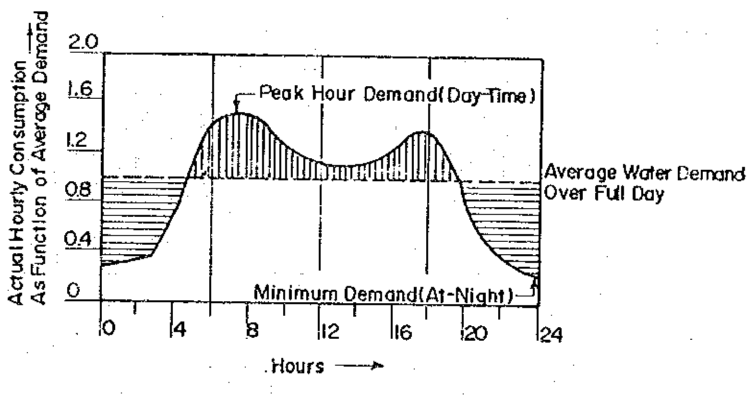


Fig. 6-1. Variation of Water Demand during the Day (Ref. Book No. 1)

6.2. Design Consideration

6.2.1. Distribution System

1) Where there are suitable heights for locating a distribution reservoir in the district, a natural flow-down method is used for water distribution.

   If no suitable height is in the districts, water is delivered by elevated tank which height can ensure the minimum dynamic water pressure, or by the pump pressuring method.

2) For pressure control of the distribution system, elevated tank is usually provided. The following two systems are considered. The distribution system will be usually determined by the system A.

A-type B-type

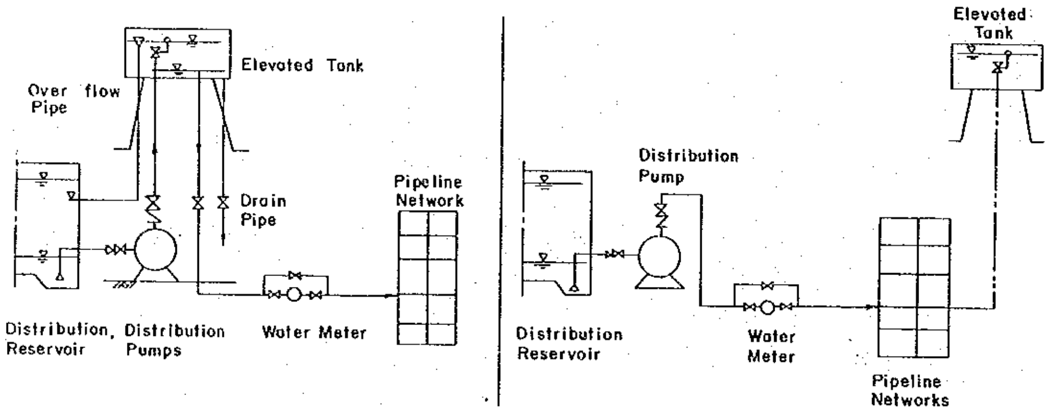


Fig. 6-2. Distribution System

6.2.2. Distribution Water Demand

1) The planned quantity of distributing water is to be the hourly maximum water demand in normal condition or the total of the daily maximum water demand and the quantity of water for fire-fighting in case of a fire, whichever bigger.

2) The daily maximum water demand and the hourly maximum water demand can be referred from Chapter 3.2.4.

3) The quantity of water for fire-fighting is obtained in Table 6-1.

Table 6-1. Quantity of Water for Fire-Fighting (Ref. Book No. 5)

Population	Q
more than 10,000	$0.5 m^3/min.$
less than 10,000	$0.26 m^3/min.$

6.2.3. Water Pressure

1) The minimum dynamic water pressure is recommended to be 1.0 kg/cm^2 at end point of distribution pipes are connected with service pipes.

2) The service connection can be classified as follows:

- House connection
- Yard connection
- Communal tap connection

The service level at target year is to be house connection in principle.

6.2.4. Distribution Reservoir

1) The distribution reservoir is provided to balance the constant supply from treatment plant with the fluctuating water demand in the distribution area. The storage volume should be large enough to accommodate the cumulative differences between the water supply and the demand.

2) A distribution reservoir with a storage volume of 20 to 40 percent of the total daily maximum water demand shall be generally adequate.

The required storage volume can be determined as follows. (Refer to Fig. 6-3.)

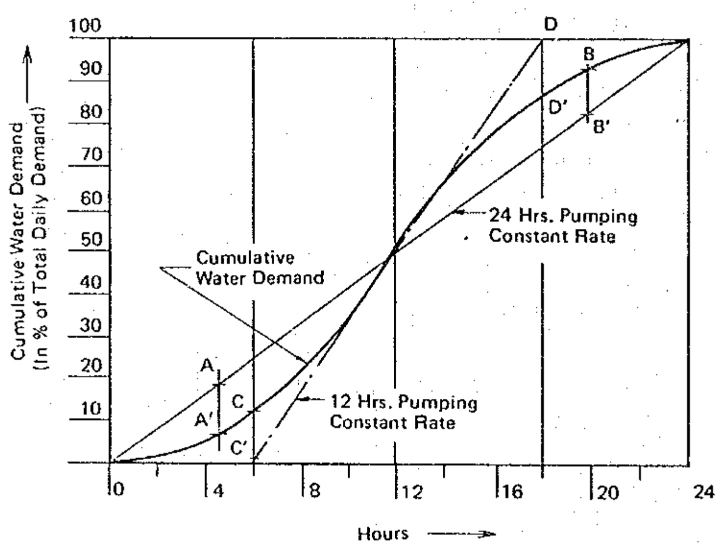


Fig. 6-3. Graphical Determination of Required Storage Volume (Ref. Book No. 1)

The estimated hourly water demand is expressed as a percentage of the total demand over the peak day and plotted in a cumulative water demand curve. The constant supply rate is then drawn in the same diagram as a straight line.

The required volume of storage can now be read from the graph. For a constant supply, 24 hours a day, the required storage volume is represented by A-A' plus B-B', about 28% of the total daily maximum water demand.

If the supply capacity is so high that daily demand can be met with 12 hours pumping a day, the required storage is found to be C-C' plus D-D', about 22% of the total daily maximum water demand. Therefore, the storage volume will be 20 to 40 percent of the total daily maximum water demand including the quantity of water for fire-fighting.

6.2.5. Distribution Pump (Refer to Fig. 6-4)

1) Purpose of Provision of the Pumps

The distribution pumps, which are installed between the distribution reservoir and the elevated tank, deliver the treated water to service area through distribution networks. The treated water is pumped up to the elevated tank where the minimum dynamic water pressures of about 15 to 18 m cannot keep at the distribution reservoir.

2) Type of Pump

The pump to be used for distribution is generally single suction centrifugal type from the viewpoints of total lifting head, suction and delivery conditions of the pump systems.

3) Number of Pumps to be Required

Three unit pumps including one stand-by unit of the same size and specifications shall be provided taking into account changeability of spare parts and convenience of O & M. The prime mover of the pumps is an electric motor for two units and a diesel engine for the remainder.

4) Design Discharge

The design discharge of the distribution pumps should be the hourly maximum water demand.

5) Specifications of Pump and Motor

The preliminary specifications of pump and motor can be determined from the figures and diagrams described in sections 4.4.3 and 4.4.4.

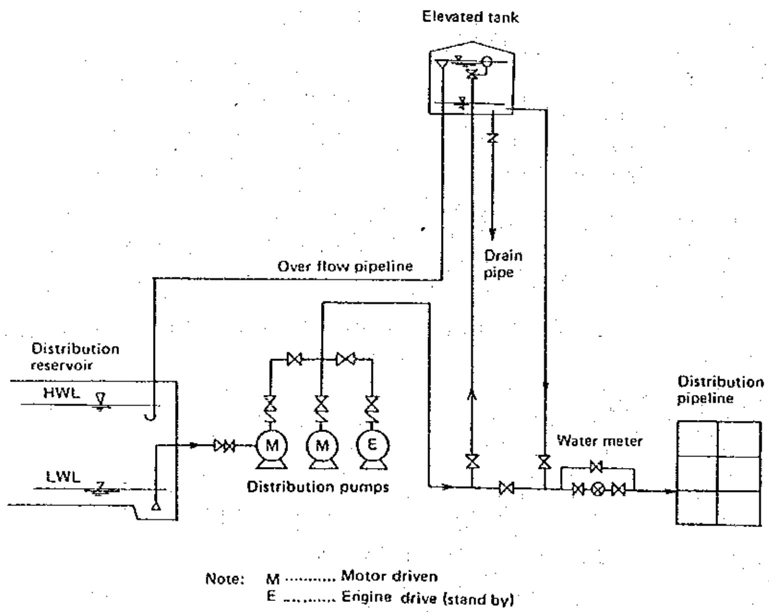


Fig. 6-4. Distribution Pumps Arrangement

6.2.6. Elevated Tank

1) The required volume of elevated tank is 20-10 percent of the total daily maximum water demand including the quantity of water for fire-fighting.

2) Low water level of the tank shall be the level which can ensure the minimum water pressure at all distribution branch ends.

Appropriate low water level is generally 15 to 18 m above the ground level in the Northeastern region of Thailand.

3) Elevated tank is also used for backwash and surface wash of the rapid sand filter.

6.2.7. Distribution Pipe

1) Distribution pattern

There are two main systems of distribution water, the dead-end system and the network system. The network system is generally recommendable.

2) Diameter and velocity of distribution

a) Main distribution pipe shall not be less than 100 mm in diameter. When fire protection is provided, no hydrant can be served by a pipe smaller than 100 mm in diameter. Sub-main distribution pipe shall not be less than 50 mm in diameter.

b) Velocity of flow shall not be more than 1.8 m per second in main pipes and about 0.9 m per second in sub-main. Recommendable average velocity is shown as follows:

Diameter (mm)	Velocity (m/s)
50 - 75	0.6 - 0.8
75 - 150	0.7 - 1.0
200 - 300	0.8 - 1.2
350 - 600	0.9 - 1.4

c) The Hazen-Williams formula is adopted as average velocity formula as mentioned in Chapter 4.5.4.

The hydraulic calculation of pipeline networks is explained in detail as follows:

The following two methods are generally considered:

* Discharge method ; Harday Cross method
* Hydraulic head method; Contact hydraulic head method

The same results can be obtained from the both methods. The former, however, can be applied only for network pipeline. The latter is the method that discharge can be obtained by assuming pressure on every contact, then can be applied for the both type, network pipeline and branch pipeline.

Accordingly, the hydraulic head method is considered to be adequate for calculating the hydraulics of pipeline in sanitary distrrict.

The contact hydraulic head method.

$$qij = 0.27853 \cdot C \cdot Dij^{2.63} \left(\frac{hi-hj}{Lij}\right)^{0.54}$$

Where, qij : quantity of water flow between (i) and j

Dij : diameter of pipeline between (i) and (j)

Lih : length of pipeline between (i) and (j)

hi : dynamic hydraulic pressure at (i)

hj : dynamic hydraulic pressure at (j)

Therefore, the contact equation on (i) will be as follows:

$$\sum_{j=1}^{n} qij + Qi = 0$$

Where, qij : quantity of water flow between (i) and (j)

Qi : quantity of outflow from (i)

Now, if there are (j) pipeline, (N) contacts and (M) distribution plant where dynamic hydraulic pressure is constant, (j) water flow equations and (N-M) dimension linear equation can be obtained.

$$qij = Kij\ (hi - hj)$$

$$Kij = 0.27853.C.Dij^{2.63} \cdot Lij^{-0.54} \cdot (hi-hj)^{-0.46}$$

This calculation is so complicated that it may be done by computer by using a program mentioned in Appendix I.

3) Pipe material

Pipelines frequently cause a considerably high investment, and selection of the suitable type of pipe is important. Pipes are available in various materials, sizes and pressure classes. The most common materials are:

i) Cast iron pipe
ii) Ductile cast iron pipe
iii) Steel pipe
iv) Prestressed concrete pipe
v) Asbestos cement pipe
vi) Polyvinyl chloride pipe
vii) High-density polyethylene pipe

Asbestos cement pipe is generally used because it is most economical in cost and relatively durable if it is handled carefully and buried at adequate depth.

Characteristics of pipes to be laid are indicated in Table 6-1.

Steel pipe can be used under the railway and high crossing.

6.3. Pipe Fittings

6.3.1. Pipes

1) The depth is determined according to the road-surface load. Namely, the burying depth should be more than 100 cm above the pipe under the road, more than 80 cm above the pipe under field.

2) Pipeline foundation

   a) When the pipeline is lain in soft ground, ground improvement is required by the "Soil replacement" method and the method of consolidation by dewatering, including the simple consolidation process.

   b) Sand layer with certain thickness should be lain under pipes.

6.3.2. Valves

1) Sluice valve

   a) Sluice valve should be installed at 500 to 1,000 m intervals.

   b) They are installed on branched points, siphon culverts and and before/after bridges, etc.

   c) They are installed at the points which minimize water-supply suspended area.

2) Air valve

Air valves are installed on convexes of pipeline.

3) Drain valve

For the cleaning of the pipeline, drain valves are installed mean the river, waste gutter or the pipeline end.

4) Stop valve

Stop valves are installed on the service pipe.

6.3.3. Fire Hydrant

Fire hydrants are provided at convenient points for fire-fighting (i.e., near road intersections and branched points). Moreover, they are installed at 100 to 200 m intervals depending upon the existing arrangement of buildings on the pipeline.

6.3.4. Water Meter

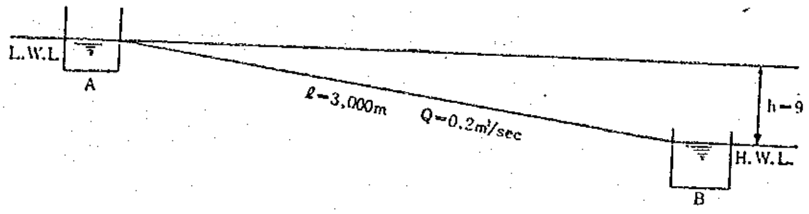
i) Water quantity measuring devices should be installed at the starting point of distribution main pipe, and service connection of users.

ii) It is recommendable to install self-recording type.

Table 6-1. Characteristics of Pipe
(Ref. Book No. 5)

Materials	Merit	Demerit
Cast iron pipe (Inside mortar lining)	(1) Intensive and corrosion resistance (2) Cutting easy (3) Mechanical joint is flexible, expansive and easy to construct	(1) Weak to impact (2) Heavy (3) Need specials protection against joint removed (4) Surface and joint corrosion proof in the exceedingly humus
Ductile cast iron pipe (Inside mortar lining)	(1) Intensive and corrosion resistance (2) Strong to impact (3) Mechanical joint is flexible and expansive (4) Easy to construct (5) Many kinds joints	(1) Heavy (2) Need specials protection against joint remove (3) Need outside lining in humus (4) in case large size pipe impossible to repair from inside
Steel pipe (lining pipe)	(1) Intensive (tension and bend) (2) Strong to impact (3) No need countermeasure to joint remove by welding joint (4) Light (5) Easy manufacturing	(1) Need temperature expansion joint or flexible joint (2) Weak to electric corrosion (3) Take much time welding and lining difficult to construct in spring ground (4) Flexibility is large (large size pipe)
Asbestos cement pipe	(1) Corrosion and electric corrosion resistance are good (2) Joint is flexible and expansive (3) Light and easy to construct (4) Inside roughnsse is not changing (5) Cheap	(1) Shear strength is small (2) Weak to impact (3) Need specials protection against joint removed (4) Easy to erode by water and soil quality
Risid poly venyl chloride	(1) Corrosion and electric corrosion resistance (2) Light, easy to construct (3) Possible to adhere (4) Inside roughness is not changing (5) Cheap	(1) Weak to impact at low temperature (2) Weak against heatness, ultraviolet rays and organic solvent (3) Caution to fire solvent cement (4) Need temperature expansive and flexible joint

[Example 6.1) Determination of Pipe Diameter



natural flow-down

a) C = 130 (mortar lining)

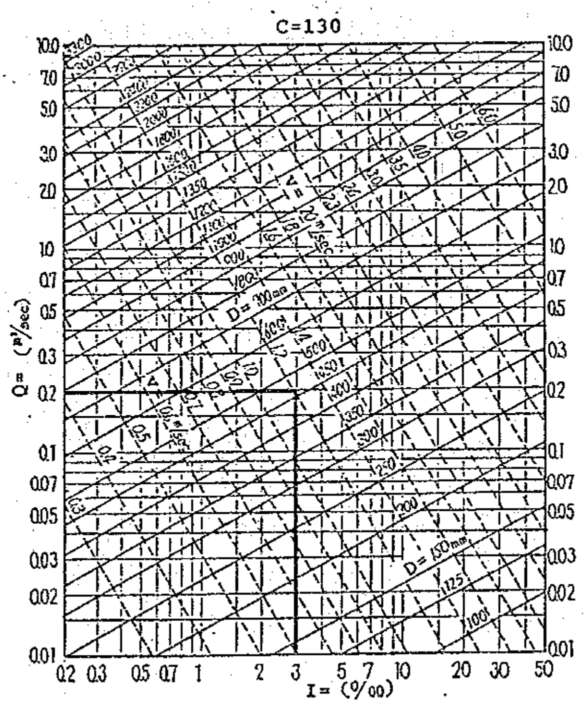
b) $Q = 0.2\ m^3/sec$

c) 1 = 3,000 m

d) h = 9 m

e) I = h/1 = 9 3,000 = 0.03

By using the Hazen-William's formula diagram



Hazen-William's Formula Diagram

Pipe diameter will be 500 mm (The velocity will be 1.2 m/s in this case.)

[Example 6.2] Determination of Pump Type

1) Design data

   a) Planned treatment capacity = 2,000 m^3/day
   b) H.W.L. of receiving well = 159.0 m
   c) H.W.L. of river intaking = 148.0 m
   d) L.W.L. of river intaking = 139.0 m
   e) Distance between intake to treatment plant = 3,000 m
   f) Diameter of transmission pipe = 300 mm

   Dimensions and type of intake pump can be determined in the following procedures.

2) Calculation

   Planned intake quantity = 2,000 x 1.1 = 2,200 m^3/day = 0.025 m^3/s

   Hydraulic gradient (I) will be 0.42‰ in case that C-value is 140, referring from Fig. E-5.

   * Friction loss = $0.42 \times \frac{3{,}000}{1{,}000} = 1.26$ m
   * Actual head = 159.0 - 139.0 = 20.0 m
   * Other head losses = 2.0 m

   Total head ≑ 24.0 m

   Number of pump is two (one is ordinary, another is stand-by.

   The capacity of a pump is 1.5 m^3/min.

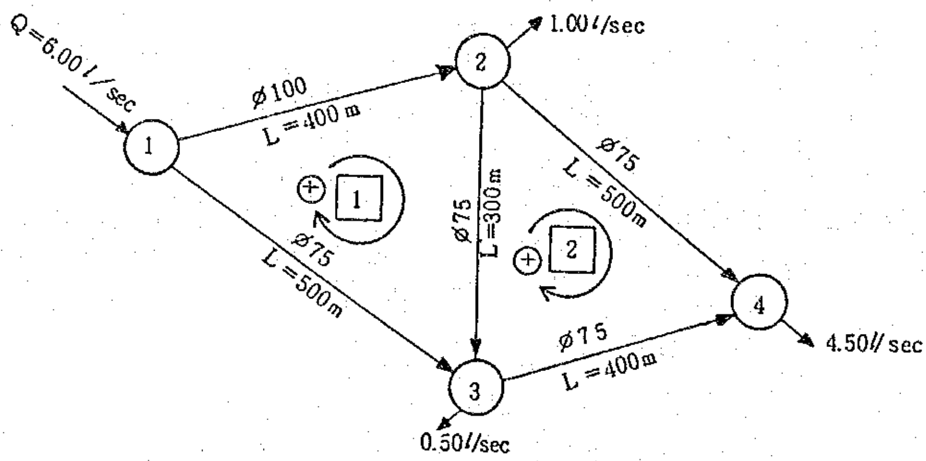
   Therefore, the required pump will be a single-suction centrifugal pump (H = 24.0 m, Q = 1.5 m^3/min), referring from Fig. 4-3.

[Example 6.3] Determination of the Capacity of Distribution Facilities

1) Population served 16,000
2) Daily maximum demand, 16,000 x 150/24 = 100 m^3/h
3) Hourly maximum demand .... 150 m^3/h
4) Quantity for fire-fighting .... 30 m^3/h
5) Planned quantity for distributing water
   $100 + 30 = 130\ m^3/h < 150\ m^3/h$
6) Capacity of storage reservoir
   Referring from Appencix H, the capacity of storage reservoir will be 6 hours water demand of daily maximum. Therefore, 100 m^3/h x 6 h = 600 m^3
7) Capacity of elevated tank
   Referring from Appendix H, the capacity of elevated tank will be 2 hours water demand of the daily maximum. Therefore, 100 m^3/h x 2 h = 200 m^3
8) Capacity of distribution facilities
   a) Population 16,000
   b) Hourly maximum water demand per capita .... 225 l/c/d
   c) Water supply level .... House connection
   d) Planned water pressure....... 1.0 kg/cm^2
   e) Height of elevated tank ..... 18.0 m

[Example 6.4] Hydraulic Calculation of Pipeline Networks

1) Design data



Note: C-value will be 140 for all pipelines.

Fig. S6-1. Network Pipeline Head Method

The hydraulic calculation of the pipeline will be made by the contact hydraulic head method.

For the pipeline as shown in Fig.S6-1, the hydraulic will be calculated repeatedly. Number of pipelines is 5, number of contacts is 4, number of distribution plant is 1. Therefore, there will be five water flow equations and three dimensions linear equation.

a) First trial

The dynamic hydraulic pressure (h_1) at contract is known, h_1 = 40.0 m.

h_2, h_3, h_4 is assumed to be 39.0 m, 38.0 m and 37.0 m, respectively.

$$\begin{cases} -(k_{21}+k_{23}+k_{24})\cdot h_2+(k_{21}\cdot h_1+k_{23}\cdot h_3+k_{24}\cdot h_4)=Q_2 \\ -(k_{31}+k_{32}+k_{34})\cdot h_3+(k_{31}\cdot h_1+k_{32}\cdot h_2+k_{34}\cdot h_4)=Q_3 \\ -(k_{42}+k_{43})\cdot h_4+(k_{42}\cdot h_2+k_{43}\cdot h_3)=Q_4 \end{cases}$$

$k_{12}=k_{21}=0.27853\ C\cdot D_{12}{}^{2.63}\cdot L^{-0.54}\cdot|h_1-h_2|^{-0.46}$
$=0.27853\times140\times0.100^{2.63}\times400^{-0.54}\times|40.0-39.0|^{-0.46}$
$=0.0036$

$k_{13}=k_{31}=0.27853\times140\times0.075^{2.63}\times500^{-0.54}\times|40.0-38.0|^{-0.46}$
$=0.0011$

$k_{23}=k_{32}=0.27853\times140\times0.075^{2.63}\times300^{-0.54}\times|39.0-38.0|^{-0.46}$
$=0.0020$

$k_{24}=k_{42}=0.27853\times140\times0.075^{2.63}\times500^{-0.54}\times|39.0-37.0|^{-0.46}$
$=0.0011$

$k_{34}=k_{43}=0.27853\times140\times0.075^{2.63}\times400^{-0.54}\times|38.0-37.0|^{-0.46}$
$=0.0017$

$Q_2=0.001\,m^3/sec$, $Q_3=0.0005\,m^3/sec$, $Q_4=0.0045\,m^3/sec$

Rearrange by substituting these figuers into the equation.

$$\begin{cases} -0.0067\,h_2+0.0020\,h_3+0.0011\,h_4=-0.1430 \\ 0.0020\,h_2-0.0048\,h_3+0.0017\,h_4=-0.0435 \\ 0.0011\,h_2+0.0017\,h_3-0.0028\,h_4=\ \ 0.0045 \end{cases}$$

by solving this,

$h_2=38.845^m$, $h_3=38.325^m$, $h_4=36.922^m$

Accordingly,

$|38.845-39.0|=0.155>0.001$ ········· NO
$|38.325-38.0|=0.325>0.001$ ········· NO
$|36.922-37.0|=0.078>0.001$ ········· NO

(b) The second trial

$k_{12}=k_{21}=0.27853\times140\times0.100^{2.63}\times400^{-0.54}\times|40.0-38.845|^{-0.46}$
$=0.0034$

by the same way

$k_{13}=k_{31}=0.0012$, $k_{23}=k_{32}=0.0027$, $k_{24}=k_{42}=0.0011$
$k_{34}=k_{43}=0.0014$

$$\therefore \begin{cases} -0.0072\,h_2+0.0027\,h_3+0.0011\,h_4=-0.1350 \\ 0.0027\,h_2-0.0053\,h_3+0.0014\,h_4=-0.0475 \\ 0.0011\,h_2+0.0014\,h_3-0.0025\,h_4=\ \ 0.0045 \end{cases}$$

by solving this equation

$h_2=38.786,\ h_3=38.440,\ h_4=36.792$

$|38.786-38.845|=0.059>0.001$ ……… NO

$|38.440-38.325|=0.115>0.001$ ……… NO

$|36.792-36.922|=0.130>0.001$ ……… NO

(c) The third trial

$k_{12}=k_{21}=0.27853\times 140\times 0.100^{2.63}\times 400^{-0.54}\times|40.0-38.786|^{-0.46}$

$=0.0033$

by the same way

$k_{13}=k_{31}=0.0012,\ k_{23}=k_{32}=0.0032,\ k_{24}=k_{42}=0.0011$

$k_{34}=k_{43}=0.0013$

$$\therefore \begin{cases} -0.0076\,h_2+0.0032\,h_3+0.0011\,h_4=-0.1310 \\ 0.0032\,h_2-0.0057\,h_3+0.0013\,h_4=-0.0475 \\ 0.0011\,h_2+0.0013\,h_3-0.0024\,h_4=\ \ 0.0045 \end{cases}$$

$\therefore h_2=38.743,\ h_3=38.457,\ h_4=36.713$

$|38.743-38.786|=0.043>0.001$ ……… NO

$|38.457-38.440|=0.017>0.001$ ……… NO

$|36.713-36.792|=0.079>0.001$ ……… NO

(d) The fouth trial

$k_{12}=k_{21}=0.0032,\ k_{13}=k_{31}=0.0012,\ k_{23}=k_{32}=0.0035$

$k_{24}=k_{42}=0.0011,\ k_{34}=k_{43}=0.0013$

$$\therefore \begin{cases} -0.0078\,h_2+0.0035\,h_3+0.0011\,h_4=-0.1270 \\ 0.0035\,h_2-0.0060\,h_3+0.0013\,h_4=-0.0475 \\ 0.0011\,h_2+0.0013\,h_3-0.0024\,h_4=\ \ 0.0045 \end{cases}$$

$\therefore h_2=38.708,\ h_3=38.446,\ h_4=36.691$

$|38.708-38.743|=0.035>0.001$ ……… NO

$|38.446-38.457|=0.011>0.001$ ……… NO

$|36.691-36.713|=0.022>0.001$ ……… NO

(e) The fifth trial

$k_{12}=k_{21}=0.0032,\ k_{13}=k_{31}=0.0012,\ k_{23}=k_{32}=0.0037$

$k_{24}=k_{42}=0.0011,\ k_{34}=k_{43}=0.0013$

$$\therefore \begin{cases} -0.0080\,h_2+0.0037\,h_3+0.0011\,h_4=-0.1270 \\ 0.0037\,h_2-0.0062\,h_3+0.0013\,h_4=-0.0475 \end{cases}$$