FEASIBILITY STUDY & MASTER PLAN

FOR

PATTANI MUNICIPALITY

OVERSEAS TECHNICAL COOPERATION AGENCY

GOVERNMENT OF JAPAN

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Chapter 1 Introduction

1.1 Services of Japanese Experts

The Government of Thailand asked Japanese Government to provide water works engineers under the Colombo Plan and to studying the feasibility of water supply plans for about 20 municipalities which had confronted with stortage of water. In response to the request, the Overseas Technical Cooperation Agency of the Japanese Government (OTCA), dispatched the First Japanese Experts Team consisted of 6 engineers leaded by Mr. N. Ichinohe in March 1968.

Since then, the First Japanese Experts Team surveyed about 8 municipalities and sumitted their report for Pattani Municipality in July 1969. This report must be evaluated as a preliminarly feasibility study, but not be emphasized as a master plan.

In addition to the First Japanese Experts Team, Japanese Government agreed to dispatch other experts under the Colombo Plan in response to the additional request from Government of Thailand. The Second Japanese Experts Team was organized consisting of 4 engineers leaded by Dr. S. Naito and dispatched to Bangkok after the First Team was expired on March 1970. Since then, the Second Japanese Experts Team surveyed again 15 municipalities and submitted their report for Pattani Municipality in June 1970. This report also must be evaluated as a preliminary feasibility study in addition to the report from the First Japanese Experts Team.

After being investigated for 15 municipalities by the Second Japanese Experts Team, the both parties talked about further steps in which an important municipality shall be taken care in master plan and/or detailed design. So as to provide sufficient time to design their schemes in details to the most theoretical and accurate degree since an important municipality would be challenged into a request for foreign loan, the higher priority has been noticed in four municipalities Chiang Mai, Nakohn Rachasima, Pattani and Hua-Hin.

Among four municipalities, the Japanese Government agreed their use of budget in special consideration of detailed design for Chiang Mai and Nakohn Rachasima, but it has not applied into other two municipalities, Pattani and Hua-Hin. Therefore, the main purpose of this report is to investigate the present situation of water supply system in Pattani, including the survey of served population, consumption and production of water, and to make the master plan involving the feasibility study both an urgent works and future plan.

The report, accordingly, includes the study of existing water supply system and future plan. Financial projection for future plan is also carried out. It is regretted however that the sufficient data were not available during the survey and the further attempt has to be undertaken for satisfactory collection of necessary informations.

1.2 Survey and Study

1.2.1 Field Survey

The field survey was conducted during their stay at Thailand by the Second Japanese Experts Team consisting of following engineers.

Chief: Sachiho Naito	Former Professor of Chuo University

Deputy Chief:	Susumu Sakaguchi	Former staff of Sanitary Engineering Laboratory in Chuo University
Member:	Koichi Degawa	ditto
Member:	Kenji Sugi	ditto

1.2.2 Preparation of Report

The Second Japanese Experts Team was commissioned to prepare the whole report under the supervision of Mr. Damrong Cholvijarn, Director General, Public & Municipal Works Department, Ministry of Interior, the Government of Thailand.

1.3 Source of Data

The various data for the study was collected at Provincial Water Supply Division, Public and Municipal Works Department, Pattani Municipality and the branch office of Royal Irrigation Department. The map and statistical data was informed from the Provincial Water Supply Division mainly.

Field survey was done by the Second Japanese Experts Team in leveling and plain table surveying roughly for the transmission line and purification plant, but these shall be also checked latter when detailed design would be started.

1.4 Concept of Planning and Designing

Planning is interpreted in different ways by different people. The use of planning methods may be divided very broadly, among several stages of overall planning. Planning can best be done by arranging a meeting or a series of meetings at which all the interested participants can discuss the future procedures.

When any engineers would make a plan of water supply system, it is as usual that they would consider about it emphasizing on a farsighted policy. Simultaneously, it should be feasible and adaptable in actual construction from the views of not only technically but also budgetary. However, according to the condition of budgetary limitation, it is not always completely feasible even though it is adaptable technically. In other word, although anyone knows about what a larger container is always suited for a smaller material, a smaller container should be ready for a smaller material, due to the budgetary limitation.

On the contrary, it is absolutely necessary to plan the long range period of planning such as 20 to 30 years if some one would suppose to borrow the aids from the World Bank or Asian Development Bank although there is still doubt about it. Further among the long range of planning, it is needed to separate any programme into few stages of programme, and the first stage of this programme might be covered by detailed designing as a general rule.

a) Master Planning

The time being, the word of Master Planning is not always used as it is correctly defined.

There are many terms in which the word of master is used in combination with other word, such as master-key, and master-hand which are understood as it is superior wherever it goes. Following these understanding, the Master Planning might be defined as only one solution, but it may be possible after the comparative study such as feasibility study. This kind of understanding is also proved by the fact that there is a sentence of "feasibility study of master planning."

On the other hand, it is not sure whether the Master Planning is always adaptable, because the scheme mentioned in Master Planning is little far from the present circumstances, even thought it will be good enough - might be best - to improve present situation. In other word, the best one is not always accepted by a country which is confronted with a lack of budget. Further, the best one might be extravagant from standpoint of present national economy.

In occasion of Master Planning, it is required that the recommendation is done with regard to the number of phases. However, although any one understand well that the first stage of Master Planning should be forwarded in order to satisfy the long range planning such as 20 to 30 years, he also has to consider about the circumstances which are confronted with strict needs of water. It may say such a condition as Emergency Programme.

b) Emergency Programme

It is little difficult problem whether the Emergency Programme must be prior to the first stage of Master Planning. More, there are great doubts about the fact which the Emergency Programme must be considered as almost same scheme comparring with the first stage of Master Planning, although both programme should be coincided in general concept. For example, in case of planning of the Emergency Programme, any one may assume the design population in range of next decade or less, but the first stage of Master Planning may be one part of long range programme such as 20 to 30 years, even if it is least requirement of large scale of planning. More, taking the data of the water demand per capita, the Emergency Programme is limited to satisfy supplying of drinking water, but the first stage of Master Planning should be considered for not only drinking water but also fire fighting water.

For example, in case of large scale of planning (Master Planning) designed by surface water utilization, cheaper deep wells would be dug in connection with smaller size of pipe line in the stage of Emergency Programme. The deep wells should be scheduled in the Master Planning as supplementary intakes. Besides of intakes, the distribution system should be avoided from duplex investments in pipe line. In other word, the pipe line would be allowed to lay on the line from smaller size to larger size in the direction of flow. More, the pipe would be temporarily allowed to set on the surface of earth, when it could not be matched on the Master Planning. In any case, it is absolutely necessary to avoid from being wasted of any materials, even if it is Emergency Programme.

c) Rehabilitation Work

In addition to Emergency Programme, it is also pointed out that existing facilities of water supply should be carefully improved until it would reach to normal condition for operation. Abnormal condition might be caused by a lack of budget, knowlege of operator and/or supervisor and unconcerned behaviour against compulsory works, however it is absolutely necessary to emphasize promptly present capacity of water supply facilities in order to determine value and amount of extension work.

From these point of views, there is importance about Rehabilitation Work by which any equipment and apparatus shall be recovered and improved until normal capacity recognized on design criteria and standard. After the possibilities of reducing water losses and increasing own capacity by being rehabilitated, are not sufficiently explored, sometimes additional consideration would mostly taken care of it.

d) Feasibility Study

It is common that comparative studies would be taken care in order to consider about overall programme including the Emergency Programme on the occasion of making Master Plan. In general speaking, the preparation for fund lending from abroad has required proportionately much time and effort. The reason are not so much deficiencies in detailed design but unsatisfactory analysis of comparative studies. This refers to the identification and analysis of alternative ways under the Feasibility Study.

In case of feasibility study, it is possible to study three alternative plannings which are designed by changing the period of plan and the future prospect of water demand. Thus three alternatives would be maximum and it is apparently much cost by if more than three might be thought.

When it is technically difficult to compete among two alternative designs, usually economic comparative study would be done. It is always clear that the comparative study has a big advantage which partial changing of step would not influence to following step in designing.

Chapter 2 Summary & Recommendation

2.1 History and Present Situation

Ever since the very beginning of the municipality planning and construction work which started on October 1955 using construction expenditure 1,510,000 Bahts, the municipality had endeavoured to provide sufficient amount of water to the Pattani. However, the increasing needs are so rapid that the supply had never been able to satisfy them. The maximum capacity of the supply was 75 m^3/hr. using three deep wells (q_1 = 20 m^3/hr, q_3 = 15 m^3/hr. with treatment, q_2 = 40 m^3/hr. without treatment), indicating the existence of fairly long period of shortage of supply within the year. The fact is, however, that there exists almost cronic shortage of water supply all through the year, because of the added difficulties of the followings in addition to the shortage of supply itself.

a) The treatment capacity was not enough to cover the excess amount of iron contents which had existed in q_1 and q_2 wells.

b) The fair amount of supply was lost due to the leakage on the pipes and valves.

c) Insufficient pipe lines for the distribution system was causing uneven distribution of supply to all the areas covered by existing system.

d) House meters was not working correctly due to several reasons and together with the inadequate current tariff system for water consumption, this seems to be inviting the wastage of water by the consumers.

It was natural, therefore, that action was to be taken on beginning of 1970 to ensure the shortage of water. Thus, another wells q_4 and q_5 was dug in addition to the purification plant capable of 80 m^3/hr. This extension work was completed at the end of 1970. Since then, up to now, three wells q_2, q_4 and q_5 have been used in service, in addition to the new purification plant 80 m^3/hr after withdrawing old treatment plant capable of 20 m^3/hr. In consequence, the existing wells and treatment plan look like normal in operation without further complainment of serious shortage of water, but it is easily expected that another complainment would be raised in next few years because of another shortage of water.

It has been reported from Pattani municipality that the amount water supply per year is 584,000 m^3 at 1970 in comparison with present capacity of the plant as 2,880 m^3/d x 365 = 1,051,200 m^3/year. This shows the effective percentage as about 55.6%. Therefore, it shall be improved as quickly as possible to increase the effective percentage about 70% after repairing the pipes, valves and water meters.

2.2 Need for Action

The Pattani municipality used the primary procedure over the past decade to combat the water shortage by means of drilling the wells while this procedure has been reasonably successful in the past even though decreasing groundwater levels and increasing iron content. However, it should be stated that the problem have resulted in critical conditions and immediate action is required to overcome them.

2.2.1 Future Population and Water Demand

The following future projection are arrived at as a result of surveys and studies on the water supply in Pattani municipality on the basis of which the proposal for action is recommended.

Fig 2. 1: Location of Pattani Municipality

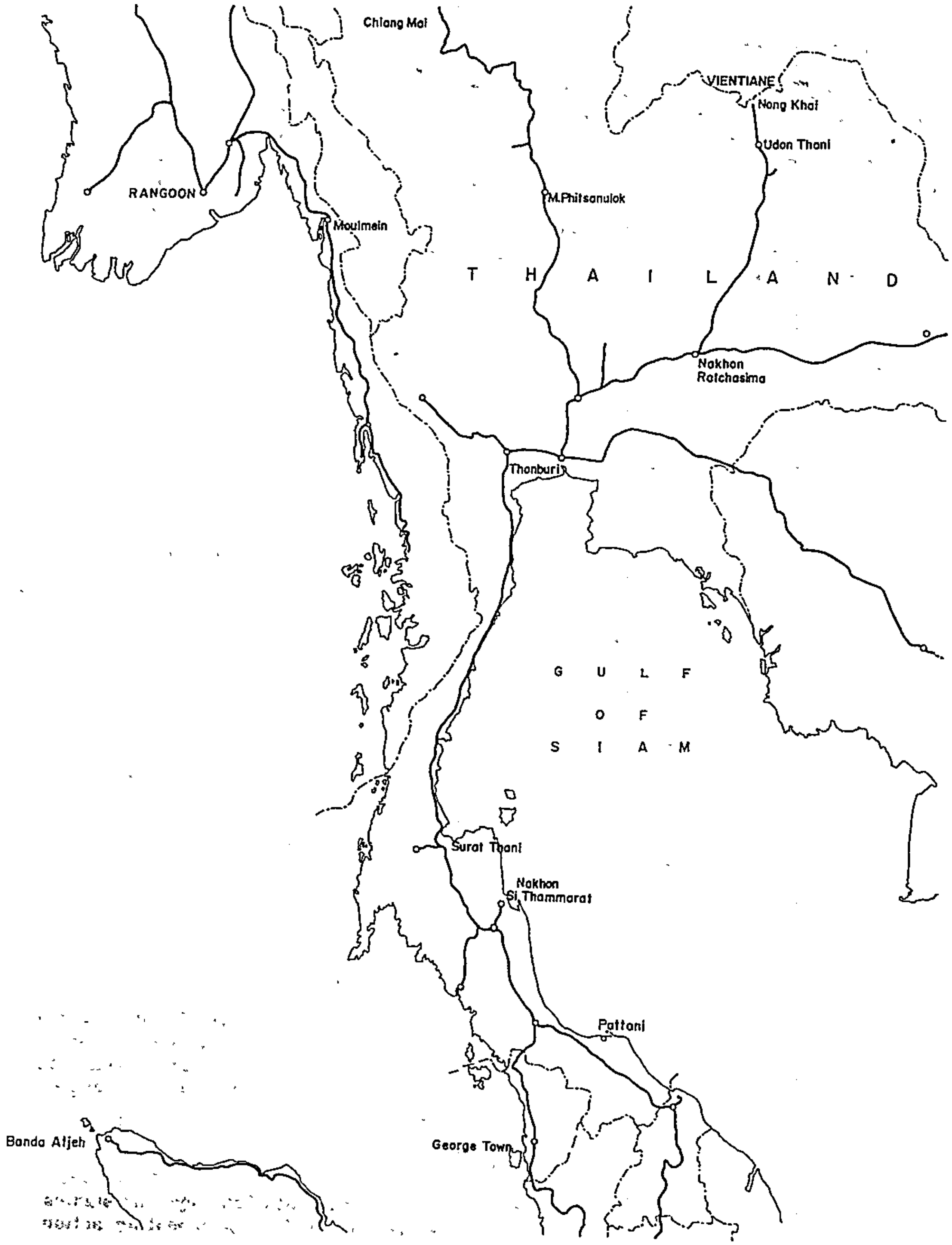
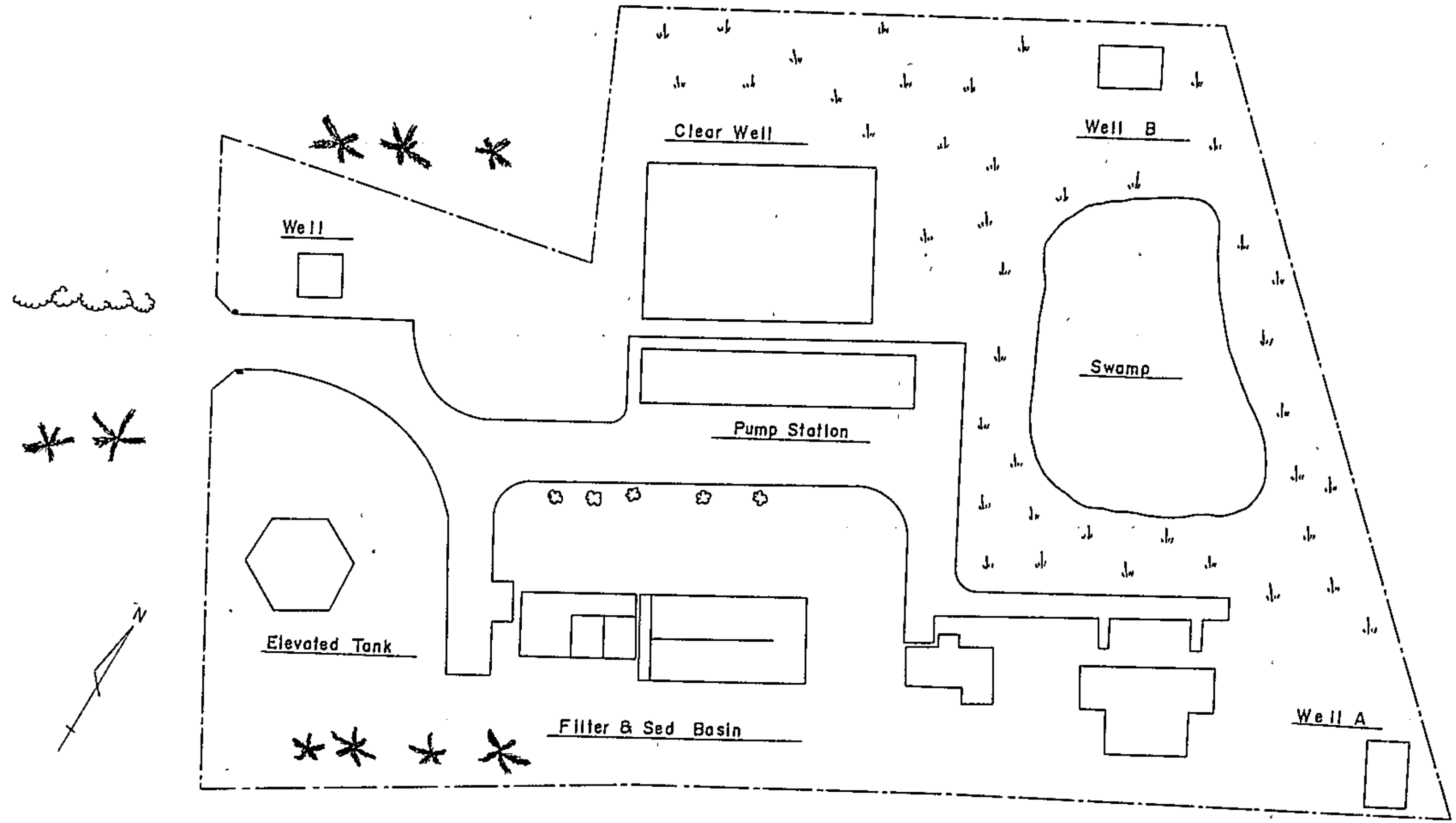


Fig. 2.2 Treatment Plant in Pattani



1) The future population is estimated on the basis of power function's estimation as shown in the following.

Year	Population
1980	33,800
1990	43,600
2000	53,800

2) The house connection is estimated as shown in the following.

Year	House Connection(%)
1980	65
1990	70
2000	75

3) The daily maximum consumption per capita is estimated as shown in the following.

Year	Consumption(1/c/d)
1980	210
1990	230
2000	250

4) Additional need for outside of municipality is estimated as shown in the following.

Year	Need(m^3/d)
1980	500
1990	600
2000	800

5) Effective percentage is determined as 70% after allowing leckage and wastage.

6) The total water demands by year in future program are shown as follows.

Year	1980	1990	2000
Served Population	33,800	43,600	53,800
House Connection(%)	65	70	75
Maximum Daily Demand (1/c/d)	210	230	250
Maximum Daily Demand (m^3/d)	4,620	7,020	10,090
Outside Need(m^3/d)	500	600	800
Total Demand(m^3/d)	5,120	7,620	10,890
Average Daily * Consumption(m^3/d)	3,414	5,080	7,260
Average Consumption (m^3/year)	1,246,110	1,854,200	2,649,900
Effective(%)	70	70	70
Yearly Consumption (m^3/year)	872,277	1,297,940	1,854,930

*: Average Daily Demand = Total Demand/1.5

2.2.2 Proposed Action

Supposing period of one year respectively for construction and for detailed design of the first stage, it can be expected to complete the first stage in sometime around 1974. It is unfortunate therefore that the Japanese Experts Team has no alternative but to conclude that there is no drastic solution for water supply shortage until the year 1974. While the period of shortage of water, the present situation of water shortage can be cased to great extent by rehabilitating the existing water supply facilities.

First, all the leakage of pipes, valves and meters should be repaired, as it is the commonsense of water supply engineering. The next step should be to rehabilitate purification facilities in which appropriate dosage of chemicals shall be scheduled.

2.3 Recommendation

2.3.1 Construction Program

1) Rehabilitation Works (1972-1973)

There is no drastic solution for shortage of water until the year 1974 when the first stage is expected to be completed. Therefore, firstly, all visible leakage on the distribution system should be repaired. Second, water meter should be installed at each service connection and the water charges should be collected according to the meter reading.

Regarding to rehabilitation work of existing treatment facilities, there are two alternative idea, such as temporary improvement to be used for existing underground water and permanent improvement to be used for new surface water. Among two improvement, the temporary improvement is recommended. It would cover only chemical feeding and rapid mixing of chemical coagulation, because there is only one problem about iron removal in existing underground water. Necessary specification of the temporary improvement is new installations of lime feeder, chlorinator, volti-mixer and their attachment.

2) First Stage Works (target year 1980)

There are two alternates of extension project such as overall usage of underground water (extension schedule 1) and combined usage of underground and surface water (extension schedule 2), although the former has higher priority than the latter from the views of cheaper construction cost.

Extension Schedule 1:

Intake:	Two wells, 2,100 m^3/d x 2
Treatment:	Volti-Filter, 2,400 m^3/d x 1
Distribution:	Main Distribution Pipe, 500 mm - 100 mm, L = 14,150 meters.

Extension Schedule 2:

Intake:	Intake gate and storage basin, 8,100 m^3/d x 1
Treatment:	Complete treatment, 2,400 m^3/d x 1
Distribution:	Main Distribution, as well as the extension schedule 1.

3) Second Stage Works (target year 1990)

Extension Schedule 1:

Intake:	One well, 2,400 m^3/d x 1
Treatment:	Volti-Filter, 2,400 m^3/d x 1
Distribution:	Branches, 150 mm - 100 mm, L = 10,970 meters.

Extension Schedule 2:

Intake:	none
Treatment:	Complete treatment, 2,400 m^3 x 1
Distribution:	Branches, as well as the extension schedule 1.

4) Third Stage Works (target year 2000)

Extension Schedule 1:

Intake:	One well, 2,100 m^3/d x 1
Treatment:	Volti-Filter, 3,300 m^3/d x 1
Distribution :	none

Extension Schedule 2:

Intake:	none
Treatment:	Complete treatment, 3,300 m^3/d x 1
Distribution:	none

5) Summary of Construction Cost

The summary of construction cost in given in Table 1.1. These costs are based on following rules.

a. Constrution costs are calculated taking into consideration the size of the various works, based on parallel examples in Japan.

b. Cost of materials, pumps, electric equipment, cast iron pipe, etc., which are imported from foreign countries include the prices CIF Bangkok, landing costs and custom duties. In case the diameter of the pipe is less than 300 mm asbestos cement pipe made in Thailand shall be used.

c. Land costs are calculated at 20,000 Bahts per Rai, not including compensation costs.

d. Reserve funds for unforeseen costs shall be 10% of the construction cost.

e. Expenditures are estimated to be 20% of the local currency costs for supervision, office expenses, construction site expenses, and other miscellaneous use.

f. Administrative expenditures of Thai Government Agencies and its delegated representatives are estimated as 4% of total construction costs.

Financial plan and water charges was based on the allocation of foreign and local currencies as follows.

a. Foreign Currency

Interest Rate: 4.0%
Term of Loan: 20 years
Grace Period: 5 years

b. Local Currency

Interest Rate: 8.0%
Term of Loan: 30 years
Grace Period: 5 years

c. Amortization

Foreign currency: 15 equal annual payments
Local currency: equal yearly payments over a 25 year period.

d. Charge for served water

should be less than 3.0 Bahts.

Table 1.1 Summary of Construction Cost

I. Underground Water:

(unit: Bahts)

Item	Total Cost	1st Stage	2nd Stage	3rd Stage
1) Intake	1,560,000	780,000	390,000	390,000
2) Raw Water Main	580,000	320,000	130,000	130,000
3) Water Treatment Plant	5,340,000	3,000,000	840,000	1,500,000
4) Distribution System	10,940,000	7,260,000	3,000,000	680,000
5) Electric Equipment	1,900,000	1,500,000	300,000	100,000
Sub-Total	20,320,000	12,860,000	4,660,000	2,800,000
Engineering Fee	1,010,000	640,000	230,000	140,000
Administration Cost	800,000	510,000	180,000	110,000
Reserve	2,030,000	1,290,000	460,000	280,000
Grand-Total	24,160,000	15,300,000	5,530,000	3,330,000

II. Surface Water:

Item	Total Cost	1st Stage	2nd Stage	3rd Stage
1) Intake	3,260,000	2,800,000	230,000	230,000
2) Raw Water Main	440,000	440,000	0	0
3) Water Treatment Plant	7,430,000	4,900,000	1,150,000	1,380,000
4) Distribution System	10,940,000	7,260,000	3,000,000	680,000
5) Electric Equipment	1,900,000	1,500,000	300,000	100,000
Sub-Total	23,970,000	16,900,000	4,680,000	2,390,000
Engineering Fee	1,200,000	840,000	240,000	120,000
Administration Cost	946,000	670,000	180,000	96,000
Reserve	2,424,000	1,690,000	500,000	234,000
Grand-Total	28,540,000	20,100,000	5,600,000	2,840,000

Chapter 3 The Existing Water Works

3.1 Water Works Facilities

3.1.1 General

At present, the Pattani Water Works belong to Pattani Municipality, so that any responsibility of construction and maintenance is not covered by the Public and Municipal Works Department (as stated as PMWD hereinafter), the Ministry of Interior, Government of Thailand. However, the PMWD shall be taken in charge of technical assistanceship in which any design and specification is written or approved by the PMWD.

Long time ago, the Pattani Municipality constructed simple water treatment facility nearby the Pattani River, but almost two decades latter, the natural river had been influenced by tidal water in result of salty teste. It is interested to hear about this problem from Mayor of Pattani that when the tidal water came to the intake flowing through lower part of river, the suction pipe was lifted up as good as taking the water from surface of river flow. However, consequently the municipality gave up to take the water from the Pattani River, and constructed again new water supply system composed by deep well and treatment plant which was to remove iron from the underground water. The water supply system pumped up from deep wells was operated until 1970 with great difficulty which was caused by heavy iron in deep well water, under capacity of 20 m^3/hr. with treatment and 40 m^3/hr. without treatment.

Fortunately, however, another new water supply system has been constucted and completed on the end of 1970, under the capacity of 80 m^3/hr. composed by two deep wells and water treatment plant. These were uniquely planned and designed by PMWD, but these new wells have been also contaminated by heavy iron against their expectation. It is recommended that new water treatment plant shall be improved by means of additional chemical dosing in addition to present chemical dosing.

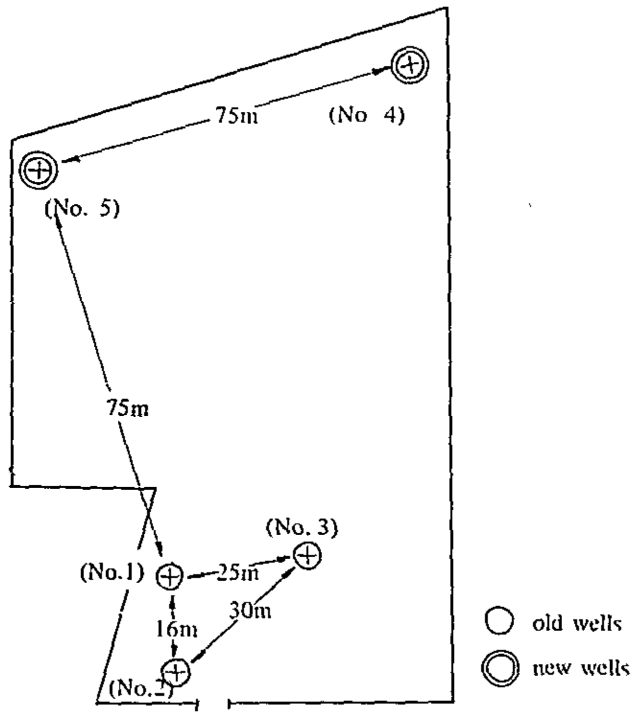
3.1.2 Water Source

Existing facilities of water source are as follows, including old and new wells, as shown in Fig-3.1.

Old Wells:	q_1 = 20 m^3/hr,	63 metres deep,	125 mm in dia.
	q_2 = 40 m^3/hr,	114 metres deep,	250 mm in dia.
	q_3 = 15 m^3/hr,	63 metres deep,	100 mm in dia.
New Well:	q_4 = 80 m^3/hr,	92 metres deep,	250 mm in dia.
	q_5 = 80 m^3/hr,	92 metres deep,	250 mm in dia.

Old well q_1 and q_2 had been operated mercilessly and driven by old style pump until new wells was installed, but at present there are only three wells under the operation, such as q_2, q_4 and q_5. In other word, two old wells, q_1 and q_3 are now out of order, since end of 1970.

Fig 3.1 Wells of Pattani Plant



Furthermore, due to the short distance of each wells shown in Fig-1, above three wells might not be able to take underground water as much as mentioned above, if each well would operate at same time. Regarding this matter, two wells can be calculated by following equation.

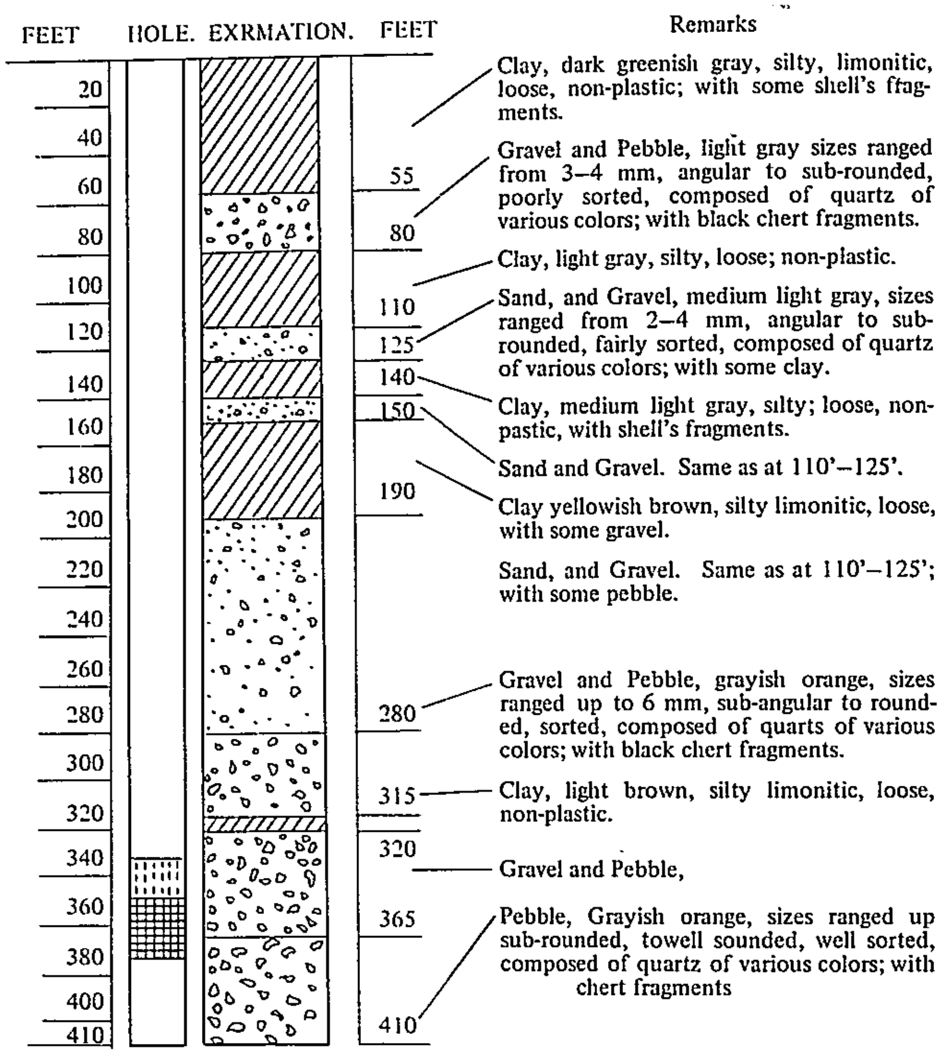
Thiem Equation: $Q = 2.73\ b\ k \dfrac{H - h}{\text{Log}\frac{D}{r}}$

here: Q = amount of pumping water (m^3/sec)
b = depth of aquifer (m)
k = coef. of permeability (m/sec)
H = height of water pressure line (m)
h = height of water pressure line after pumping water (m)
D = influence radius (m)
r = radius of well (m)

The value of these data can be obtained from the Department of Mineral Resources, as following Fig - 3.2.

If the data mentioned in Fig 3.2 is applied into the equation, like:
Q = 210.05 gpm ≑ 0.795 m^3/min = 0.0132 m^3/sec
b = 25 m
k = assuming 1.5×10^{-4} m/sec as fine sand
H = 123 m
h = 120 m
r = 0.125 m

Fig. 3.2 DRILL LOG OF WELL NUMBER H1 18PN9
PATTANI WATER WORKS



DETAILS OF THE WELL

Drilled by Department of mineral Resources "H" Rig. Driller SANTI Drilling commenced July 6, '68. Drilling completed July 12, '68. Perforated screen 340 to 350' and Screen # 350' to 370'. Static water level 3.62' below land. Pumping level 12.43' below land surface. Yield; 210.05 gpm.

Table 3.1 Water Quality of Deep Well

Run No.	1	2	3	4	Remark
Appearance	Clear	-	Clear	Clear	
True Color	none	none	-	none	
Odour	unobject	unobject	-	unobject	
Temperature	30.2	-	-	29.5	°C
Turbidity	1.0	7.0	4.0	4.0	
pH	5.8	6.4	6.7	5.7	
Total Hardness	20.0	94.0	56.0	156.0	as Ca CO_3
M-alkalinity	22.0	27.0	46.0	37.0	"
Chloride	6.5	8.0	22.5	114.0	
Iron					
Total	1.0	1.1	3.5	6.0	
Dissolved	1.0	-	0.7	-	
Sulfate	nil	nil	nil	nil	
Magnesium	4.8	4.8	6.7	19.2	

Note: 1. Well 114 m deep at Water Treatment Plant
2. " " " "
3. Well 65 m deep at Songkhla University
4. Well 63 m deep at a slaughter-house

Table 3.2 Maximum Permissible Amount, Drinking Water Standard, WHO

Item	WHO Standard
pH	6.5 - 9.2
Total Hardness	100 - 500 ppm as Ca Co_3
Chloride	200 ppm
Ammonia Nitrogen	0.5 ppm
Nitrate Nitrogen	40 ppm
Iron	0.3 ppm
Fluoride	1.0 ppm
Manganese	0.1 ppm
Phenol	0.001 ppm
Magnesium	50 ppm
Calcium	75 ppm

Then,

$$\log \frac{D}{0.125} = \frac{2.73 \times 25 \times 1.5 \times 10^{-4} \times (123 - 120)}{1.32 \times 10^{-2}} = 2.33$$

Therefore, D = 215 m

In other word, if each well would be located in about 200 metres apart each other, normal amount of water can be expected from underground. However, according to present location of each wells mentioned in Fig 3.1, q_4 and q_5 can not be operated at same time because of nearby location at same aquifer. From this point of view, it is suggested that existing capacity is called as q_2 = 40 m^3/hr, q_4 or q_5 = 80 m^3/hr and total amount = 120 m^3/hr.

3.1.3 Water Quality

a) Raw Water Quality

Table - 3.1 shows typical water quality test data of deep well from the test of PMWD laboratory.

In general, routine test of raw water quality is done on base of about once a year by PMWD, but besides of this test, there are no data about daily test checked by the municipality, because the water treatment plant has no laboratory. The time being, the raw water has been checked occasionally when the expert and other personal may visit to the municipality and bring a sample back to PMWD.

Especially note worthy of the raw water quality listed in Table - 3.1 is that each water source quoted has high values than normal in iron and turbidity. Other items are found nothing to justify the existence of significant contamination. This is because the contents of nitrogenous compounds, chloride and heavy metals, except the iron and turbidity, are lower than the WHO standard quality for drinking water. The WHO standard is attached as Table 3.2.

b) Treated Water in Water Treatment Plant

Records of analysis of water quality after treatment by means of chemical coagulation, sedimentation and filtration has not been obtained. However, judging from the data of Table 3.1, there is no doubt about whole items of component besides of iron and turbidity. In other word, it is only needed to check iron content in treated water, because the turbidity is considered as result of iron contanination.

So far as the iron contents are concerned, the data has been given from many samples which was taken by the Expert of PMWD as shown in Table 3.3.

Table 3.3 Iron Contents of Treated Water

Item	Iron Content
Run 1	0.08
" 2	0.10
" 3	0.10

It is clearly said that the treated water is not always suitable for drinking purpose, because of iron contents given in Table 3.3. Further, in addition to the problem of iron in treated water, there are serious status about damages done by the excess iron. The damages are concerned mainly with corrosion throughout existing sedimentation and filtration. Table 3.4 shows the detailed analysis of samples taken from some sediments on the surface of hand - rail (A) and rapid sand filter (B) in the existing water treatment plant.

Table 3.4 Percent of Component

Sample	$Si\ O_2$	Fl_2O_3	Ignition Loss
A	93.8	3.4	1.7
B	87.3	7.5	0.3

The reason why silicate component found a great deal is caused by being mixed with filter sand in occasion of sampling.

In addition to the chemical component, it has been found that there are many Gallionella or equivalent bacteria and common algae as result of biological analysis in Japan. The Gallionella is one of iron bacteria which corrode very much iron pipes and other equipment made by iron. Therefore, it is absolutely necessary to remove from raw water before being flowed into sedimentation basin, because iron bacteria are always grown with serious damages.

Following are general impressions on the quality of treated water at present operation of existing plant.

1) The influent of sedimentation basin, as raw water, looks very clean than effluent, since the iron is still in dissolved type under condition of plenty carbon dioxide in raw water.

2) As soon as the raw water is exposed into air, the carbon dioxide tends to be exhausted, and dissolved iron comes to ferrous iron which can be easily oxidised to ferric iron. The ferric iron shows brown sediments.

3) Following this reaction, alum put already into raw water is reacted with an alkalinity, and makes floc composed by aluminium hydroxide. The reason why the water

begins to show slightly dull in middle part of sedimentation basin is proven by this phenomenon.

4) The pH value ranges always lower than 7.0 in raw water. This is caused by the carbon dioxide above mentioned, but it is believed that the pH value will rise when the raw water has been exposed into air and the carbon dioxide has been exhausted.

5) Alkalinity in raw water is also lower than normal range, so that coagulation made by alum has not shown in a considerably reasonable condition.

c) Tap Water in Service Area

The residual chlorine is not always recorded throughout the whole service area. The chlorine may be consumed while under running through distribution system. For this reason, it may be necessary to increase the amount of dosage of chlorine until the residual chlorine may be able to maintain at 0.2 ppm or more. If possible, it is recommended to change the use of chlorine from bleaching powder to liquid chlorine.

No special discussion will be needed for other item.

3.1.4 Water Treatment Plant

a) General

The plant has a normal capacity of 80 m^3/day, at this moment, with treatment facilities for chemical coagulation, sedimentation and rapid sand filtration.

According to the initial plan of this plant, raw water taken from three deep wells was to be treated with old plant of 20 m^3/day. However, it has no more been operated since new plant has finished. The old plant was almost completely damaged by heavy iron contents in old wells, so that it shall be destroyed in order to open a space for following ewtention project. Therefore, the discussion shall be limited within the new plant.

b) Chemical Coagulation and Flocculation

For coagulation, alum is now dosed in the inlet pipe of sedimentation tank, and it is mixed with turbulent flow through the inlet pipe. Then, flocs are formed through flocculation basin designed by up and down system for about 6 minutes against the flow of 80 m^3/hr.

As fundamental discussion, the present design of chemical coagulation and flocculation are not reasonably matched with a theory and practice. For example, coagulation and flocculation are not sufficient, because rapid mixing period of 1 to 5 minutes shall be provided and slow mixing period shall be 20 to 40 minutes of the plant capacity in general speaking.

Other disadvantage of existing facility is belonged to theoretical misunderstanding about alum coagulation given by following equation.

$$Al_2(so_4)_3\ 18H_2O + 3Ca(HCO_3)_2 = 2Al(OH)_3 + 3CaSO_4 + 6CO_2 + 18H_2O$$

The second item of left side of equation, as alkalinity of water, is absolutely necessary to make flocs of aluminium dydroxide which can be easily settled on sedimentation basin. However, as shown in Table 3.1, this underground water has not enough alkalinity, so that if alum is dosed directly into water, it may be rather difficult to form the flocs. It is clearly observed that alum coagulation is now reacted in present sedimentation but not in flocculation basin. This is caused by high amount of carbon dioxide and low amount of alkalinity. In consequence of disadvantages of present facility, it is recommended to change dosing point of alum and to install new dosing of lime and chlorine prior to alum dosing.

c) Sedimentation

After the flocculation, the water flows into a sedimentation basin which has no auxiliary one. The type of sedimentation basin is rectangular in shape as it is conventional throughout country, but it has no regulating chamber of flow such as perforated baffle walls.

Total capacity of sedimentation basin is about 300 m^3 which is capable to keep the water for about 4 hours. This detention period is reasonable, and average flow is as about 13 cm/min which is little slow than normal value like 30-40 cm/min. It is observed that many iron will be accumulated on bottom of the basin as sludge which can be blown off through existing drain pipe and value. These equipment is large enough to drain accumulated sludge in reasonable time, but it is unfortunate that there is not suitable out fall and drain pit to keep the sludge. At present, the sludge is blown off to a small pond to wait until being penetrated through the earth.

d) Filtration and Clear Water Reserveir

Over flow water in the end of sedimentation basin flows into the flume after passing over the weir, and then is introduced to the rapid sand filter. Water is filtered by the gravity system. The filter consists of two basins in which each basin has about 110 m/ day of rate of filtration. Water is only used for backwashing of filter bed and no air and no surface washing facilities are provided. Filtered water is stored in the clear water reservoir after being chlorinated by bleaching powder solution.

It is observed that there are plenty amount of filter film consisted of heavy iron on the surface of filter. This is proved as an iron bacteria which growth rapidly day after day on the surface of sand, and it penetrates into deep which causes heavy contamination of sand. Mud-balls have been formed on the surface of sand, so that it shall be needed to fluidize the sand as far as possible during back-washing.

It is apparently felt that the heavy smell of iron sulfate comes from area around the sand. This can be illustrated by the fact that there is still many iron in the effluent of sedimentation basin. It is also proved in the clear water reservoir which has many debris consisted by iron sulfate on the bottom of reservoir.

The waste water came from rapid sand filter is now flowing into the pond as well as the sludge from sedimentation basin. It is recommended that the waste water pit shall be installed to hold the waste water for at least one complete filter wash operation.

Since the operating efficiency of the plant is not covered by whole capacity, it may be possible to extend the running time of filter slightly longer that in ordinary basin but it is desirable to improve the system of back-washing.

e) Others,

Examination of water quality is not tested in the plant, because there is no laboratory. Although the examination does not cover full details with the result, some of important items of water analysis shall be provided even though it is simple, so that safer and more reliable water will be produced at the plant.

f) Summary

In summerising, present facilities in new treatment plant are as follows.

Chemical Feeding Equipment	
Alum solution tank	1 unit
Alum feeding pump	1 set
Flocculation Basin	1 unit
Capacity	8 m3
Dimension 0.7m x 1.5m x 7.5m	
Retension time	6 min.
Sedimentation Basin	1 unit
Capacity	2,940 m^3
Dimension 3.5m x 3.0m x 28m	
Retension time	3.75 hrs
Filtration Basin	2 Units
Capacity	80 m^3/hr.
Dimension 3.5m x 3.5m	
Rate of filtration	110 m/day
Clear Water Reservoir	1 Unit
Capacity	1,000 m^3
Dimension 20m x 15m x 3.3m	
Retention time	12.5 hrs.

3.1.5 Distribution System

a) General

The pipeline is laid under almost in public roads. The maintenance and management of the pipeline is generally rather difficult, because there is soft foundation under the surface of earth, and there are many river or klong crossings.

The pipeline was made up to progress of the inhabitants, so that there is not definite rule of piping in general. It is thought that the size of pipeline is not always enough for the inhabitants living near the pipeline.

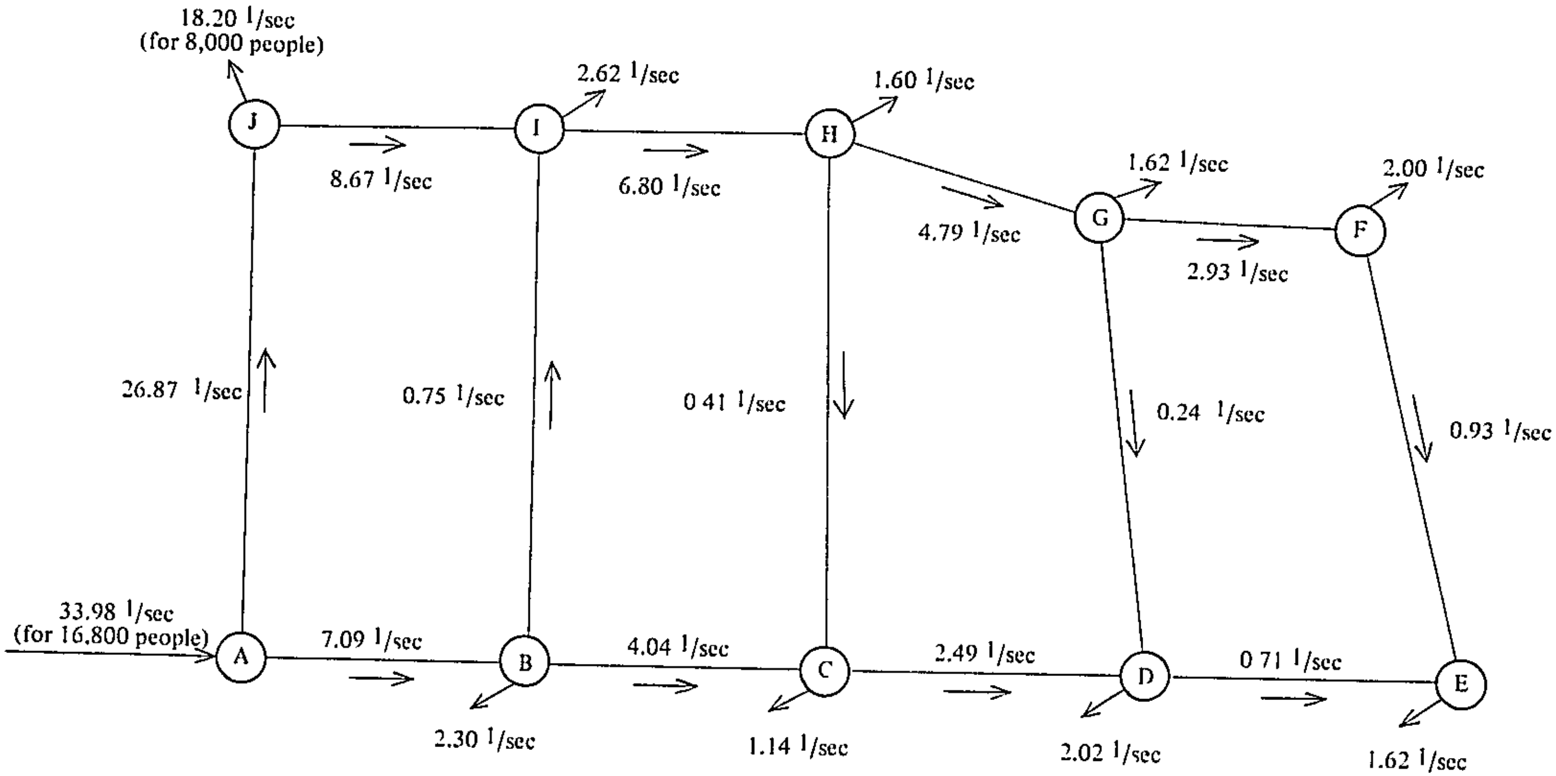
There are some pipelines laid to a greater depth than necessary as the result of improper planning and/or installation works. These may be brought about trouble or difficulties in maintenance and control of pipes and sometimes caused water leakage.

b) Networks

The networks of distribution pipes are rather complicated, because the size of pipe has not always been installed to satisfy a consumption of water by inhabitants. Particularly, there is not enough size of pipe around most densed area in distribution system. The road is almost paved by concrete, so it is difficult to change the pipe if it is not enough. This means that the water shall be covered by networks in order to keep the water pressure and amount of water.

For the purpose of this check, Fig-3.3 has been made in addition to Table 3.5, assuming total population as 16,800, by the Hardy-Cross method. Consequently, it may be said that outside loop is enough to cover the consumption, but branch pipes from the loop are not satisfactory to supply enough water to inhabitants.

Fig. 3.3 Hardy-Cross Calculation



Teble 3.5
c = 100

No.	D(MM)	L(m)	Q(1/sec)	I(0/00)	H(m)	h/Q	0.54.Eh / E h/Q	4Q(1/sec)	Q(1/sec)
AB	150	200	7.02	2.3	0.46	0.07	0.54x(-0.04)	+0.07	7.09
BI	100	500	0.76	0.3	0.15	0.20	0.31	0.07-0.08	0.75
IJ	200	260	-8.74	0.8	-0.21	0.02	= -0.07	0.07	-8.67
JA	300	490	-26.96	0.9	-0.44	0.02		0.07	-26.87
					-0.04	0.31			
BC	150	170	3.96	0.8	0.14	0.04	0.54x(-0.06)	0.08	4.04
CH	100	465	0.28	0.1	0.05	0.18	0.43	0.08+0.05	0.41
HI	200	160	- 6.88	0.6	-0.10	0.01	= -0.08	0.08	- 6.80
IB	100	500	- 0.76	0.3	-0.15	0.20		0.08-0.07	- 0.75
					-0.06	0.43			
CD	150	350	2.54	0.4	0.14	0.06	0.54x0.03	-0.05	2.49
DG	100	400	- 0.44	0.1	-0.04	0.09	0.34	-0.05+0.25	- 0.24
GH	200	130	- 4.74	0.3	-0.04	0.01	= 0.05	-0.05	- 4.79
HC	100	465	- 0.28	0.1	-0.05	0.18		-0.05-0.08	- 0.41
					0.03	0.34			
DE	150	160	0.96	0.1	0.02	0.02	0.54x0.05	-0.25	0.71
EF	200	370	- 0.68	—	—	—	0.11	-0.25	- 0.93
EG	200	140	- 2.68	0.1	-0.01	0.00	= 0.25	-0.25	- 2.93
GD	100	400	0.44	0.1	0.04	0.09		-0.25+0.05	0.24
					0.05	0.11			

The pressure throughout distribution system is limited depending upon the height of elevated tank, because there is no pumping system besides of the gravity flow from elevated tank. It may be good enough, at this moment to supply usually and normally the water to each house, but increasing the water pressure will provide much smooth water supply, prevents discharging for extinguishing fires and offer other various advantages to essential requirements for water supply. Therefore, efforts must be used to examine the whole aspects of water sources and distribution system for increasing water supply pressure.

c) House Connection

House connection is to supply water directly from the distribution pipe to each house. This is usual throughout Thailand that water branches from the distribution pipe is metered by a water meter.

There is no regulation of house connection in Thailand, in which it is to supply water to each house after it is stored in ground water tanks and then pumped up to either overhead tank or house tap. This system may be useful for more or less easing the current water shortage due to the low water pressure in the distribution pipe. However, contamination of the underground tank from outside owing to its defective structure and/ or incomplete installation is observed and, as a result, the water quality of the tank is exposed to danger.

From this views, the direct water supply system can be made more efficiently and effectively.

3.2 Improvement of Existing Water Supply System (Rehabilitation Work)

3.2.1 General Rule

Existing system should be examined to see if, it can provide the water supply. If existing water facilities can be put to see with less expenditure of time, effort, and equipment than it would take to develop a new facility.

The water source of an existing facility is the part most likely to be used even if the entire facility is not suitable. Unsuitable treatment plant can often be operated by reducing the rate of treatment, or by omitting some part of process, wish the exception of chemical dosing, until repair or improvement are made. Sometimes, certain operations which previously had been done automatically will have to be performed manually, and other expedient measures will have to be devised. Substitutes should be brought into use whenever possible for unsuitable equipment that cannot be replaced, or for which replacements would be a long time in arriving.

The most difficult part of a waterworks is its distribution system. For example, the leakage not only introduce the danger of contamination, but also allow loss of large quantities of water. Unfortunately, the distribution system is not always the part that can most easily be dispensed with, or substituted for, in taking over a municipal facility.

3.2.2 Water Source

Judging from the heavy irons contained in existing deep wells, it is absolutely necessary to remove irons prior to be treated in rapid sand filter, if otherwise many damages would be expected in corrosion of pipes, fittings and valves.

It looks like very delicate expression that 114 metres deep well has no irons comparing with heavy irons from 92 metres deep wells. However, if it is proved by the fact, it can be said that shallow layer may have heavy iron water caused by iron ores distributed widely around here, but there is almost no particular water contained heavy irons. From this point of view, it is recommended that new deep well shall be dug as deeper as possible more that about 120 - 150 metres in future.

3.2.3 Water Treatment

a) Jar Test

According to field study by Jar-Tester, present alum feeding is not enough to remove heavy iron. It is absolutely necessary to dose lime prior to the alum dosing for the purpose of increasing alkalinity. There are several alternate methods in practice as shown hereinafter.

Table 3.6 shows the field study of comparison test between several methods by means of Jar-Tester. There is almost no iron content after being filtered by filter paper.

Table 3.6 Comparison Test between Several Methods

Run	Lime	Aeration	Alum	Chlorine	Coagulation & Flocculation	Sedimentation	Iron Content	Remarks
A	●	15 min. ●	●		●	●	0.05	good
B	●	20 min. ●	●		●	●	trace	good
C	●		●	2 ppm ●	●	●	0.02	good
D	●		●	5 ppm ●	●	●	0.10	good
E	●		●		●	●	0.25	good
F		●				●	0.80	no good
G				5 ppm ●	●	●	0.40	no good
H				2 ppm ●	●	●	0.90	no good
I			●		●	●	0.10*	no good

*: with filtration

In general, the results from A to E looks good in spite of no good results through F to I. This means that in any case the lime and alum dosing are always necessary in addition to supplementary dosing of chlorine as prechlorination or aeration as pre-aeration. Aeration is effective in range of pH more than 9.0, so the lime feeding shall be always done in ahead of other process. The effect of aeration depends upon the time of aeration, but 15 minutes is good enough to oxidize dissolved iron. Chlorination is same condition as aeration, in which 2 ppm is good enough rather than dosing of 5 ppm.

The amount of lime is added until pH value comes to about 9.0, which may be evaluated as 10 ppm in practice. Floc forming is best in 20 ppm, depending upon comparative test. Jar-test has been conducted under 5 minutes for rapid mixing, 10 minutes for slow mixing and 20 minutes for sedimentation.

b) Alternative idea

Regarding to rehabilitation work of existing facility, there are two alternative idea, such as (1) temporary improvement to be used for existing underground water, and (2) permanent improvement to be used for new surface water.

The temporary improvement would cover only chemical feeding and rapid mixing of chemical coagulation, because there is only one problem about iron removal in existing underground water. Although present flocculation and sedimentation are not perfect design, it may be able to be utilized as well as present status.

On the other hand, permanent improvement would include repairs of whole facility such as chemical dosing, coagulation, flocculation, sedimentation and filtration, because surface water would not be able to be treated by existing facility.

Furthermore, the improvement of existing facility shall be related with a possibility of new water source whether surface water would be fully used in future or not. That is to say, there are only two alternative idea about extension project as mentioned in

Table 3.7.

Table 3.7 Alternative Extension Project

Alternative	Water Source	Water Treatment	
		Improvement of Existing	New Installation
	New Wells	—	Iron Removal Only
	Existing Well ($80m^3$/hr.)	Temporary Improvement	—
	Existing Well* ($40m^3$/hr.)	—	—
	Existing Well ($80m^3$/hr.)	Permanent Improvement	—
	New Surface Water	—	Complete Treatment

*: to be common

c) Temporary improvement

Fig 3.4 shows the flow sheet of the temporary improvement about coagulant feeding and rapid mixing. Necessary specification of the temporary improvement is as followings.

Lime Feeding : Chemical Mixer & Feeder ....1 set
10 ppm of lime in order to raise pH to about 9, and increase alkalinity to help alum coagulation;
10 ppm x $80m^3$/hr x 24 hrs x 10^{-6} x 1,000 = 19.2 kg/d

Coagulation : Fisher & Porter Chlorinator..............1 set
2 ppm of liquid chlorine in order to oxidise, and remove heavy iron as pre-chlorination;
2 ppm x $80m^3$/hr x 24 hrs x 10^{-6} x 1,000 = 3.84 kg/d

Chlorine Container: 50 kg ....5 cylinder

Coagulation : Volti-Mixer Agitator, $120m^3$/hr ....1 set
Tank made by steel plate, 1.3m x 1.3m x 1.3m

Alum Feeding : Use existing equipment, but change the feeding point to other place after adding lime and chlorine.
20 ppm x $80m^3$/hr x 24 hrs x 10^{-6} x 1,000 = 38.4 kg/d

d) Permanent Improvement

When existing water treatment plant would be used for new surface water source, overall facility shall be checked in order to be coincided with standardized criteria. However, the improvement shall be based on whole layout about total capacity of surface water. It is suggested in here to see 4.4: Ist Stage Works.

3.2.4 Distribution System

As the most urgent program of rehabilitation works, it is recommended to found water leakage through distribution system. However, it is not so simple that rehabilitation works shall be done at same time with new installation of pipe.

Water leakage from these distribution pipes is mainly caused by incomplete backfill tamping when they were laid, resulting in irregular settlement and loose joints. Therefore, it is necessary to tighten again the joints. Any values responsible for the greater part of visible leakage detected should be repaired.

Early repair of water mains is desirable and vital. It is a difficult and hazardous job, which will be performed by an engineer unit. Emergency action will consist of isolating the section by closing a minimum number of values. This action serves to conserve the supply, and to prevent loss of pressure, flooding and contamination.

The required repair operations can be planned, scheduled, and undertaken at a later date. Meanwhile, temporary repairs are sometimes called for to control water wastage and maintain the necessary pressure for fire protection.

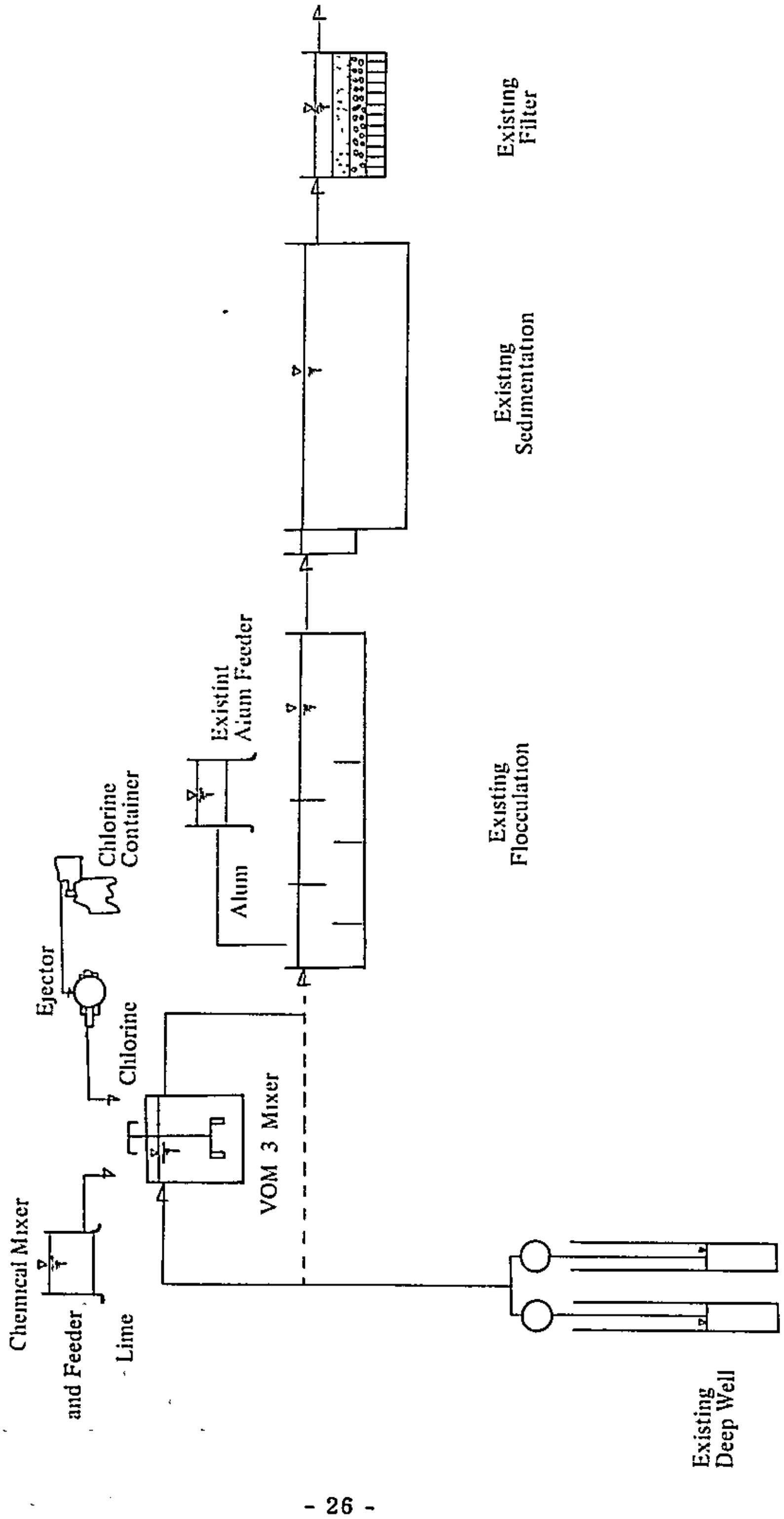


Fig. 3.4 Flow Sheet of Rehabilitation of Existing Water Treatment

Chapter 4 The Basic Proposal

4.1 Population Projection

4.1.1 Vital Statistics

According to the statistics of Statistical Sub-Section, Provincial Water Supply Division, PMWD, Table 4.1 shows the past records of population and consumer.

Regarding to the increase of population, there is comprehensive rule throughout the country such as "rate of population growth of Thailand between 1968 to 1988 is estimated 3.2% per year." If this rule will be applied to the duration from 1958 to 1968 in Table 4.1, the population in 1968 must be calculated as follows.

$$P = 15,683 \times 1.032^{10} = 15,683 \times 1.374 = 21,548$$

Thus, the actual population in 1968 is higher than above figure, because there were irregular increase of population since 1964. This phenomena can be acknowledged by the fact of group movement of people from border to this municipality.

4.1.2 Past Population Forecast

More difficult is the estimation of population in some future year. Several methods are used, but it should be pointed out that judgement must be excercised by the engineers as to which method is most applicable. There have been an accurate forecast of population as shown as followings.

a) First Japanese Colombo Plan Expert

First Japanese Colombo Plan Expert studied about population increase in future, based on 4% per year for municipality and 3% per year for suburbs under the geometric increase. It is listed in Table 4.2.

Table 4.2 Future Population

Year	Municipality	Suburbs	Total
1970	25,000	10,400	35,400
1975	31,000	12,000	43,000
1980	38,000	14,000	52,000
1985	46,000	16,400	62,400
1990	56,000	18,800	74,800
1995	68,000	21,800	89,800
2000	82,000	25,200	107,200

Table 4.1 Past Records of Population and Consummer

Pattani Water Works
Starting Construction Expenditure 1,510,000 Baht
Started on 20 October 1955

Source of Water Deep Well

Capacity per day $80m^3/hr$

Year	Population (person)	Increased+ Decreased-	Area (km^2)	Density of popn/km^2	House hold	Average Person/ House	Serving area km2	Consumer			
								Consumer (meter)	Consumer (meter) Household %	Consumer (Head)	% of Consumer
(1)	(2)	(3)	(4)	(5)	(6)	(7)= (2)/(6)	(8)	(9)	(10)= (9)/(6)x100	(11)= (7)x(9)	(12)=(11)/(12)x100
1958	15,683		4,87		2,520	6.2		445	17.66	2,759	17.59
1959	16,202	+ 519	"		2,600	6.2		482	18.54	2,988	18.44
1960	16,970	+ 768	"	3,485	2,691	6.3		557	20.70	3,509	20.68
1961	17,483	+ 513	"	3,592	2,795	6.3		628	22.29	3,925	22.45
1962	18,007	+ 524	"	3,701	2,805	6.4		710	25.31	4,544	25.53
1963	18,504	+ 497	"	3,803	2,852	6.5		776	24.65	4,569	24.69
1964	19,902	+1,398	"	4,090	2,921	6.8		842	27.25	5,413	27.19
1965	20,535	+1,333	"	4,220	2,986	6.9		908	26.66	5,492	26.74
1966	21,459	+ 924	"	4,406	3,050	7.4		974	28.59	6,453	28.55
1967	22,424	+ 965	"	4,604	3,152	7.2		1,040	32.99	7,488	31.54
1968	23,741	+1,317	"		3,145	7.5		1,200	38.15	9,000	37.90
1969	24,170	+ 429	"		3,214	7.5		1,239	38.55	9,293	38.45

b) Increase Curve

According to Table 4.1, past increase of population can be shown by a curve shown in Fig 4.1. Duration between 1958 and 1963, population curve shows apparently arithmetic increase, but since then as stated before, population increase tended to much complex.

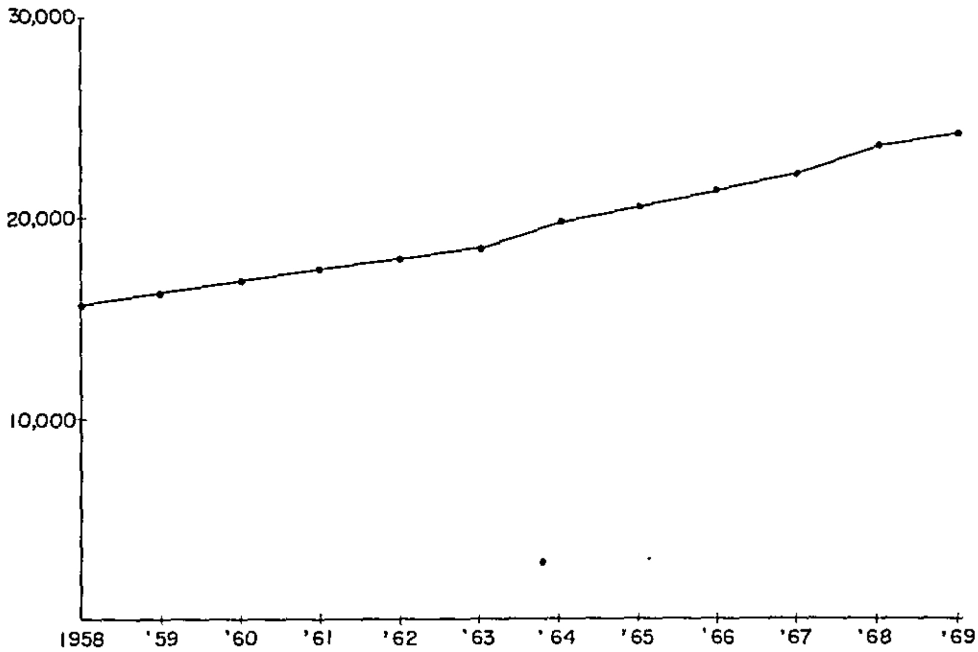
4.1.3 Future Population

a) General Discussion

There are several methods of determination of planned population to be supplied such as arithmetical increase, geometric increase, logistic curve's increase and power function's increase. However, either one has only the meaning of mathermatical consolidation, because vital statistics are not so simple knowledge which can be easily calculated on the mathematical equation. Population increase must be renewed by industrial growth, human revolution, industrilization, modernization of life, urbanization and so forth.

The developing country always uses the way of five year program which has good sounds of emergency planning. However, it shall always be renewed again after two to three years. This is proved what how the future estimation is difficult or how so complex the fact is.

Figure 4.1. Past Population Increase



There will never be an accurate forcast of population over the next thirty years. This is caused by the reason that economic pressure, accelerated Government transfers, the attraction to the capital tend to increase the population during the early years of the city's growth. A knowledge of the municipality and its surburbs, whether or not its industries are expanding, the state of development will enter into the estimation of future population.

b) Determination of Future Population

The ultimate figures arrived by this planning are to be said as tentative and will have to be further refined by undertaking a more detailed study in future and a continuous watch over the growth of population will have to be kept so as to enable precise and accurate forecast for the future.

Based on the fundamental consideration of population estimation above mentioned, future population is estimated by various methods as stated before, and the summarization in general is as Table 4.3.

Table 4.3 Projected Population

Year	Geometric*	Power function**	Arithmetic***
1970	25,000	24,900	24,896
1980	38,000	33,744	33,146
1990	56,000	43,514	41,396
2000	82,000	53,771	49,646

* from Table 4.2

** as stated latter

*** Arithmetic increase (Constant percentage rate of growth)

Equation	: Y = 825X + 19,946
Population in 1964	: Po = 0
Rate of Increase	: from past curve
Population in 1980	: Pn = 825 x 16 + 19,946 = 33,146

It can be said that the geometric calculation is unreasonably too high because this method is most applicable to industrial and aggresive municipality with a great deal of manufacturers. Then, the power function's calculation is reliable method for the newly modernised municipality, comparing with the arithmetic increase which is applicable to old and small municipality without particular industries.

Regarding to the percentage rate of growth, it may be most simple to adopt 3.2% which has been authorized in statistical authority in Thailand. However, on the other hand, constant percentage may also apply to old municipality not undergoing great expansion, but this percentage may come to rather small. It should be recognized also that, as municipality grow large, there is a tendency to decrease in the rate of growth.

Even though it is a sort of mathematical consolidation, consequently, it is recommended to adopt the power function calculation method, because this is most applicable to any municipality which has shown complex increase in the past.

Consequently, the culculation of power function's estimation is stated hereinunder which shows the detailed method of calculation of population listed in Table 4.4.

Year	n	Pn
1970	12	24,900
1980	22	33,744
1990	32	43,514
2000	42	53,771

Table 4.4 Future Population Estimation

Year	Population	n	X = log	X^2	Pn	Pn-Po	Y = log(Pn-Po)	XY
1958	15,683	0	—	—	64.89	—	—	—
1959	16,202	1	0.00000	0.00000	67.04	2.15	0.33244	0
1960	16,970	2	0.30103	0.09062	70.22	5.33	0.72673	0.21877
1961	17,483	3	0.47712	0.22764	72.34	7.45	0.87216	0.41612
1962	18,007	4	0.60206	0.36248	74.51	9.62	0.98318	0.59193
1963	18,504	5	0.69897	0.48856	76.56	11.67	1.06707	0.74585
1964	19,202	6	0.77815	0.60552	79.45	14.56	1.16316	0.90511
1965	20,535	7	0.84510	0.71419	84.97	20.08	1.30276	1.10096
1966	21,459	8	0.90309	0.81557	88.79	23.90	1.37840	1.24482
1967	22,424	9	0.95424	0.91051	92.78	27.89	1.44545	1.37931
1968	23,741	10	1.00000	1.00000	98.23	33.34	1.52297	1.52297
1969	24,170	11	1.04139	1.08449	100.00	35.11	1.54543	1.60940
Σ	—	—	7.60115	6.29964	—	—	12.33975	9.73524

$N = 11,\ \Sigma X = 7.60115,\ \Sigma X^2 = 6.29964,\ \Sigma Y = 12.33975,\ \Sigma XY = 9.73524$

$$a = \frac{N.\Sigma XY - \Sigma X.\Sigma Y}{N.\Sigma X^2 - \Sigma X.\Sigma X} = \frac{11 \times 9.73524 - 7.60115 \times 12.33975}{11 \times 6.29964 - (7.60115)^2} = \frac{13.29135}{11.51856} = 1.15391$$

$$b = \frac{\Sigma X^2.\Sigma Y - \Sigma X.\Sigma XY}{n.\Sigma X^2 - \Sigma X.\Sigma X} = \frac{6.29964 \times 12.33975 - 7.60115 \times 9.73524}{11 \times 6.29964 - (7.60115)^2} = \frac{3.73696}{11.51856} = 0.32443$$

$\log A = b$

$\therefore A = 2.1107$

$Pn = Po + A.n^a$

$= 64.89 + 2.1107 \times n^{1.15391}$

4.1.4 Percentage of House Connection

a) Water Supply Area

Percentage of house connection is same meaning as propagation ratio. In general, the water supply area shall be determined by the municipality itself within the administrative area. In other word, the water supply area is up to 100 percent of administrative area depending upon the local condition of housing location of the inhabitants. It is easy to determine the water supply area on the map of the administrative area, but it has no significant if the population in the water supply area cannot be definitely determined. If the vital statistics are completely arranged in some office, it can be done. However, it is almost impossible to determine peoples those who are living in the water supply area under present data concerned.

From these viewpoints, the percentage of house connection shall be determined with reference to the records of water supply of a municipality with similar characteristics and developing situation. And then, the increase of house connection after the completion of construction will depend on the needs of drinking water, progress of new cultural life, improvement of social life, condition of environmental sanitation and so on. According to several data in the past records, the first year after the completion of construction is determined as 60%. Further, 75% is determined as final goal of percentage of house connection, as shown in Table 4.5.

Table 4.5 Percentage of House Connection

Year	%
1973	60
1980	65
1990	70
2000	75

4.2 Water Demand

4.2.1 Average daily consumption

a) Histogram

Since 1959, the daily consumption of water per capita (Xi) has been reported for seventeen municipalities (n), which are listed on Table 4.6-4.14. Then, the arithmetic total mean (m) can be gained as $m = \frac{\Sigma Xi}{n}$

Table 4.6 (1958)

Name of Municipality	Average Consumption Per Capita per Day (X_1) (l/c/d)	$X_1 - m$	$(X_1 - m)^2$	Remarks
Krathumbaen	28	- 73.2	5,358.24	$n = 13$
Yala	38	- 63.2	3,994.24	$m = \frac{\Sigma X}{n}$
Saraburi	44	- 57.2	3,271.84	
Roi-Et	54	- 47.2	2,227.84	$= 101.2$
Pattani	74	- 27.2	749.84	$\sigma = \sqrt{\frac{\Sigma(X - m)^2}{n}}$
Photharam	91	- 10.2	104.04	
Uthai-Thani	92	- 9.2	84.64	$= 53.4$
Ratchaburi	115	+ 13.8	190.44	
Chiang-Rai	121	+ 19.8	392.04	
Korat	123	+ 21.8	475.24	
Chiang-Mai	149	+ 47.8	2,284.84	
Nakorn-Sawan	170	+ 68.8	4,733.44	
Phatthalung	216	+114.8	13,179.04	
Σ	1,315	—	37,045.72	

Table 4.7 (1959)

Name of Municipality	Average Consumption Per Capita per Day (X_1)	$X_1 - m$	$(X_1 - m)^2$	Remarks
	(1/c/d)			
Krathumbaen	30	-29.6	876.16	$n = 9$
Roi-et	36	-23.6	556.96	
Phichit	39	-20.6	424.36	$m = \frac{\Sigma X_1}{9}$
Saraburi	52	- 7.6	57.76	$= 59.6$
Chiang Rai	56	- 3.6	12.96	
Panat nikhom	60	+ 0.4	0.16	$\sigma = \sqrt{\frac{\Sigma(X_1 - m)^2}{n}}$
Phathalung	74	+14.4	207.36	
Ratchaburi	86	+26.4	696.96	$= \sqrt{\frac{4716.24}{9}}$
Photharam	103	+43.4	1,883.56	$= 22.9$
Σ	536	—	4,716.24	

Table 4.8 (1960)

Name of Municipality	Average Consumption Per Capita per Day (X_1)	$X_1 - m$	$(X_1 - m)^2$	Remarks
	(1/c/d)			
Krathumbaen	32	-42.7	1,823.29	$n = 15$
Yala	48	-26.7	712.89	
Uthai-Thani	49	-25.7	660.09	$m = \frac{\Sigma X_1}{n}$
Chiang-Rai	50	-24.7	610.09	$= 74.7$
Phathalung	59	-15.7	246.49	
Rathaburi	63	-11.7	136.89	$\sigma = \sqrt{\frac{\Sigma(X_1 - m)^2}{n}}$
Pattani	65	- 9.7	94.09	$= 29.5$
Panut-Nikhom	66	- 8.7	75.69	
Phichit	70	- 4.7	22.09	
Photharam	77	+ 2.3	5.29	
Roi-et	77	+ 2.3	5.29	
Saraburi	83	+ 8.3	68.89	
Chiang-Mai	114	+39.3	1,544.49	
Korat	125	+50.3	2,530.09	
Nakorn_Sawan	142	+67.3	4,529.29	
Σ	1,120	—	13,065.35	

(1961)

Table 4.9

Name of Municipality	Average Consumption Per Capita per Day (X_i) (1/c/d)	$X_i - m$	$(X_i - m)^2$	Remarks
Krathumbaen	27	-36.1	1,303.21	$n = 9$
Panat Nikhom	46	-17.1	292.41	$m = \frac{\Sigma X_i}{n}$
Chiang-Rai	48	-15.1	228.01	
Ratchaburi	58	- 5.1	26.01	$= 63.1$
Photharam	70	+ 6.9	47.61	
Phathalung	70	+ 6.9	47.61	$\sigma = \sqrt{\frac{\Sigma(X_i - m)^2}{n}}$
Phichit	77	+13.9	193.21	
Pattani	81	+17.9	320.41	$= \frac{3,236.89}{9}$
Saraburi	91	+27.9	778.41	$= 19\ 0$
Σ	586	—	3,236.89	

(1962)

Table 4.10

Name of Municipality	Average Consumption Per Capita per Day (X_i) (1/c/d)	$X_i - m$	$(X_i - m)^2$	Remarks
Krathumbaen	28	-63.8	4,070.44	$n = 16$
Panat-Nikhom	47	-44.8	2,077.04	$m = \frac{\Sigma X_i}{16}$
Chiang-Rai	48	-43.8	1,918.44	
Uthai-Thani	55	-36.8	1,354.24	$= 91.8$
Chiang-Mai	68	-23.8	566.44	
Yala	72	-19.8	392.04	$\sigma = \sqrt{\frac{\Sigma(X_i - m)^2}{n}}$
Saraburi	81	-10.8	116.64	$= 36.6$
Roi-et	93	+ 1.2	1.44	
Phathalung	93	+ 1.2	1.44	
Phichit	96	+ 4.2	17.64	
Nakorn-Sawan	107	+15.2	231.04	
Pattani	112	+20.2	408.04	
Ratcha-Buri	130	+38.2	1,459.24	
Hua-Hin	140	+48.2	2,323.24	
Photharam	148	+56.2	3,158.44	
Korat	150	+58.2	3,387.24	
Σ	1,468	—	21,413.04	

Table 4.11 (1963)

Name of Municipality	Average Consumption Per Capita per Day (X_1)	$X_1 - m$	$(X_1 - m)^2$	Remarks
	(1/c/d)			
Krathumbaen	28	- 32.4	1,049.76	$n = 11$
Panat-Nikhom	34	- 26.4	696.96	$m = \frac{\Sigma X_1}{n}$
Saraburi	53	- 7.4	54.76	$= 60.4$
Phichit	53	- 7.4	54.76	
Pattani	56	- 4.4	19.36	$\sigma = \sqrt{\frac{\Sigma(X_1 - m)^2}{n}}$
Ratchaburi	56	- 4.4	19.36	$= 18.9$
Samut-Sakhorn	61	+ 0.6	0.36	
Chiang-Rai	68	+ 7.6	57.76	
Photharam	76	+ 15.6	243.36	
Phathalung	86	+ 25.6	655.36	
Roi-Et	93	+ 32.6	1,062.76	
Σ	664	—	3,914.56	

Table 4.12 (1964)

Name of Municipality	Average Consumption Per Capita per Day (X_1)	$X_1 - m$	$(X_1 - m)^2$	Remarks
	(1/c/d)			
Krathumbaen	27	- 80.5	6,480.25	$n = 17$
Panat-Nikhom	46	- 61.5	3,782.25	$m = \frac{\Sigma X_1}{n}$
Yala	64	- 43.5	1,892.25	
Saraburi	66	- 41.5	1,722.25	$= 107.5$
Samut-Sakhorn	67	- 40.5	1,640.25	$\sigma = \sqrt{\frac{\Sigma(X_1 - m)^2}{n}}$
Chiang-Mai	75	- 32.5	1,056.25	
Photharam	79	- 28.5	812.25	$= 57.3$
Chiang-Rai	82	- 25.5	650.25	
Phathalung	88	- 19.5	380.25	
Roi-Et	104	- 3.5	12.25	
Phichit	106	- 1.5	2.25	
Uthai-Thani	122	+ 14.5	210.25	
Ratchaburi	137	+ 19.5	380.25	
Nakorn-Sawan	137	+ 19.5	380.25	
Korat	160	+ 52.5	2,756.25	
Pattani	203	+ 95.5	9,120.25	
Hua-Hin	264	+156.5	24.492.25	
Σ	1,827	—	55,770.25	

(1965)

Table 4. 13

Name of Municipality	Average Consumption Per Captia per Day (X_i)	$X_i - m$	$(X_i - m)^2$	Remarks
Krathumbaen	(1/c/d) 30	- 73.7	5,431.69	$n = 11$
Panut-Nikhom	61	- 42.7	1,823.29	$m = \frac{\Sigma X_i}{n}$
Saraburi	64	- 39.7	1,576.09	
Samut-Sakhorn	67	- 36.7	1,346.89	$= 103.7$
Phathalung	78	- 25.7	660.49	$\sigma = \sqrt{\frac{\Sigma(X_i - m)^2}{n}}$
Chiang-Rai	95	- 8.7	75.69	
Roi-Et	100	- 3.7	13.69	$= 53.0$
Phichit	104	+ 0.3	0.09	
Photharam	142	+ 38.3	1,466.89	
Pattani	200	+ 96.3	9,273.69	
Rachaburi	200	+ 96.3	9,273.69	
Σ	1,141	—	30,942.19	

(1966)

Table 4. 14

Name of Municipality	Average Consumption Per Capita per Day (X_i)	$X_i - m$	$(X_i - m)^2$	Remarks
Krathumbaen	(1/c/d) 29	- 89.8	8,064.04	$n = 12$
Pattani	62	- 56.8	3,226.24	$m = \frac{\Sigma X_i}{n}$
Panat-Nikhom	67	- 51.8	2,683.24	
Samut-Sakhorn	69	- 49.8	2,480.04	$= 118.0$
Phichit	104	- 14.8	219.04	
Phathalung	108	- 10.8	116.64	$\sigma = \sqrt{\frac{\Sigma(X_i - m)^2}{n}}$
Chiang-Mai	113	- 5.8	33.64	
Roi-Et	114	- 4.8	23.04	$= 65.1$
Nakorn-Sawan	131	+ 12.2	148.84	
Yala	140	+ 21.2	449.44	
Ratchaburi	215	+ 96.2	9,254.44	
Photharam	274	+155.2	24,087.04	
Σ	1,426	—	50,785.64	

Getting deviation from the mean, the histogram can be figured as shown in Fig 4.2 and 4.3.

Fig 4-1

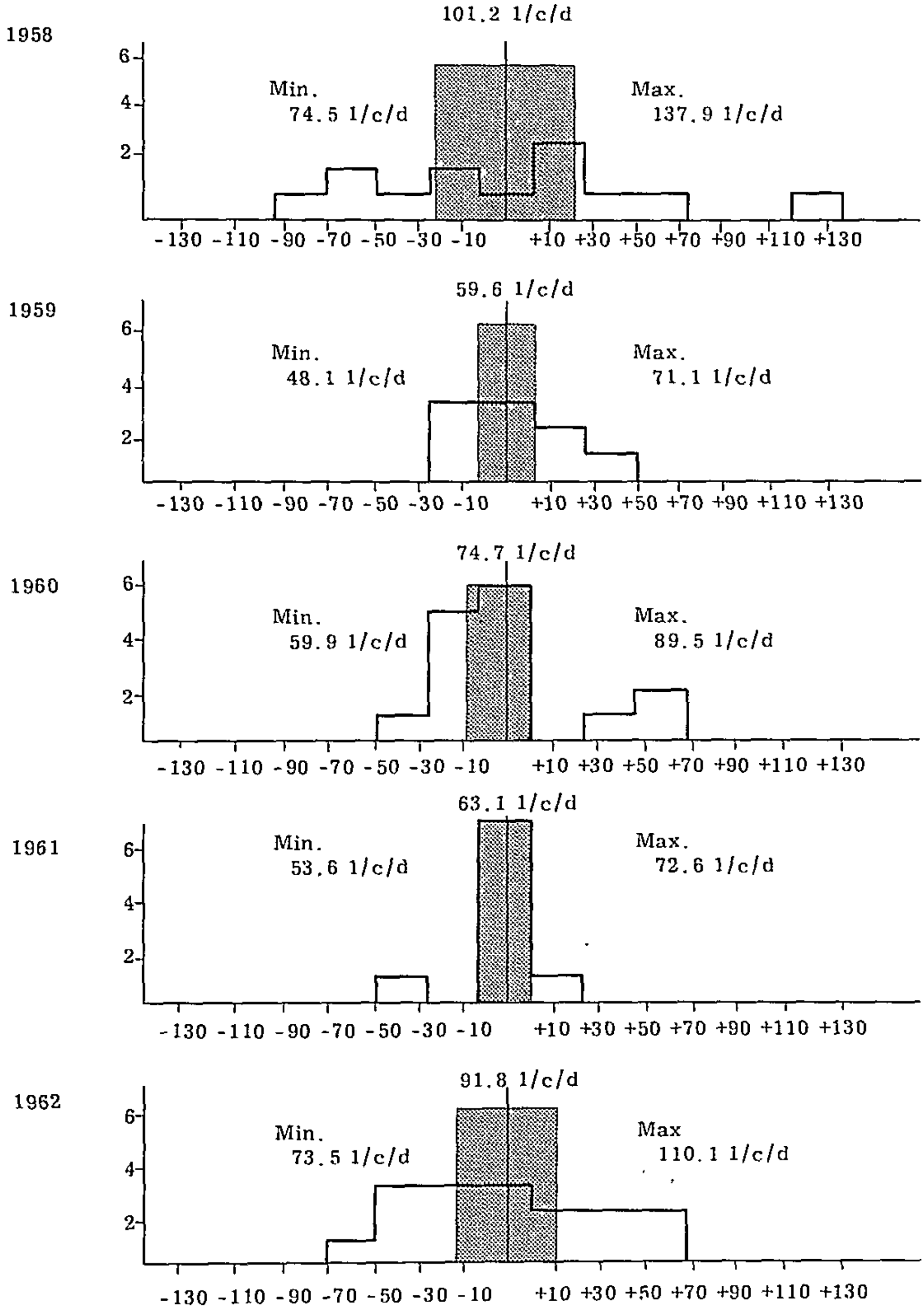
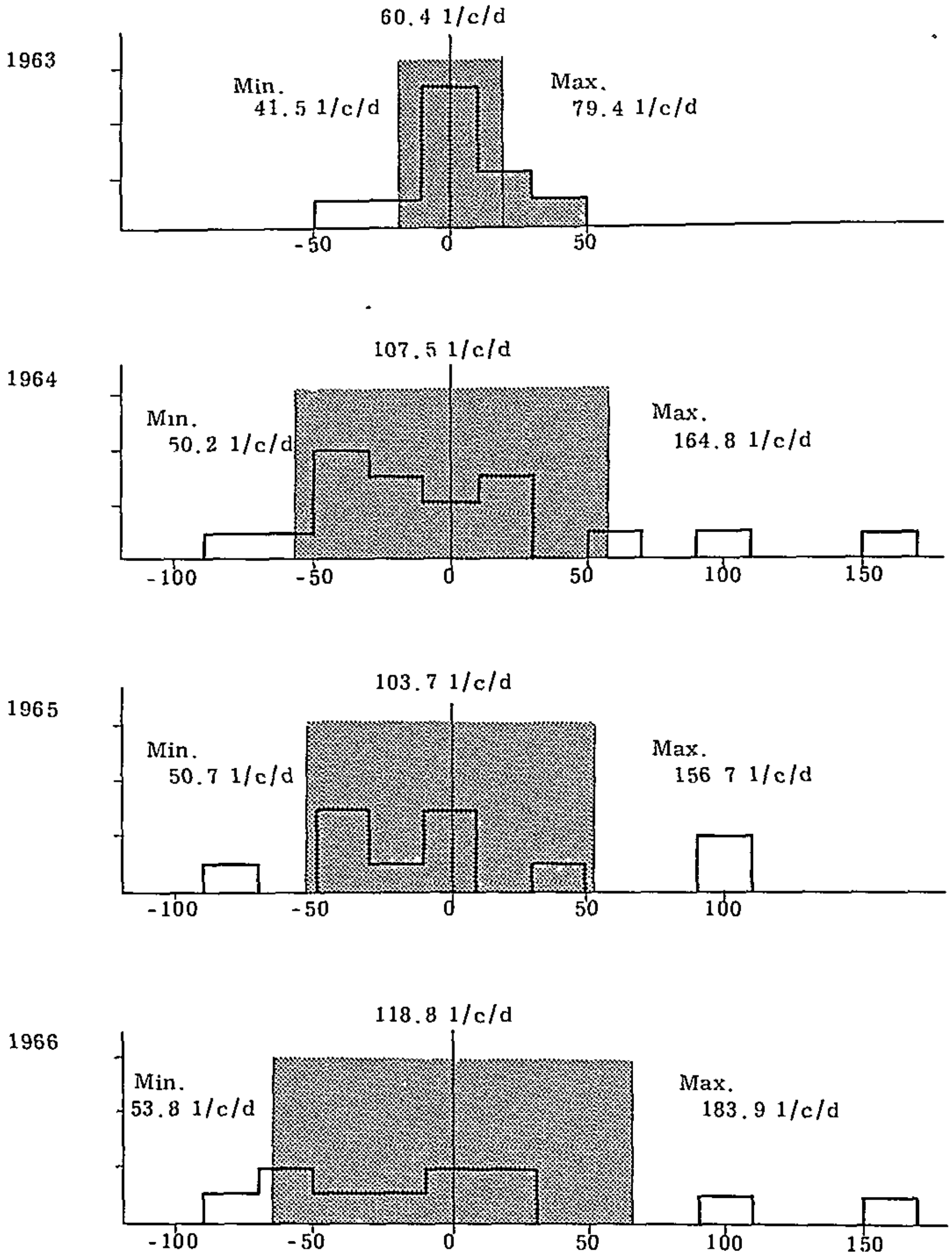

1958
101.2 1/c/d
Min.
74.5 1/c/d
Max.
137.9 1/c/d
1959
59.6 1/c/d
Min.
48.1 1/c/d
Max.
71.1 1/c/d
1960
74.7 1/c/d
Min.
59.9 1/c/d
Max.
89.5 1/c/d
1961
63.1 1/c/d
Min.
53.6 1/c/d
Max.
72.6 1/c/d
1962
91.8 1/c/d
Min.
73.5 1/c/d
Max
110.1 1/c/d
6
4
2
-130 -110 -90 -70 -50 -30 -10 +10 +30 +50 +70 +90 +110 +130

Fig 4-2


1963
60.4 1/c/d
Min.
41.5 1/c/d
Max.
79.4 1/c/d
-50
0
50
1964
107.5 1/c/d
Min.
50.2 1/c/d
Max.
164.8 1/c/d
-100
-50
0
50
100
150
1965
103.7 1/c/d
Min.
50.7 1/c/d
Max.
156 7 1/c/d
-100
-50
0
50
100
1966
118.8 1/c/d
Min.
53.8 1/c/d
Max.
183.9 1/c/d
-100
-50
0
50
100
150

b) Computation of standard deviation

By way of the square root of the mean square of deviation's square in each year, the standard deviation (σ) can be defined as $\sigma = \sqrt{\frac{\Sigma(m - X_i)^2}{n}}$. Therefore, statistically significant value can be shown as, $m - \sigma \leqq$ water consumption $\leqq m + \sigma$ as listed in Table 4.15.

Table 4.15

Year	average (m) (l/c/d)	σ (l/c/d)	$m - \sigma$ (l/c/d)	$m + \sigma$ (l/c/d)
1959	59.6	22.9	36.7	82.5
1960	74.7	29.5	44.2	104.2
1961	63.1	19.0	44.1	82.1
1962	91.8	36.6	55.2	128.4
1963	60.4	18.9	41.5	79.3
1964	107.5	57.3	50.2	164.8
1965	103.7	53.0	50.7	156.7
1966	118.8	65.1	53.7	183.9

c) Differences between municipalities

Table 4.15 shows that the standard deviation tends to increase year by year, which means that the differences of water consumption are increasing between municipalities consumed large amount of water and small amount of water. This means that the water consumption shall be depended on the scale of municipalities (particularly population), and shall not be one value throughout country.

d) Determination of daily consumption

As mentioned above, it is recommended that the past data shall be separated into three groups of which each group is classified on the population less than 50,000, between 50,000 and 100,000, and more than 100,000 respectively at AD 2000.

From this step, any data has been arranged in each group for each year respectively and a mean value has been calculated among the data taking off the eccentric maximum and minimum values. As the result, the average value can be used for further step as the typical data of each year.

e) Municipality populated less than 50,000

Regarding the municipalities populated less than 50,000 at AD 2000 as listed as Table 4.16, the mean value per each year is shown as Y, in addition to other value such as X, X^2 and XY.

Table 4.16 Daily Consumption, mean value per each year

Year	x	Y	$X = \sqrt{x}$	X^2	XY
1959	0	53	0.000	0	0
1960	1	62	1.000	1	62.00
1961	2	62	1.414	2	87.67
1962	3	72	1.732	3	124.70
1963	4	71	2.000	4	142.00
1964	5	84	2.236	5	187.82
1965	6	88	2.449	6	215.51
1966	7	98	2.645	7	259.21
1967	8	99	2.828	8	279.97
Total	-	689	16.304	36	1,358.88

Considering about paraholic curve if one of the best solution to determine the future estimation of the daily consumption, the least square method is applied for $Y = a\sqrt{x} + b$ as follows.

$$a = \frac{n\Sigma XY - \Sigma X.\Sigma Y}{n\Sigma X^2 - \Sigma X.\Sigma X} = \frac{9 \times 1358.88 - 16.304 \times 689}{9 \times 36 - (16.304)^2} = 17.1 \quad \text{say } 17$$

$$b = \frac{\Sigma X^2 \Sigma Y - \Sigma X \Sigma XY}{n\Sigma X^2 - \Sigma X.\Sigma X} = \frac{36 \times 689 - 16\,304 \times 1358.88}{9 \times 36 - (16.304)^2} = 45.5 \quad \text{say } 45$$

In consequence, equation $Y = 17\sqrt{x} + 45$ is determined as basic equation for future estimation of daily consumption. Then, Table 4.17 can be obtained.

Daily Consumption of Municipalities
Table 4.17 Populated less than 50,000 (l/cap.)

Year	Average, from equation	Average, determine	Maximun, Average x 1.5
1970	101	117	175
1980	123	127	190
1990	140	137	205
2000	155	147	220

f) Municipalities populated more than 50,000

Because of lack of data, it can not be satisfied to apply same idea into other municipalities populated more than 50,000. Then, according to experimental data, following equation is defined respectively.

Population between 50,000 and 100,000: $Y = 17\sqrt{x} + 65$ (Table 4.18)

Population more than 100,000: $Y = 17\sqrt{x} + 90$ (Table 4.19)

Daily Consumption of Municipalities
Table 4.18 Populated between 50,000 to 100,000 (l/cap.)

Year	Average from equation	Average, determine	Maximum, Average x 1.5
1970	121	127	190
1980	143	140	210
1990	159	153	230
2000	175	167	250

Daily Consumption of Municipalities
Table 4.19 Populated more than 100,000 (l/cap.)

Year	Average from equation	Average, determine	Maximum, Average x 1.5
1970	146	150	225
1980	168	167	250
1990	185	183	275
2000	200	200	300

4.2.2 Maximum daily consumption

a) General

In order to plan a satisfactory water supply system, it is imperative that we should know exact seasonal, daily and hourly variation of demand. However, since the condition of water supply is such that it has never met the full demand of the people, it is not possible to have the exact idea of such a variation specifically.

b) Other country

Examples of other country are helpful for estimation of variation in demand and given below in Table 4.18.

Table 4.18 Example of Ratio of Daily Maximum over Daily Average in other countries.

Name	Population(1,000)	Ratio	Year
London	6,132	1.28	1969
Stockholm	923	1.27	1968
Paris	2,760	1.17	1967
Zurich	435	1.54	1968
Rome	2,682	1.20	1968
Chicago	4,703	1.63	1968
Tokyo	8,246	1.25	1967
Average of1504 Municipalities in Japan	—	1.28	1967

c) Field study

To comply shortage of data, several types of field study has been performed throughout developing countries. Unfortunately, however, since the water supply has been limited and restricted due to shortage of water, it is not possible to ascertain the actual consumption per house or capita. This tendency is always emphasized in a municipality in which such study is needed.

In any how, the second Colombo Plan Experts have tried field study in Sri - racha. This municipality is now developing by commercial activities such as oil refinery and fishery, and it has great advantage in compact and independent shape of municipality as for as domestic water supply is concerned. Thable 4.19 shows the montly variation of water used, depending upon money collection of water rate.

According to Table 4.19, it can be said that the maximum consumptions are recorded on December or February under about 20% over the average consumption throughout year. This may show typical variation in especially the water being used for domestic purpose. Therefore, when a commercial use would be expected in future, the ratio between average and maximum may show more larger than 1.20.

d) Determination of maximum daily demand

On the basis of the above conclusion, it is consequently determined that the ratio between average daily demand and maximum daily demand is given in 1.5. This ratio has been used since few years ago throughout Thailand, so it has no more doubt about this determination.

4.2.3 Hourly water consumption

a) Other country

It is known that the maximum hour demand in smaller municipalities is much higher than that of the larger one. It is interested in Pakistan that the World Bank has ever approved the ratio of 2.0 or 2.25 between average daily and hourly demand. Further, in design criteria of Japan Waterworks Association, maximum hourly flow over daily maximum demand is 30% up for large municipality, 50% up for middle sized municipality, and 100% up for small municipality.

b) Field study

As well as daily maximum demand, the Second Colombo Plan Experts have checked the hourly variation of water consumption in Sri-racha, as shown in Table 4.20. Hourly maximum is recorded on 7.00 pm afternoon as 17.13/13.97 = 1.23

Table 4.19 Water Consumption in Sri-racha

*: Assuming 7 families/meter

MONTH	No. Meter 1969	No. Meter 1970	Amount of Water 1969 (m^3/min)	Amount of Water 1970 (m^3/min)	Population* 1969	Population* 1970	Amount of Water 1969 (m^3/day)	Amount of Water 1970 (m^3/day)	1/c.d 1969	1/c.d 1970	Ratio 1969	Ratio 1970
1	1,252	1,443	45,351	49,272	8,764	10,101	1,511.7	1,642.4	172.5	162.6		
2	1,265	1,458	50,662	65,873	8,855	10,206	1,688.7	2,195.8	190.7	215.1		
3	1,269	1,476	45,019	59,609	8,883	10,332	1,500.6	1,987.0	168.9	192.3		
4	1,289	1,490	54,690	58,171	9,023	10,430	1,823.0	1,939.0	202.0	185.9		
5	1,297	1,513	47,925	64,452	9,079	10,591	1,597.5	2,148.4	176.0	202.9		
6	1,308	1,524	49,633	53,298	9,156	10,668	1,654.4	1,776.6	180.7	166.5		
7	1,328	1,566	51,506	60,505	9,296	10,962	1,716.8	2,016.8	184.7	184.0		
8	1,337	1,628	46,113	60,318	9,359	11,396	1,537.1	2,010.6	164.2	176.4		
9	1,350	1,635	46,964	58,243	9,450	11,445	1,565.5	1,941.4	165.7	169.6		
10	1,383	1,654	44,933	50,617	9,681	11,578	1,497.8	1,687.2	154.7	145.7		
11	1,399	1,683	44,041	67,714	9,793	11,781	1,468.0	2,257.1	149.9	191.6		
12	1,414	1,699	63,302	61,462	9,898	11,893	2,110.1	2,048.7	213.2	172.3		
Total	—	—	—	—	—	—	—	—	2,123.2	2,164.9		
Max.	—	—	—	—	—	—	—	—	213.2	215.1	1.21	1.19
Min.	—	—	—	—	—	—	—	—	149.9	145.7	0.85	0.81
Aver.	—	—	—	—	—	—	—	—	176.9	180.4	1.00	1.00

Table 4.20 Hourly Variation of Water Consumption. (Mean Value for three days in Jan. 1971)

Time	Q(m^3/hr.)	Time	Q(m^3/hr.)
1	10.94	14	15.43
2	10.58	15	14.73
3	10.30	16	14.83
4	10.40	17	15.85
5	10.58	18	16.73
6	11.54	19	17.13
7	12.98	20	16.13
8	15.40	21	14.75
9	16.23	22	13.95
10	16.45	23	11.34
11	16.38	24	10.66
12	16.33	Total	335.39
13	15.75	Average	13.97

c) Determination of hourly water consumption

Taking into account the above facts and data it has come to the conclusion that the ratio between daily maximum and hourly maximum is given in 1.5 which has been used in Thailand since few years ago.

4.2.4 Water Demand

a) General

Following by the conclusion above mentioned, the quantity of water required for thirty year's program will be as shown as Table 4.21.

Table 4.21 Water Demand in Future

Year	Population	House Connection (%)	Water Demand	
			l/c/d	m^3/day
1970	24,900	60	190	2,840
1980	33,800	65	210	4,620
1990	43,600	70	230	7,200
2000	53,800	75	250	10,090

The time schedule is made on the base of one year detailed design and one year construction, that is to say the construction will be finished at the end of 1973.

In addition to the water demand listed in Table 4.21, it is necessary to consider about water for fire fighting and for demand outside of the basic system area.

b) Water for fire fighting

It is reported that very few fire incidents took place during past years, although there are perfect arrangement of fire hydrants connected with the main pipe. A hour of fire occurrence were not identified but the fire broke out few times a year because residents do not use the fire always for heating.

The standard rate of discharge of fire fighting water per hydrant shall be more than $1m^3/min.$, and the discharge rate from a hydrant shall be assumed to be more than 500 l/min. when small fire pumps are used. There is a design rule about the amount of water in distribution system, in which a larger amount would be adopted among hourly maximum and daily maximum plus fire fighting water comparatively. In other word, in larger municipality, hourly maximum amount of water is always more than a sum of daily maximum and fire fighting water. On the contrary, small municipality has a chance that a sum of daily maximum and fire fighting is always over the hourly maximum amount.

However, if this rule is unconsiously applied to small municipality the distribution system is obliged to come into greater size than normal diameter. For developing country like Thailand, an economical design shall be prior to a perfect design in order to save the budget. When a fire would be occured, all people is better to save the use of tap water, emphasizing mutual cooperation against fire fighting. From these standpoint, it is concluded that the hourly maximum is a base of designing of distribution system.

c) Demand outside of the basic system area

At this moment, Pattani municipality is now discussing about the enlargement of municipal area, but ultimate decision may take long time, so that it is needed to consider about a demand outside of the present municipal area. One of important demand belongs to the Songkla University in which few hundreds students and other peoples concerned may use the water for drinking and miscellaneous purpose. In addition to the Songkla University, there is harbour and custom facility nearby the estuary of Pattani River.

Referring from past experiences, it is concluded that the demand outside of the basic system area must be $100m^3/day$.

d) Existing capacity

As mentioned above, there are three deep wells in existing water treatment plant, but one old well, $40m^3/hr$, and one new well, $80m^3/hr$, can be used at same time, totalizing as $120m^3/hr$ ($2,880m^3/day$).

There has been discussed about a capability or continuity of deep well, in addition to the heavy content of iron in existing wells. It is often said that deep well can not be used for more than 20 years without any repair or improvement. However, if the iron would be able to remove without any difficulty, the deep well should be utilized as long as possible. In other word, if it would be necessary to dig new well besides of existings, no man should be hesitated to do so.

However, looking about long-range plan, it shall be necessary to have an alternative plan for water source. That is to say, for 30 years master plan, the water source shall be capable to take fully whole requirements of water not only from underground but also surface, although the municipality would depend upon the underground water for 10 years program.

e) Ultimate water demand

Considering about several factors abovementioned, ultimate water demand shall be listed as shown in Table 4.22.

Table 4.22 Ultimate Water Demand

Year	Population	House Connection %	Daily Demand		Outside Need (m3/d)	Total Demand (m3/d)
			1/c/d	m3/d		
1970	24,900	-	-	-	-	-
1974	28,200	62	198	3,470	400	33,870
1980	33,800	65	210	4,620	500	5,120
1990	43,600	70	230	7,020	600	7,620
2000	53,800	75	250	10,090	800	10,890

Judging from existing capacity of wells and treatment plant, extension project must be done as shown as Table 4.23 and 4.24.

Table 4.23 Relationship between Water Demand & Existing Capacity

Phase	Total Demand m^3/d	Existing m^3/d	Needs for extension
1st	5,120	2,880	2,360
2nd	7,620	2,880	4,740
3rd	10,890	2,880	8,010

Table 4.24 Extension Schedule

Phase	Needs for extension m^3/d	Capacity of extension m^3/d	Total Capacity m^3/d
1st	2,360	2,400	2,400
2nd	4,740	2,400	4,800
3rd	8,010	3,300	8,100

4.3 Consideration of Alternatives

4.3.1 Design Criteria

Any water supplied by a water supply system must conform to the established standards of water quality. Simultaneously, the water supply system must have facilities and equipment conformable to the standards stipulated for the class of facilities.

From these standpoints, there is important need of design criteria and water quality standard. However, it shall be recommended that any criteria which have been prescribed in rich country with a shortage of labor and with a sophisticated technology should be carefully reviewed, and if necessary modified, before applying them to projects under different climatological, social and economic environments. It is absolutely necessary to determine design criteria as soon as possible, if otherwise, the desire to design a technically perfect scheme, using the most advanved techniques normally leads into the wrong direction. It is always necessary to attack an enemy who would let the design drive into excess free employing sophisticated devices.

The rationality of design will be started from points of segregation and standardization of design and will reach finally to the automatization. In general theory, there are plenty of work to be done as, connection between planning and economy; research for newly developed technology; preparation of standardization and criteria; speed up with higher accuracy of design and so on. It is easy to just list up such a plenty of work, but as a matter of fact there is not so many items which have been fixed in routine work. For instance, it can be listed only for pipe, valve, hydrant, section of excavation, pavement of road and so forth. Simple structure and piping may be possible to standardize, but it shall be even changeable depending upon different conditions. Piping also seems to be a simple work, but it would be changed awfully by an obstacle under the road.

There are many safety factors which should be included in the designing, such as qualitative, quantitative and structural factors intechnical viewpoint. In the quantitative safety, it should be noticed that the drinking standard had been made on the condition of natural contamination itself. In another work, it is necessary to consider about artificial pollution such as heavy metals from industry. No men have been known whether safety factor of drinking water standard has been involved, even though it has been decided by ways of health hazard and utility damage.

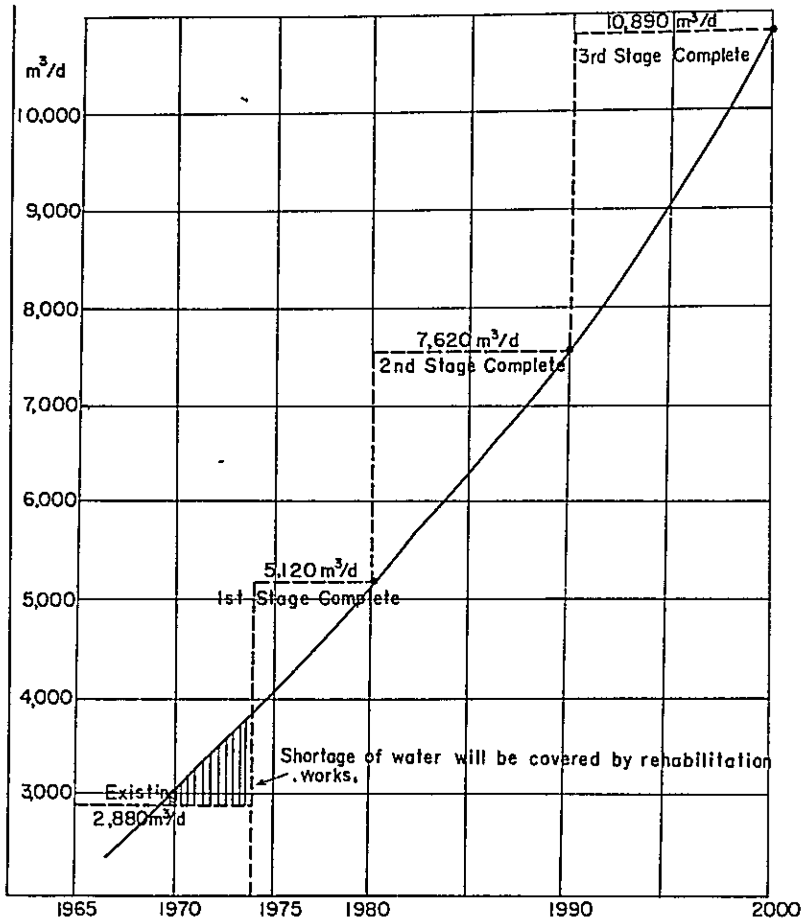
For structural analysis, it has been recently well-known that there is a word of "scrap & build" in industrial field. However, although supposing the safety factor is rather smaller value than usual, economical merit would not be expected too much under the condition of little smaller size of structure, because water supply facilities should have some minimum size by which no water would be leaked.

There is very difficult problem concerning to the concept of relationship between safety and economy. In general, both concepts shall be opposed each by each, but it depends upon individual appraisal which may be explained differently by human, generation and environment. For example, although future prospect would be little larger than usual in the demand of water, it would be unable to criticize the design as extravagant scale, because it is easy to expect the increasing amount of income by water rate and to be able to eliminate the social unwillingness caused by shortage of water.

4.3.2 General Consideration

As shown is Fig 4.4 the time schedule of extension project shall be discussed carefully, because the existing water source will be little short for ten years program up to that time when water can be supplied by underground water. In other word, shortage of water supply may be expected on the period between 1970 and 1975, although the preparation of existing project would be finished end of 1971 and moreover the construction would be finished end 1972. This means that the shortage of water will be covered by rehabilitation works if it is possible.

Fig. 4.4 Water Demand & Extension Project



There are two alternative plans about extension project. Namely, as Extension Schedule (1) in Fig 4.5, it is based in only ground water source, comparing with surface water for extension in addition to existing ground water as Extension Schedule (2) in Fig 4.6. Two alterates have either advantage or disadvantage respectively. The former has a merit of low cost of construction, in spite of disadvantage of iron content and discontinuity in ground water. The latter has doubt about salinity of surface water at high tidal water in future, but no doubt about continuity for long term in future prospect.

Fig 4.5 EXTENSION SCHEDULE (1)

Water source: Ground-water Only.

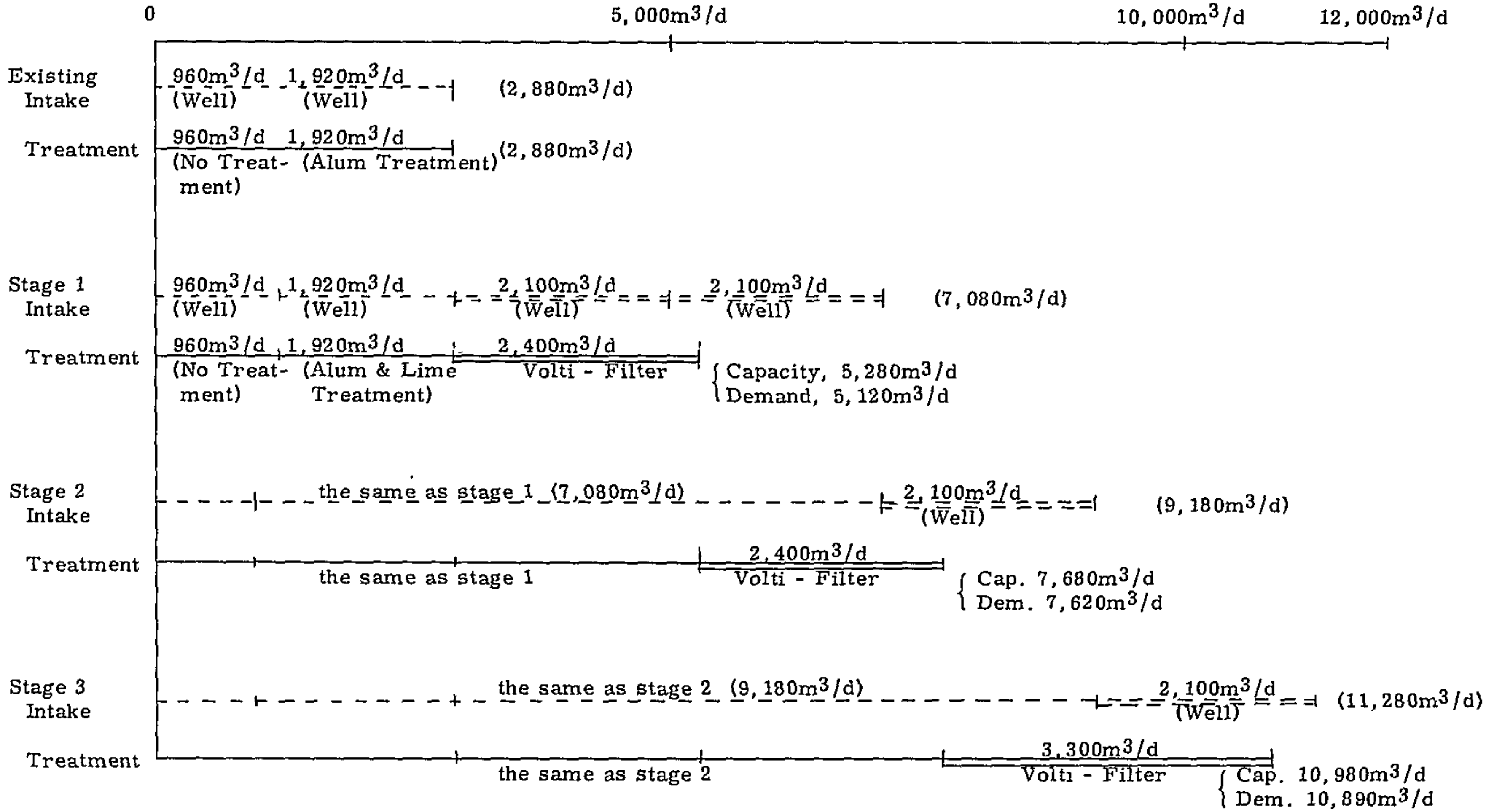
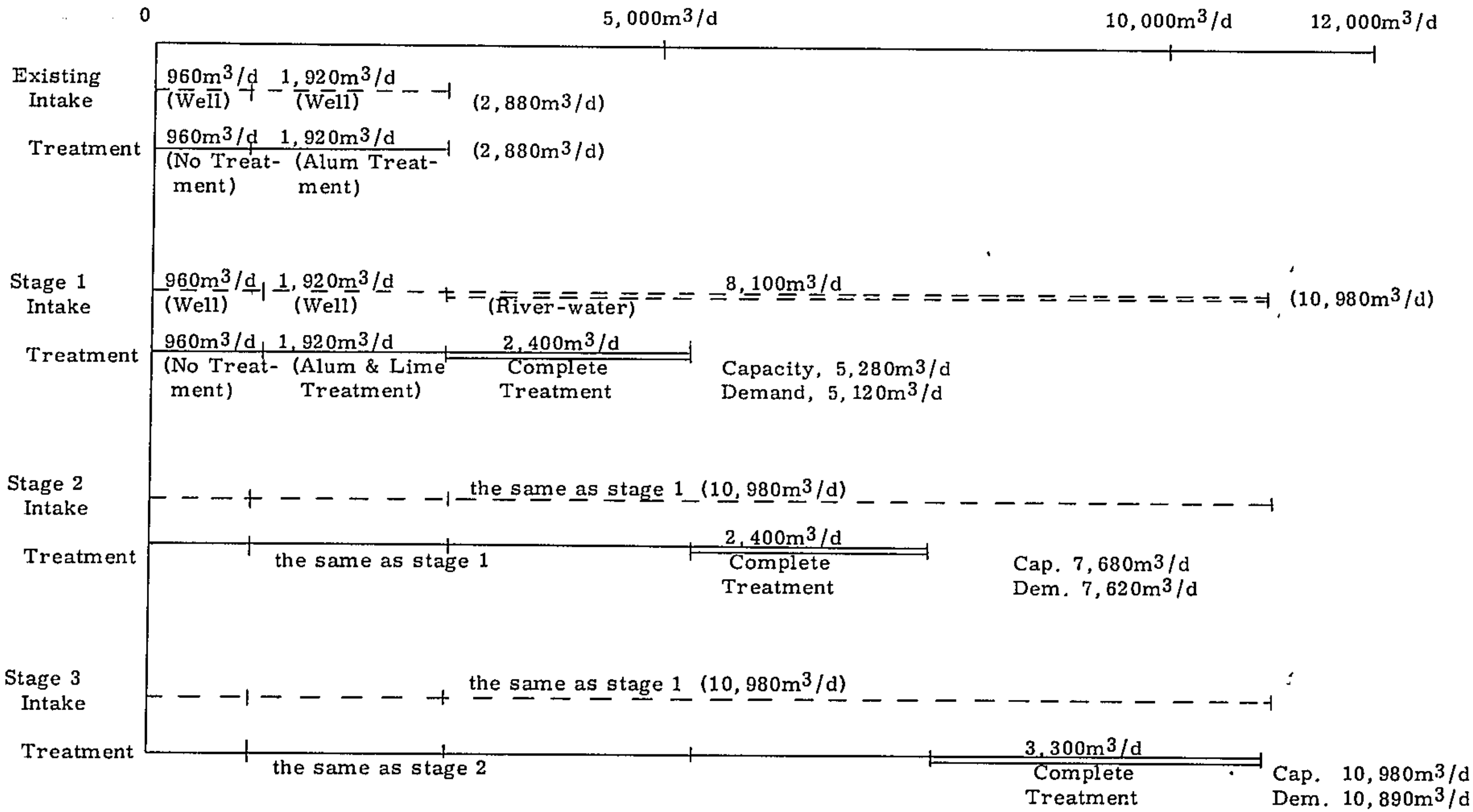


Fig 4.6 EXTENSION SCHEDULE (2)

Water source: Ground-water & Surface-water.



a) Rehabilitation Works (completed at end of 1972)

Before new construction will be started, the rehabilitation work should be taken up as shown in 3.2.3. Next important work in this program is that a leakage in existing water supply system should be prevented, and the improvement of existing water supply system should be carried out.

b) First Stage Works (completed at end of 1979)

In schedule (1), a new deep well will be expected to be dug nearby existing water treatment plant, considering about iron removal. Total capacity of well will be able to supply 7,080m^3/day including existing capacity as 2,880m^3/day, saying capacity as 4,200m^3/day for new well. Among new two wells, 2,400m^3/d is treated by Volti-Filter in result of total treated capacity 5,280m^3/d which is enough to cover the water demand as 5,120m^3/d.

In schedule (2), the existing wells and water treatment plant are used as well as the schedule (1). As mentioned above, the existing water treatment plant shall be improved by the rehabilitation work. However, as far as surface water is concerned intake facility could not be expanded often from time to time, but it shall be made at once in the first stage work in order save construction cost. Namely, intake facility of surface water has capacity of 8,100m^3/day which can cover all water demand in AD 2000. On the contrary, as far as the water treatment of surface water is concerned, the capacity is only limited for 2,400m^3/day which be able to cover the water demand at 1980 as 5,120m^3/day.

c) Second Stage Works (completed at end of 1989)

In schedule (1), another deep well will be dug. The capacity of deep well is 2,100 m^3/day as well as 1st Stage Works. Then, total capacity at this stage is 9,180m^3/day as far as the intake is concerned, but additional water treatment is scheduled is 2,400m^3/day for this stage in the result of total capacity as 7,680m^3/day.

In schedule (2), the raw water already enough, but new complete water treatment shall be made on the lase of 2,400m^3/d. Then total capacity of treatment is 7,680m^3/day which can cover water demand of 7,620m^3/d at AD 1900.

d) Third Stage Works (completed at end of 1999)

In schedule (1), further one more well will be dug, as same capacity as abovementioned. Then, ultimate capacity comes to 11,280m^3/day which covers water demand of 10,890m^3/d at AD 2000. Additional water treatment is also scheduled as 3,300 m^3/day for this stage in the result of total capacity of 10,980m^3/day.

In schedule (2), as mentioned above, the raw water is good enough, so it is only necessary to expand water treatment plant 3,300m^3/d more in addition to existing water treatment facility, in the result of total capacity as 10,980m^3/day.

Chapter 5 Intake Reconnaissance

5. 1 Possibility of Underground Water

There are several kind of report about underground water throughout country, but almost of all report was based on investigation around Bangkok Metropolitan Area. This chapter is the summaries of the reports prepared by peoples or companies concerned.

a) Geraghty & Miller

The Central Chao Phya Valley is a geologic depression, which has been filled with sediments. The uppeı formation underlying the Bangkok metropolitan area consists of alternating beds of sand, gravel, clay and silt. However, a few thin, cemented layers of the order of 3 to 6 meters thick have been encountered at depths as shallow as approximately 100 meters. Although the entire sequence of unconsolidated deposits in the Bangkok area, it is reported that consolidated rock was encountered at a depth of 365 meters at Ayudhya

Even though the unconsolidated deposits act essentially as one hydrologic unit, the clay layers impede the movement of water between the permeable sand and gravel beds. Even under severe pumping conditions, the vertical movement of groundwater between two apuifers is very slow. In other words, each artesian a quifer exchanges water with other aquifers by slow vertical leakage through the overlying or underlying clays.

Some of existing wells have shown a loss in the capacity to produce as much water per meter of drawdown as initial tests indicated. The aquifers themselves are as transmissive as they always were and still can yield in clude plugging of the gravel pack and aquifer adjacent to the well wish silt and fine sand and, on occasion, in crustation of the well screen openings.

Within the central and northern part of the Bangkok municipality, development of aquifers shallower than 115 meters is expected to be practical. From data available, this gone appears to contain fresh water in this area. Aquifers deeper than 115 meters, except for those deeper than 270 meters for which little information is available, have experienced salt water encroachment. In addition, the piezometric head in the heavily pumped aquifer at 150 meters is declining fairly rapidly. Development of either the shallow aquifers or those deeper than 270 meters should be preceded by a program of test drilling in order to establish water quality and other geologic and hydrologic characteristics.

b) Takasuke Suzuki

Most of the cities in Thailand seem to get fresh water for human consumption and industrial purposes mainly from surface water. It is, however, not favourable to use the surface water as large-scale water resources to the cities in the future, because it has various limitations such as the delicate water concession, the large seasonal variation of river water level reaching more than 4 meters, the recently progressing water pollution, and so on. So, if people wants to get a lot of clean water as cheaper as possible, it must be advisable to get it from groundwater or from reservoir.

In Thailand, the areas allocated in the wide alluvial plain along the Chao Phya River and its tributaries are densely populated. In and around this plain, very few diluvial terraces have been developed. This fact may indicate that the alluvial deposits in this plain have their thickness of more than 100 meters. The uppermost part of this alluvial deposits in the outskirts of Bangkok is composed of a reddish brown silty-clay layer more than 10 meters thick. The occurrence of this thick silty-clay layer seems to have been an obstacle to bore the shallow digging wells which could hardly be seen in the Chao Phya plain.

According to some drilling data of this plain, however, there are some gravel beds more than 10 meters thick below the uppermost silty-clay layer mentioned above. These gravel beds are interrelated with clay and silty sand beds, and behave themselves as good aquifer. The thickness of these gravel beds seems to become larger and larger to the periphery of the Chao Phya plain. So, if some one can clarify the subsurface geological structure of the Chao Phya plain, especially the thickness, number, depth of such gravel beds, it would be able to get a lot of groundwater easily from this plain. Furthermore, it may be possible to get even the confined water.

On the contrary, it is firstly necessary to check the topographic and geologic conditions regarding the construction of dam of reservoir. The Chao Phya plain is surrounded by some maturely dissected mountains which are topographically suitable for the construction of dams. These mountains are, however, composed mostly of granites and Paleozoic sedimentary rocks such as limestone, sandstone and so on.

Among these rocks, limestone becomes an obstacle to construct dam, because in the limestone areas, the developed karst topography is usually characterized by the existence of many caves which result in the leakage of water from the reservoir. It must also be considered the various unfavourable problems as follows: The large seasonal variation of river water level, the large evaporation, the rapid fill of reservoir by the deposition of suspended materials originated from laterite, and so on. From the same reasons above, it is also advisable to construct a larger reservoir rather than many small ones for the individual cities.

According to the aforementioned facts, it is concluded that the utilization of groundwater is better than getting water from the reservoir.

For the planning of utilization of groundwater, Govt. must firstly prepare the hydrological map showing the subsurface geological structure, the groundwater level, the distribution, depth, thickness, number of aquifer, the various hydrological constants, etc. However, there are not any maps yet which are called as the hydrological map of Thailand. The main existing data on the groundwater in Thailand are as follows: a report on groundwater survey of the plateau areas in the northeastern part of Thailand by the U. S. Air Force, some drilling data for industrial purposes near Bangkok, "Hydrological Year Books" showing the change of water level and discharge in the main rivers in Thailand, published by the Royal Irrigation Department, and so on. So, it is very difficult in the present state to plan the utilization of groundwater realistically.

The present age of Thailand thus demands the hydrological map, but in order to make such a map, it is necessary to carry on the geomorphological, geological, and hydrological survey including various geophysical prospectings, drillings, test, pumpings, and so on. It is obviously takes a good deal of money and time.

In conclusion, the author would like to suggest that the preliminary hydrogeological map in Thailand should be urgently compiled from the existing available data as the first step to make the final hydrological map, because it is a common way of groundwater survey.

c) Existing data nearby Pattani

As mentioned in Chapter 3, there are few data of drill log nearby Pattani such as Fig. 3. 2. It is interested that existing new wells (q_4 and q_5) is taking water from quifer above the thin clay layer about 320 ft. deep in comparison with old well (q_2) which pumps water under. Furthermore, old well has good quality of water having no particular iron, in spite of necessity of treatment for new wells.

According to other data such as a shallow well located in Pattani Air Port, it is almost impossible to drink without treatment of iron removals.

In consequence of existing data abovementioned and long year's experiences by inhabitants, it is possible to assume plenty of underground water nearby Pattani municipality if no men don't mind about iron content.

As far is chloride content, there is nothing only one evidence shown as 100 ppm but it might tend to increase day after day because of close location of sea-side. Therefore, it is needed to check a theoretical value of penetration of sea water using Ghyben-Herzbert Law as follows.

$$h_1 = \frac{S_1}{S_2 - S_1} h_2$$

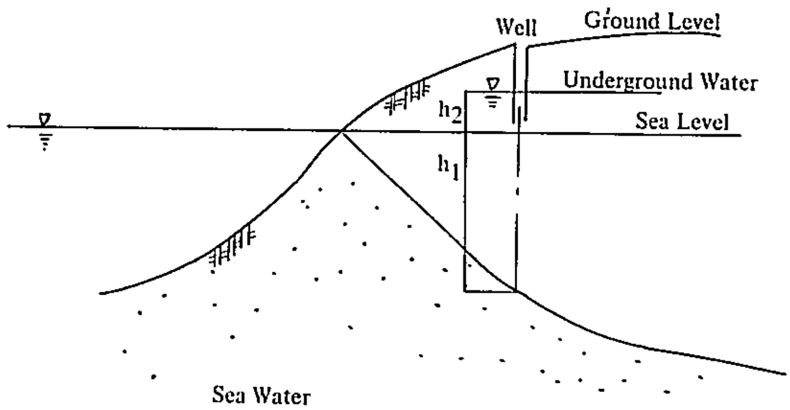
here: h_1 = height between sea level and boundary line (m)
h_2 = height between sea level and underground water level (m)
S_1 = density of underground water = 1 g/cm^3
S_2 = density of sea water = 1. 025 g/cm^3

This equation is little strange, since no factor of distance from well to sea is included, but supposing 1. 5 metres as ground level of treatment plant we can gain as follows:

$$h_2 = 1.5 - 1.08 = 0.42 \text{ m}$$

$$h_1 = \frac{1}{1.025 - 1} \times 0.42 = 17 \text{ m} < 92 \text{ m well depth}$$

Thus, there is a possibility of seepage of sea water if these data are correct. This is too simple theoretical approach against complicate surroundings under the ground, so it will be good enough to check carefully during operation whether or not underground water would be contaminated by sea water.



d) Future development

In order to determine the feasibility of future development of deep well, adequate groundwater testing will be required. Such work should be supervised by professional engineers trained and experienced in the field of groundwater and should be carried out by qualified drilling contractors.

In general, as an immediate necessity, it is recommended to organize the preliminary investigation commission of groundwater throughout Thailand as follows.

The purpose of the commission is a prepare the preliminary hydrogeological map at a scale of 1 : 500, 000 with both some hydrogeologic sections and an explanatory text. The commission will also be required to frame a plan of concluding the investigation in the future and to show a possible method of exploitation of groundwater for the present.

The commission may compile the existing data into the preliminary hydrogeological map and the explanatory text. In due course, the short general field-survey should have to be carried on.

Two or three hydrogeologists and one hydrologic engineer should be enough to be the consituents of the commission, and some assistants as official clerk, driver, coolie, and interpreter may be needed. It may take about one month for the commission to accomplish their purpose.

The followings are at least necessary for the efficient prosecution of the investigation commission.

1. Maps : a. 1 : 500, 000 and 1 : 250, 000 maps covering the whole realistic border of Thailand.
   b. 1 : 500, 000 contour maps covering the Chao Phya plain and its surroundings.
2. Aerial photographs at a scale of about 1 : 40, 000.
3. The existing geological maps and reports including any scientific papers.
4. Drilling data with their location map.
5. Data on surface water.
6. Meteorological data.
7. Data on population, land utilization and other economic activities related with water consumption.

These materials must be collected before the beginning of the activity of the commission.

5.2 Possibility of Surface Water

a) Irrigation Canal

According to the project of Royal Irrigation Department, the construction of Bangra diversion weir is scheduled on about 20 kilometers upstream from Pattani municipality at the end of 1973. In addition to the project, Bang Rang reservoir is also scheduled on about 70 kilometers upstream from Pattani at the end of 1975.

After completion of two projects, irrigation canal will be constructed not along the Pattani river to supply the water to irrigation area of Pattani and its neighbourhood from Bangra diversion weir. These relations are shown in Fig. 5. 1 and Fig. 5. 2. If the water would be enough to supply the raw water of water supply system throughout year from this canal, this water reconnaisance must be one of adequate intake. However, considering about terminal position of this canal, there is no absolute gurantee of water from the canal to water supply system in Pattani, and there is another doubt about the construction period of long distance of canal.

b) Pattani River

Discharge of Pattani River at Yala municipality was reported by Royal Irrigation Department as shown in Table 5. 1. The minimum discharge during three years listed in Table 5. 1 was 14 m^3/sec (1, 209, 600 m^3/day), but after the completion of Bang Rang Reservoir, discharge has not been fixed in vicinity of Pattani. However, so far as water quantity is concerned, there is no doubt about the capability of intake for water supply system in Pattani.

On the contrary, about water quality, influence of saline water has been reported from Royal Irrigation Department as shown in Table 5. 2, corresponding with Fig. 5. 3 for the sampling spots. The problem is the backwater of sea water in vicinity of Pattani municipality, as well as No. 2 sampling spot in Fig. 5. 3. However, since 1969 which was irregular period of shortage of water due to the scarcity of rain fall, there is no additional simulated data recorded in Pattani municipality. For example, Table 5. 3 has been investigated by Japanese Experts in 1971, in which there is no significant high chloride content in downstream of Pattani River.

Table 5.1 Discharge of Pattani River at Yala

(unit : m^3/sec)

Water Year \ Discharge \ Month		Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.
1965	Max.	41	128	42	40	144	142	378	377	1,633	616	193	84
	Min.	15	34	18	17	17	30	51	134	201	102	70	48
1966	Max.	73	79	101	110	72	262	344	584	1,409	3,463	258	146
	Min.	36	38	33	35	31	25	80	174	180	180	101	59
1967	Max.	115	155	88	68	55	97	102	471	310	58	30	20
	Min.	52	57	43	36	34	31	53	56	56	30	18	14

Remarks : These data are reported by Royal Irrigation Department.

Fig 5.1 Map of Pattani & Yala

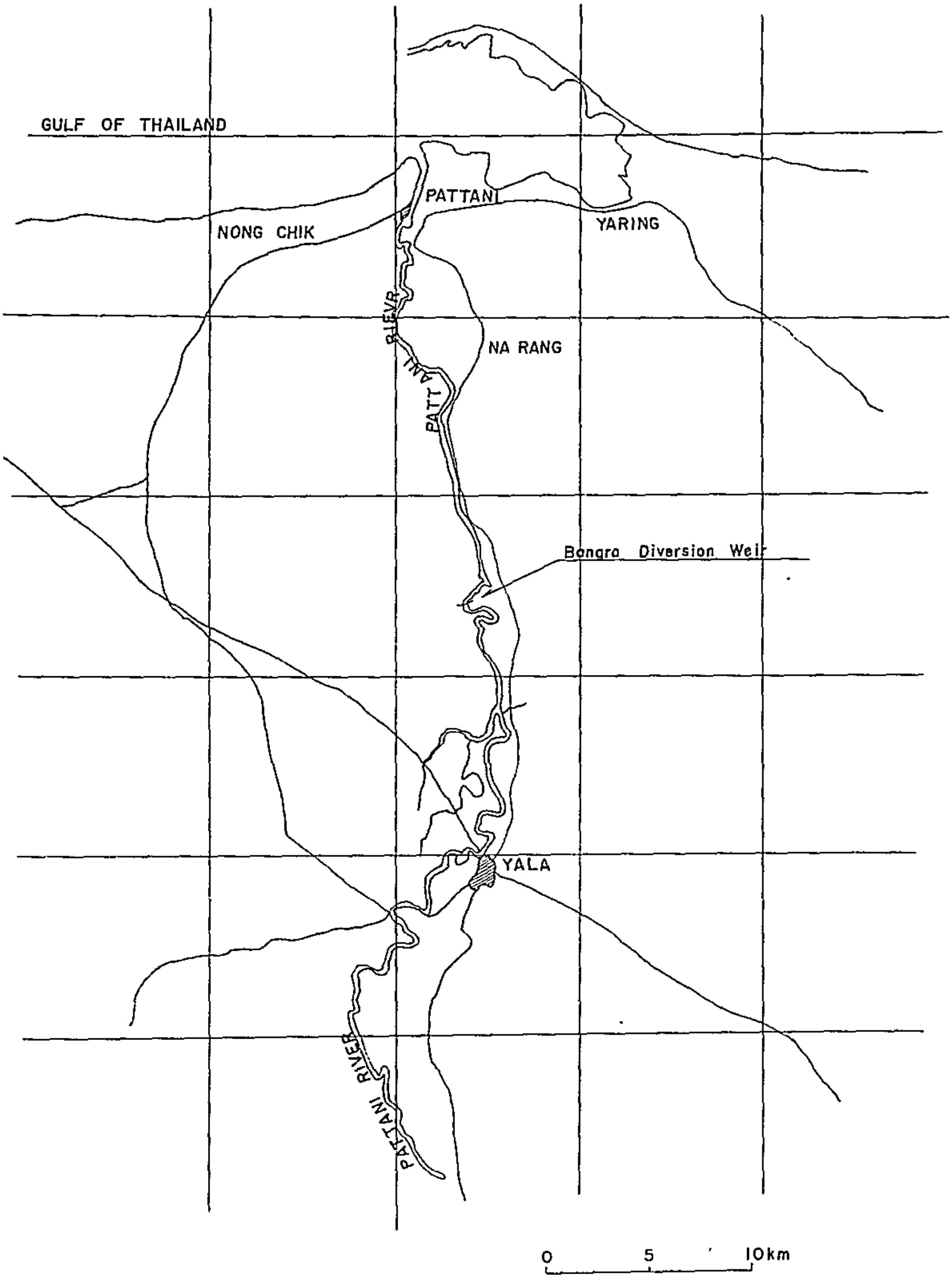


Fig 5.2: RID CANAL PROJECT

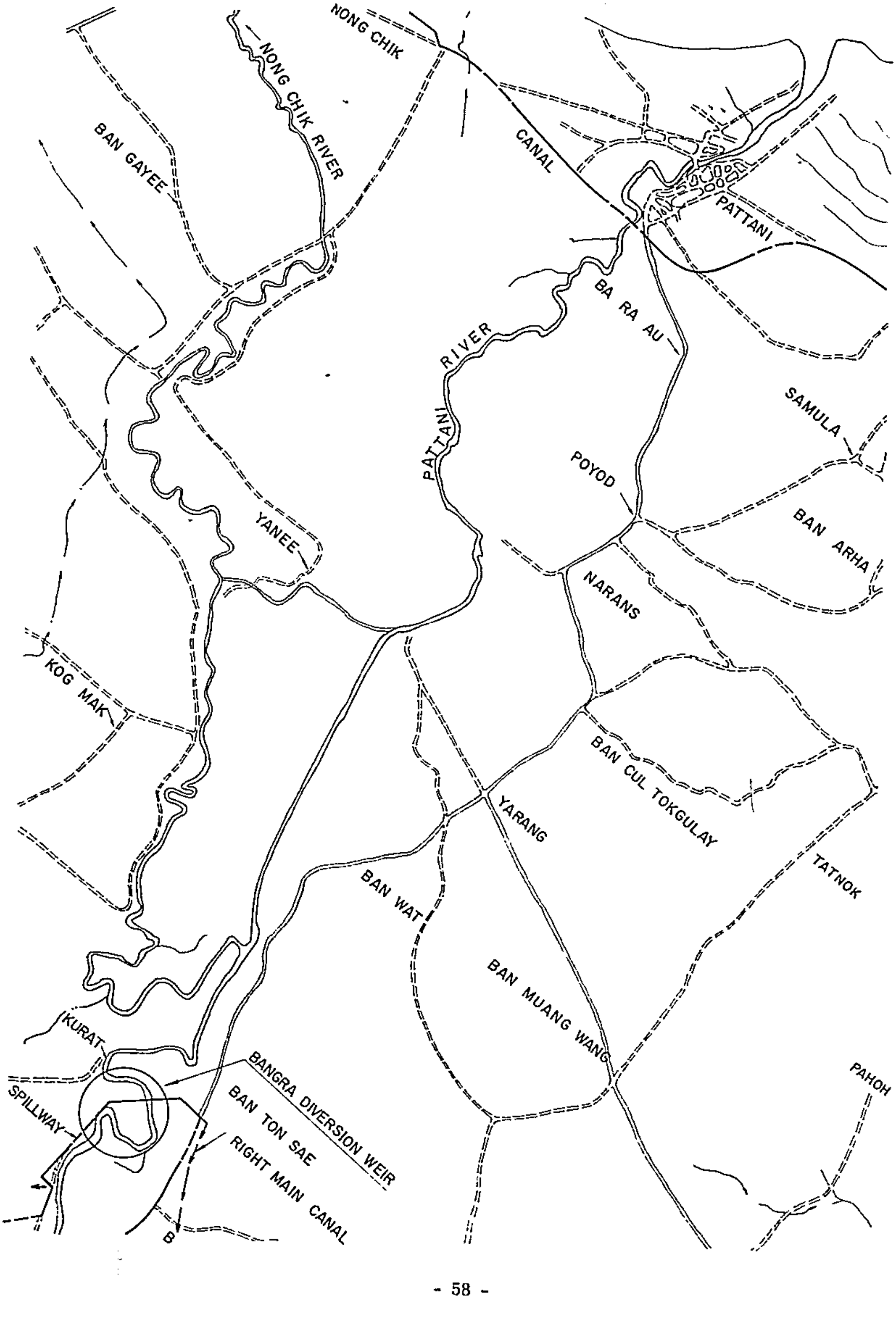


FIG 5.3: SALINITY OF PATTANI RIVER SAMPLNG POINT

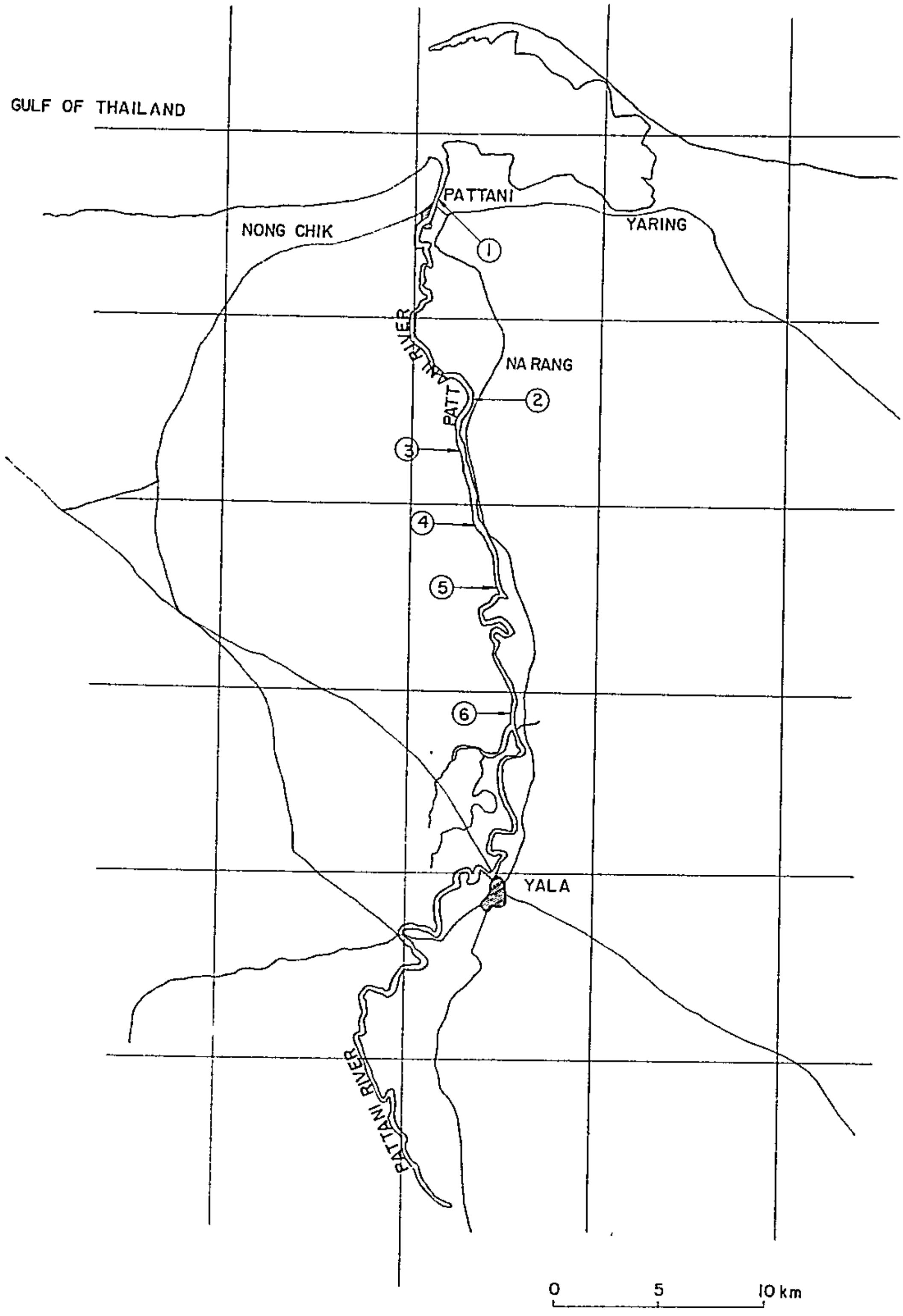


Table 5.3 Chloride Content in Pattani River

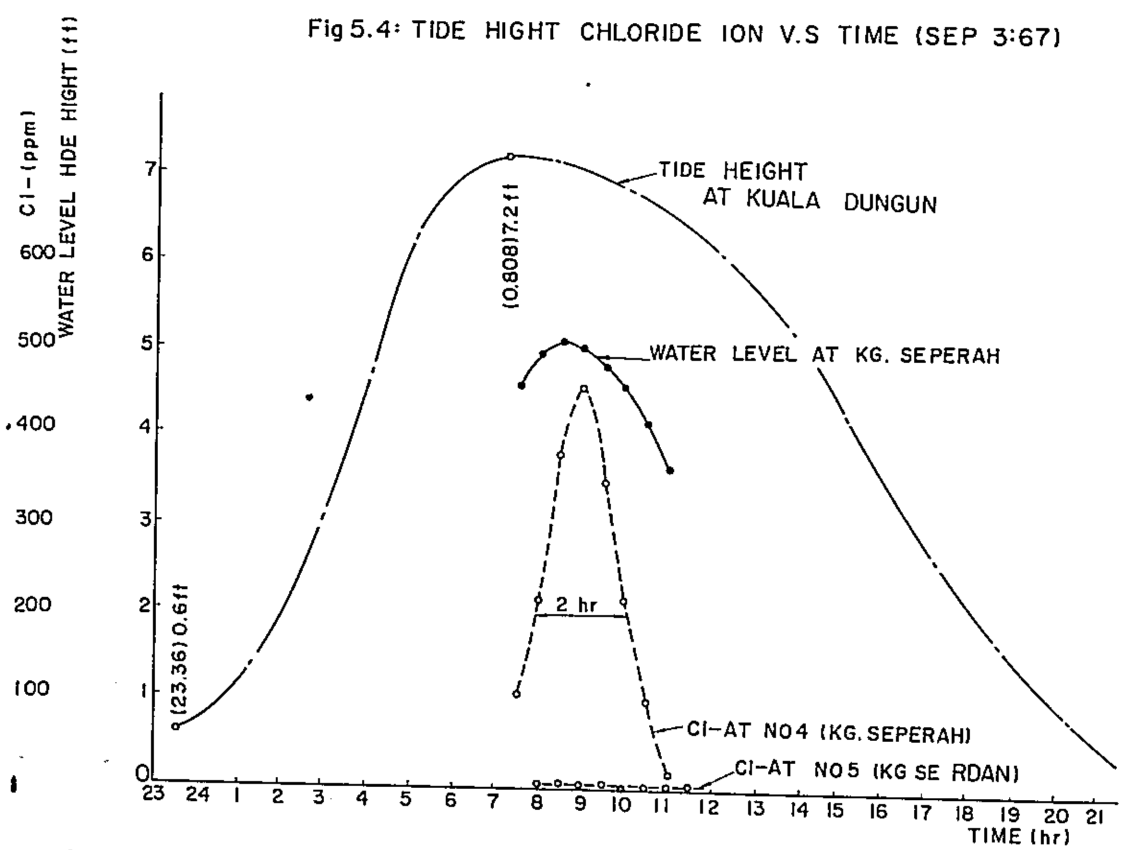
(12 Feb. 1971)

Time	pH	Chloride (ppm)		Remark
		30 cm*	100 cm*	
7	7.0	6.39	7.10	
8	7.1	6.82	6.82	
9	7.8	6.82	6.82	
10	8.1	6.82	5.68	Tidal water
11	8.1	6.39	5.68	
12	7.8	6.39	5.68	
13	8.1	5.82	6.53	
14	8.1	6.11	5.54	
15	8.0	5.82	6.39	
16	7.5	5.68	5.68	
17	7.6	6.25	5.82	
18	7.6	5.82	5.82	
19	7.6	5.65	6.11	

* : under the water level

On the other hand, according to the report of Kuala Dungun Water Supply in Malaysia which is located on about 2° latitude east and 2° logitude south from Pattani, the influence of sea water is shown as Fig. 5.4 in seaside. Namely, the duration of high chloride content over 200 ppm is limited for about 2 hours in September. From these data, it is possible to assume for 3 hours of high chlorids content in spring. In other

Fig 5.4: TIDE HIGHT CHLORIDE ION V.S TIME (SEP 3:67)



word, the river water would be able to use for water supply intake by means of shutting a gate during high tide for 3 hours. There are, of course, disadvantages of stoppage in the result of increase of purification or storage facilities, but it may be covered by expenditure for long distance of conveying water from 10 km towards the upstream.

c) Ultimate Solution

In consequence of discussion abovementioned, as far as surface water is concerned, it can be said that the Pattani River is best water source among three alternative plans such as irrigation canal, water fall and Pattani River. However, final decision shall be done after being checked for tidal water, in addition to other investigation of underground water reconnaissance. At this moment, the underground water has much higher possibility than the surface water from Pattani River although there are many iron content in underground water, because of plenty underground water undoubtedly.

d) Water Fall

According to the investigation of water fall in Tam Bae, it has been found that there are plentiful and clean waters although detailed investigations shall be done for not only quantity but also for quality throughout a year. However, unfortunately, the water fall is located 40 km far from Pattani municipality, in the result of high construction cost (about half million Bahts, considering 230 meters of loss of head by 450 mm cost iron pipe).

The advantage of this kind of plan is concerned with extensive water works which are being used for various water works such as those involving supplying area outside of administrative region, or those grouped in small scale water works and so forth. Usually, the construction cost of extensive waterworks is considerably higher than that of ordinary waterworks. Financial standings of regional public authorities are sometimes insufficient to meet the expenses of works though the necessity of construction is acknowledged. Therefore, under such circumstances, financial supports by Central Government are absolutely called for.

Chapter 6 Proposed Plan

6.1 Proposed Plan on Underground Water (excluding distribution system)

6.1.1 Water Demand

As far as the extension project is concerned, following figures have been decided in ultimated conclusion.

Table 6.1 Proposed Plan

	1st Stage	2nd Stage	3rd Stage
Water Demand (m^3/d)	5,120	7,620	10,890
Existing Cap. (m^3/d)	2,880	2,880	2,880
Balance (m^3/d)	2,240	4,740	8,010
Intake Cap. (m^3/d)	4,200	6,300 (+2,100)	8,400 (+2,010)
Water Treatment Cap. (m^3/d)	2,400	4,800 (+2,400)	8,100 (+3,200)

( ) : additional capacity over the last stage

6.1.2 1st Stage Works

a) Deep Well .... 2 ea

Intake quantity per each: 2,100 m^3/d (available for 1st stage only)

Diameter: 300 mm
Depth: 110 meters

b) Deep Well Pump .... 2 sets

Type: submerged pump

Capacity per each: 1.5 m^3/min. x 30^m x 2,900 rpm x 15KW
(available for 1st stage only)

c) Raw Water Main (Route 1, 2)

Material: Asbestos Cement Pipe
Diameter: 150 mm
Length: 750 meters each (available for 1st stage only)

d) Recieving Well .... 1 unit

Structure: Reinforced concrete
Dimension: 1.5^m (W) x 3.4^m (L) x 1.9^m (H)

Capacity: 9.7 m^3 (available for three stages)

Detention Period: 1.7 min. (for total capacity in 3rd stage)

Table 6.2 Summary of Extension Project on Underground Water

Item	1st Stage	2nd Stage	3rd Stage
Deep Well	300^{mm} x 110^{m} deep - 2^{ea}	300^{mm} x 110^{m} deep - 1^{ea}	300^{mm} x 110^{m} deep - 1^{ea}
Deep Well Pump	1.5 m^3/min x 30^{m} - 2 sets	1.5 m^3/min x 30^{m} - 1 set	1.5 m^3/min x 30^{m} - 1 set
Raw Water Main	150^{mm} x 750^{m} long x 2 sets	150^{mm} x 800^{m} long x 1 set	150^{mm} x 600^{m} long x 1 set
Receiving Well	1.5^{m} x 3.4^{m} x 1.9^{m} - 1 unit	——	——
Mixing Basin	1.5^{m} x 1.5^{m} x 1.5^{m} - 2 sets	——	——
Rapid Mixer	1.5 KW - 2 units	——	——
Flocculation & Filtration Chamber	22.2 m^2 - 1 unit	22.2 m^2 - 1 unit	26.4 m^2 - 1 unit
Alum Feeding Equipment	1.94^{m} dia. - 1 set	——	1.94^{m} dia - 1 set
Lime Feeding Equipment	1.94^{m} dia - 1 set	——	1.94^{m} dia - 1 set
Chlorination Equipment	1.88 kg/h - 1 set	——	1.88 kg/h - 1 set
Necessary Pipe & Valves	1set	1set	1set

e) Mixing Basin .... 2 sets

Structure: Reinforced concrete

Dimension: 1.5^m (W) x 1.5^m (L) x 1.5^m (H)

Capacity: $6.75m^3$ for 2 sets (available for three stages)

Detention Time: 1.2 min. (for total capacity in 3rd stage)

f) Rapid Mixer .... 2 units

Capacity: available for three stages
Type: Vertical shaft with impeller, motor drive
Motor: 1.5 KW with CYCLO reducer
Attachment: Walkway, Ladder, Handrail

g) Flocculation & Filtration Chamber .... 1 unit

Capacity: available for 1st stage only

Type: Volti-Filter, VFR - 53, Mild Steel

Dimension: 5.32^m (D) x 3.8^m (H)

Flocculator: Vertical shaft with impeller, 0.75 KW motor drive, CYCLO reducer

Filter: 108 m/day x 22.2 m^2 = 2,400 m^3/d, Anthracite medium and Manganese sand, underdrain system

h) Alum Feeding Equipment .... 1 set

Capacity: available for 1st & 2nd stages

Dissolving Tank: 1.94^m (D) x 1.21^m (H)

Agitator: Vertical type, gear drive with 0.75 KW motor

Chemical Pump: 3.55 l/m x 15^m x 0.4 KW

i) Lime Feeding Equipment .... 1 set

Capacity: available for 1st & 2nd stages

Dissolving Tank: 1.94^m (D) x 1.21^m (H)

Agitator: Vertical type, gear drive with 0.75 KW motor.

Chemical Pump: 3.55 l/m x 15^m x 0.4 KW

j) Chlorination Equipment ... . 1 set

Capacity: available for 1st & 2nd stages

Chlorinator: Vacuum type, 1.88 - 0.09 Kg/H

Cylinder: 2-50 Kg chlorine cylinder

Balance: 1-50 Kg

k) Necessary Pipe & Valves .... 1 set

l) Drainage Pit & Pump .... 1 set

Pit: 4^m (w) x 4^m (L) x 5^m (H)

Pump: 2.0 m^3/min x 10^m x 7.5 KW x 950 rpm

6.1.3 2nd Stage Works

a) Deep Well .... 1 ea

Capacity as well as 1st stage

b) Deep Well Pump .... 1 set

Capacity as well as 1st stage

c) Raw Water Main (Route 3)

Material: Asbestos Cement Pipe
Diameter: 150 mm
Length: 800 m (available for 2nd stage only)

d) Flocculation & Filtration Chamber .... 1 unit

Capacity as well as 1st stage

e) Necessary Pipe & Vlaves .... 1 set

6.1.4 3rd Stage Works

a) Deep Well .... 1 ea

Capacity as well as 1st or 2nd stage

b) Deep Well Pump .... 1 set

Capacity as well as 1st or 2nd stage

c) Raw Water Main

Material: Asbestos Cement Pipe
Diameter: 150 mm
Length: 600 m (available for 3rd stage only)

d) Flocculation & Filtration Chamber .... 1 unit

Capacity: available for 3rd stage only

Type: as well as 1st or 2nd stage

Dimension: 5.8^{m} (D) x 3.8^{m} (H)

Flocculator: as well as 1st or 2nd stage

Filter: 125 m/day x 26.4 m^2 = 3,300 m^3/d, anthracite medium and manganese sand, underdrain system

e) Alum Feeding Equipment .... 1 set

Capacity as well as 1st and 2nd stages

f) Lime Feeding Equipment .... 1 set

Capacity as well as 1st and 2nd stages

g) Chlorination Equipment .... 1 set

Capacity as well as 1st and 2nd stages

h) Necessary Pipe & Vlaves .... 1 set

Fig 6-1 Location of Served Area

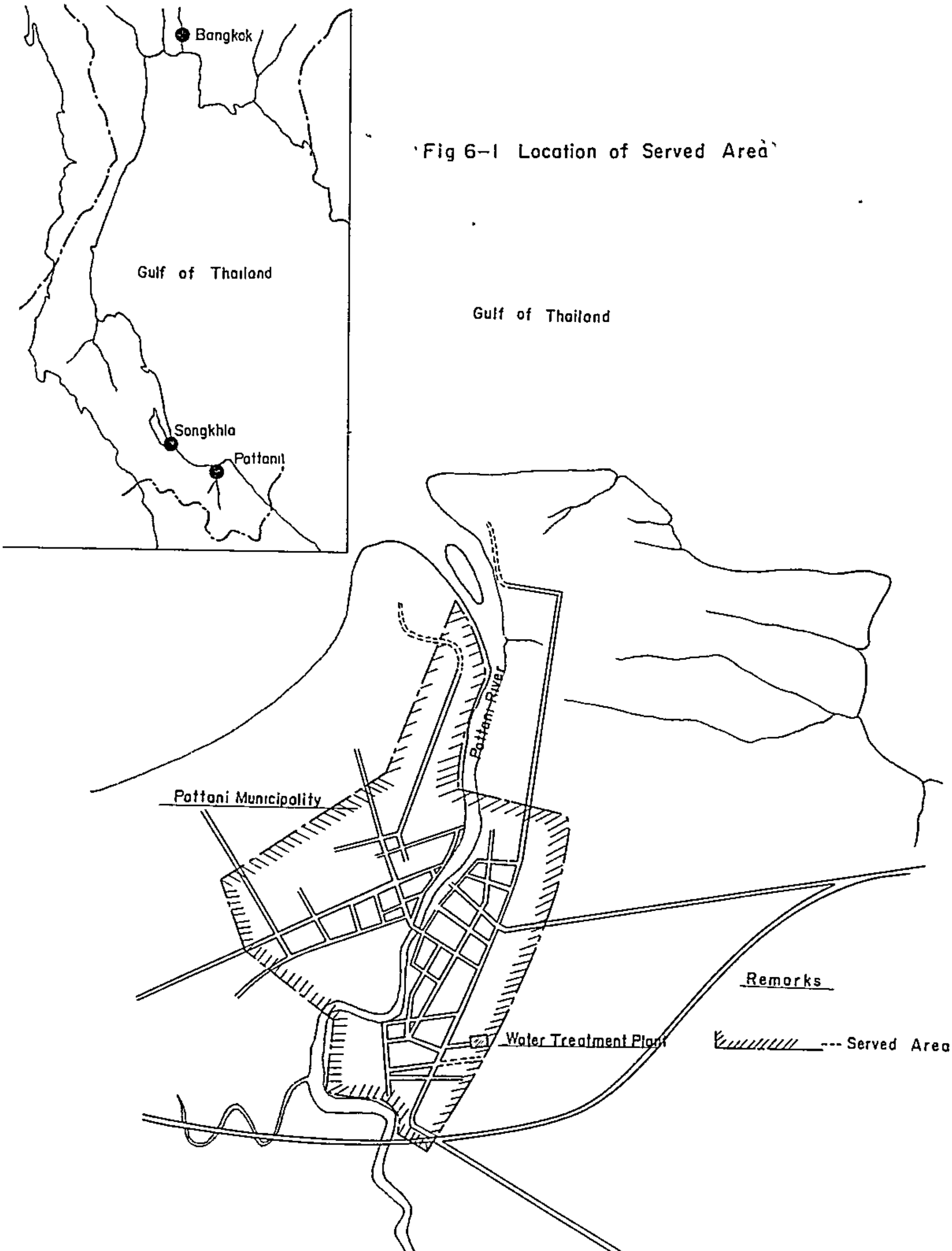


Fig. 6-2 Location of Proposed Wells

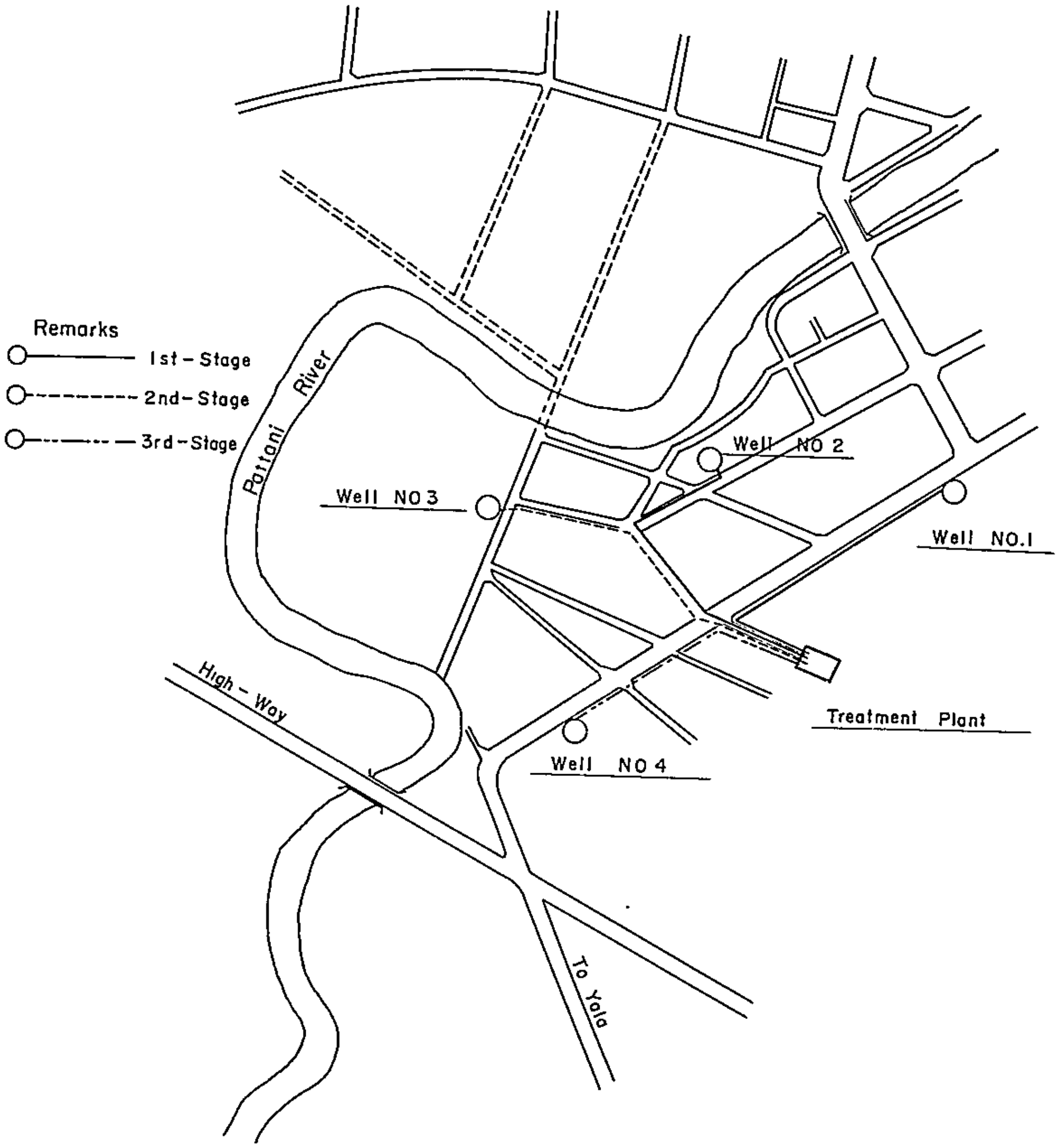


Fig 6-3 Intake Pump House

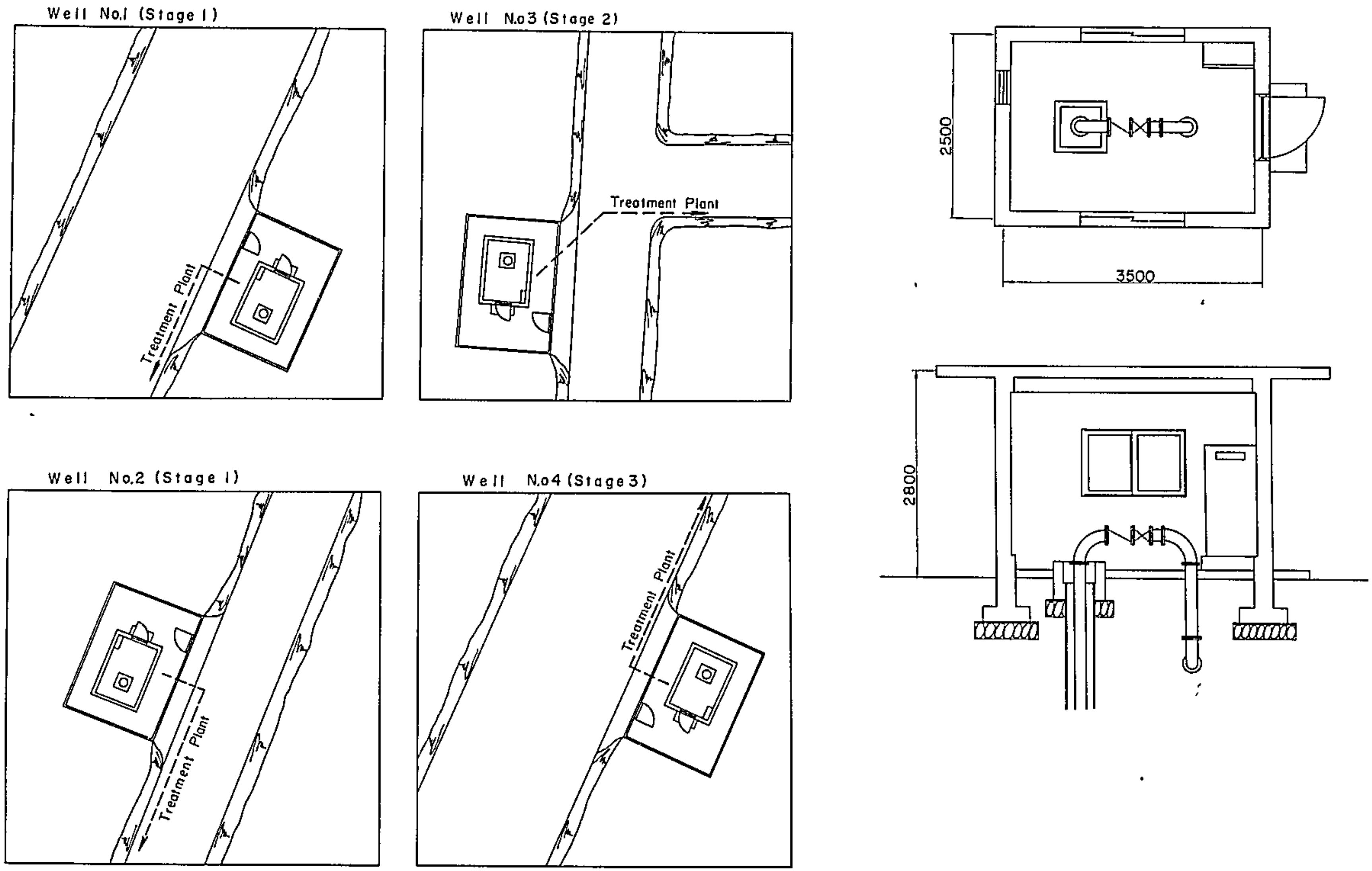


Fig. 6-4 Water Treatment Plant
(Underground Water)

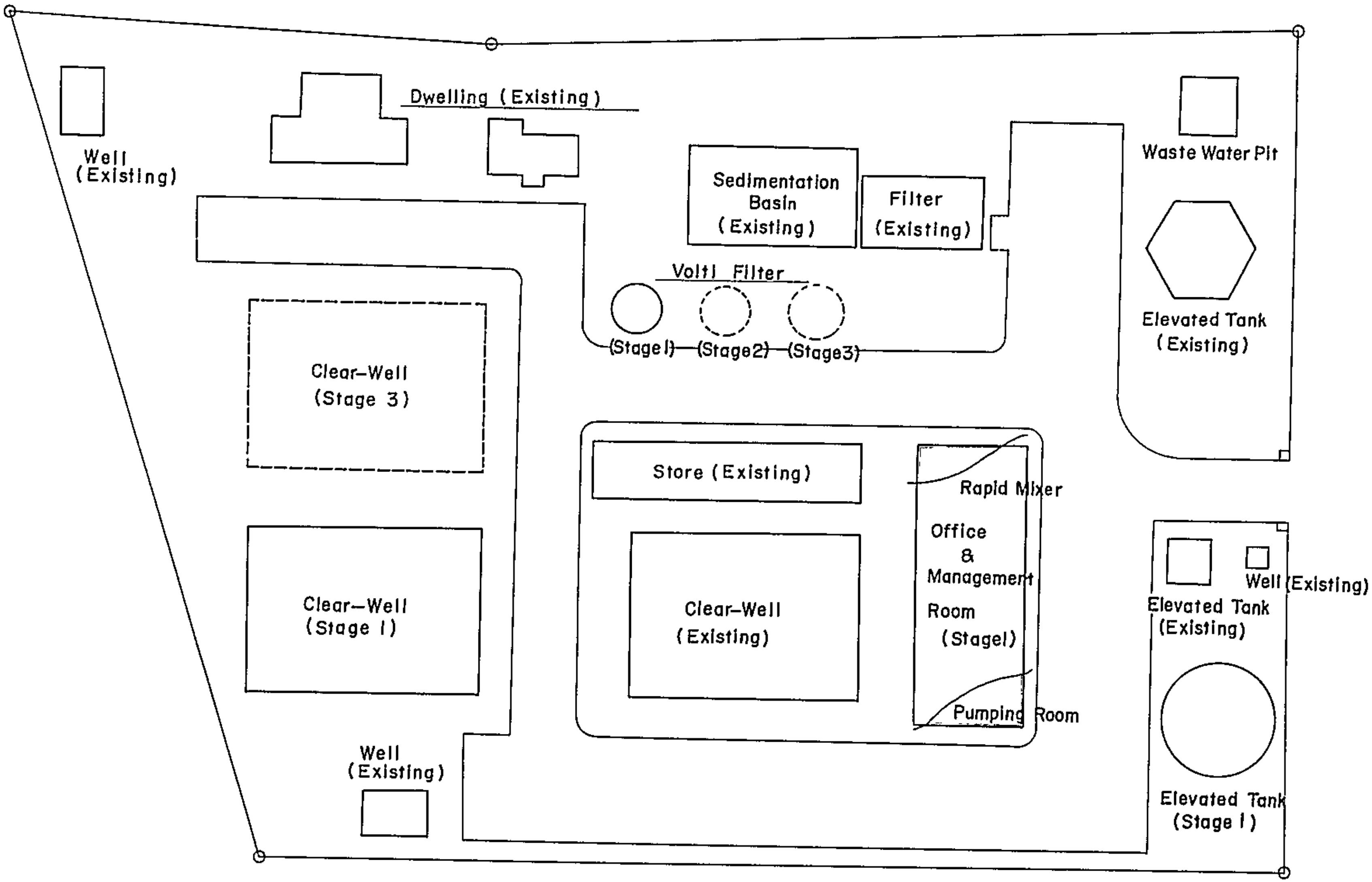


Fig. 6-5 Receiving Well & Mixing Basin

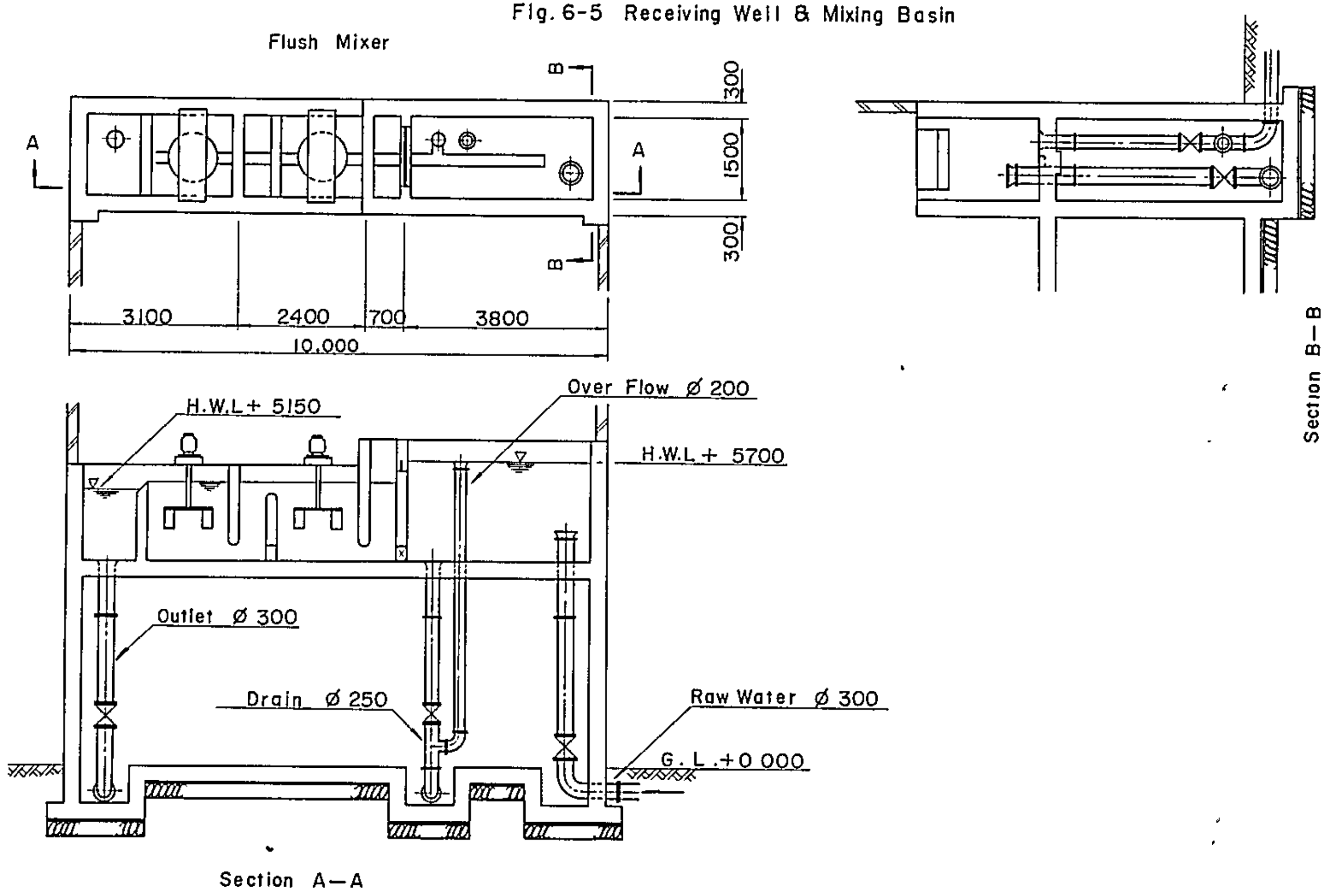


Fig 6.6 Layout of Water Treatment

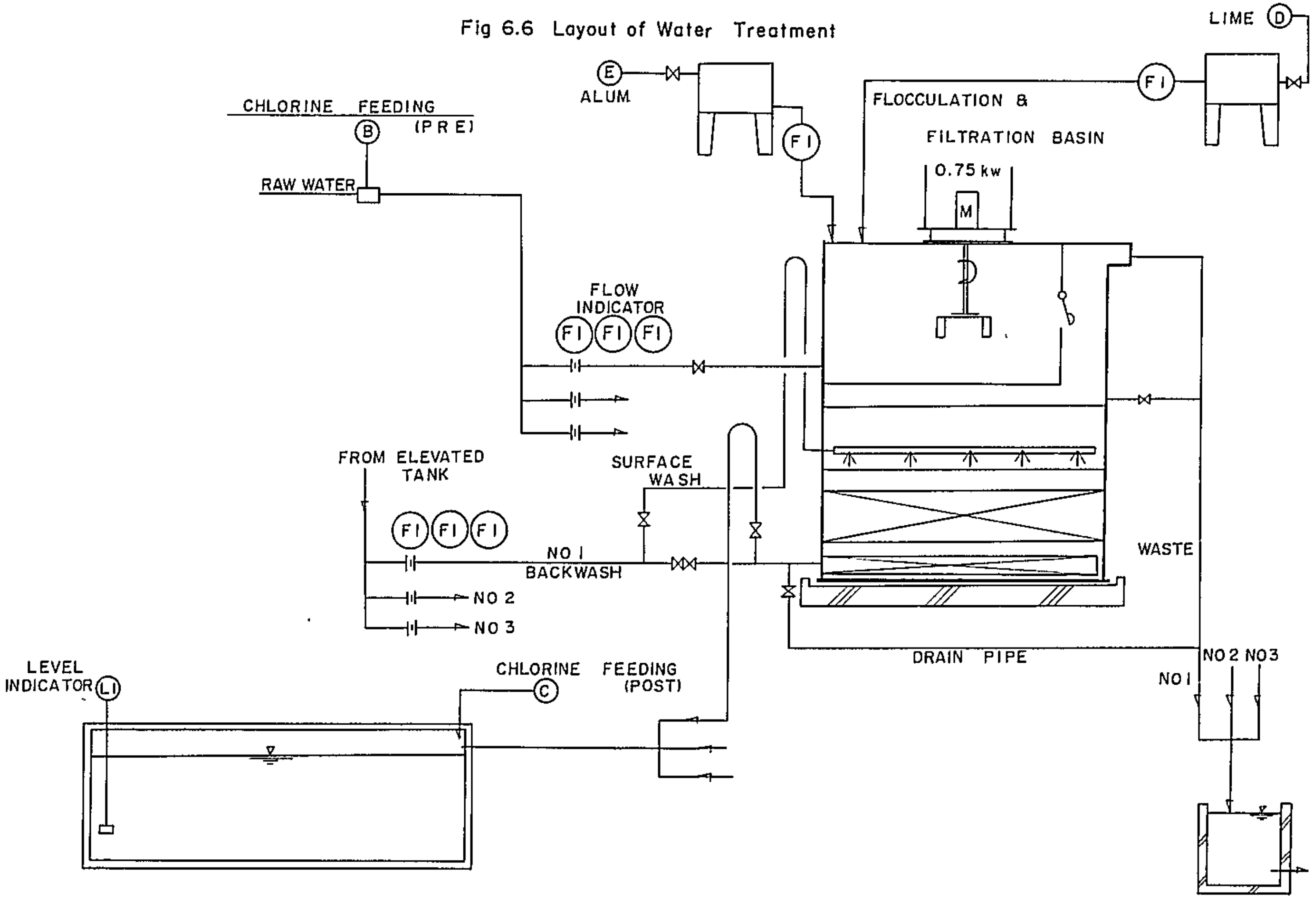
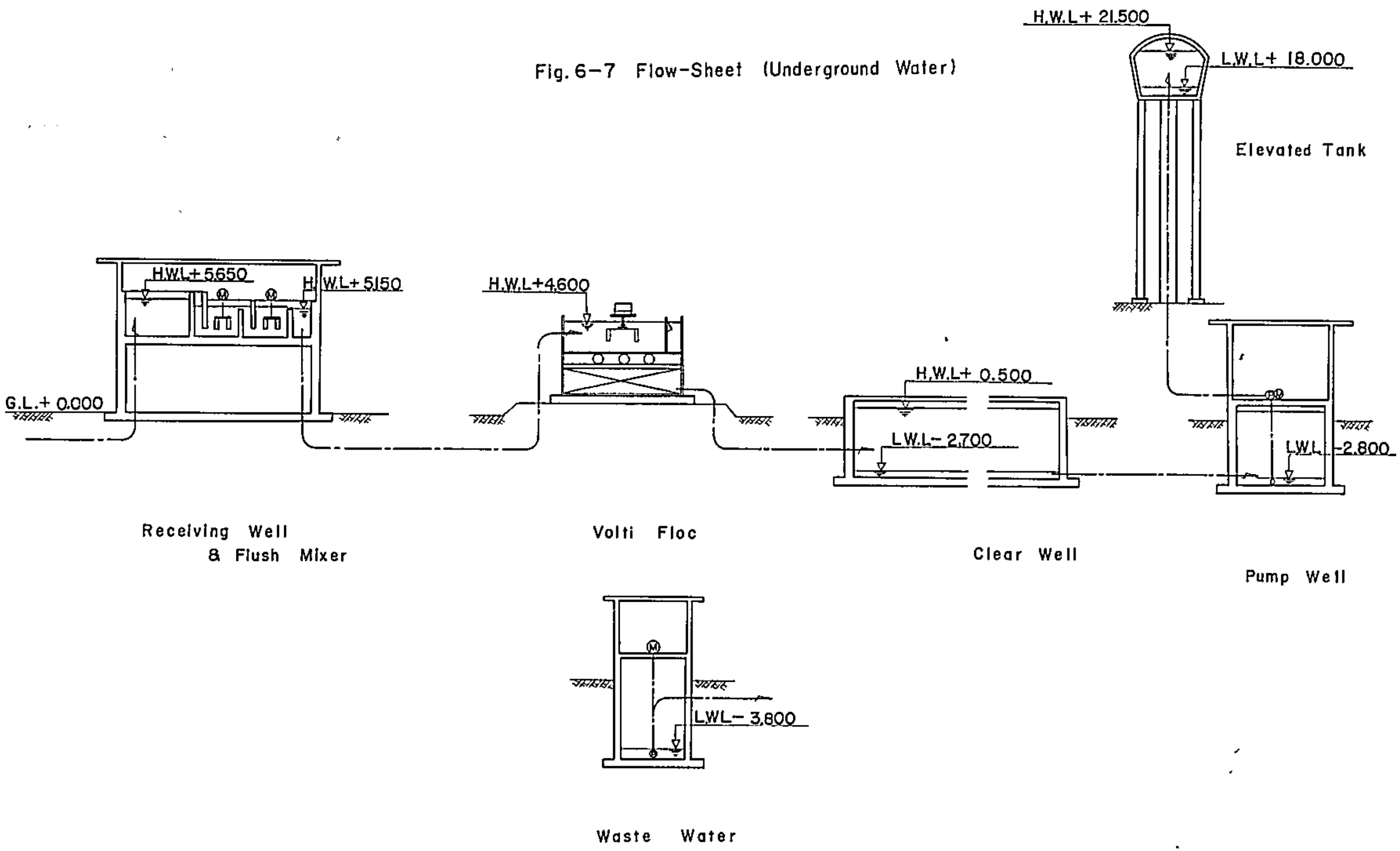


Fig. 6-7 Flow-Sheet (Underground Water)



6.2 Proposed Plan on Surface Water (excluding distribution system)

6.2.1. Water Demand

The water demand is same as 6.1.1 except the capacity of intake as shown in Table 6.3.

Table 6.3 Proposed Plan

	1st Stage	2nd Stage	3rd Stage
Water Demand (m^3/d)	5,120	7,620	10,890
Existing Cap. (m^3/d)	2,880	2,880	2,880
Balance (m^3/d)	2,240	4,740	8,010
Intake Cap. (m^3/d)	8,100	8,100 (0)	8,100 (0)
Water Treatment Plant (m^3/d)	2,400	4,000 (+2,400)	8,100 (+3,300)

( ) : additional capacity over the last stage

6.2.2 1st stage Works

a) Intake Facility .... 1 unit

Capacity: available for three stages, 8,100 m^3/d

Structure: Reinforced concrete

Dimension: 2.0^m (W) x 4^m (L)

Attachment: φ 400 mm pipe, screen, ladder and man-hole

b) Raw Water Main

Capacity: available for three stages
Material: Asbestos Cement Pipe
Diameter: 400 mm
Length: 10 m
Flow Velocity: 1.47 m/sec (for 3rd stage)

c) Pumping Well .... 1 unit

Capacity: available for three stages

Structure: Reinforced concrete

Dimension: 4.5^m (W) x 10.0^m (L) x 6.0^m (H)

d) Pumping House......................... 1 unit

Capacity: available for three stages

Structure: Reinforced concrete

Dimension: 7.0^m (W) x 10.0^m (L) x 3.0^m (H)

Table 6.4 Summary of Extension Project on Surface Water

Item	1st Stage	2nd Stage	3rd Stage
Intake Facility	$2.0^m \times 4.0^m$ - 1 unit	——	——
Raw Water Main	$400^{mm} \times 10^m$ long - 1 unit	——	——
Pumping Well	$4.5^m \times 10.0^m \times 6.0$ - 1 unit	——	——
Pumping House	$7.0^m \times 10.0^m \times 3.0^m$ - 1 unit	——	——
Raw Water Pump	For storage $1.87\ m^3/m \times 10^m$ - 2 sets	$1.87\ m^3/m \times 10^m$ - 1 set	$1.87\ m^3/m \times 10^m$ - 1 set
Raw Water Pump	For Transmission $1.87\ m^3/m \times 45^m$ - 2 sets	$1.87\ m^3/m \times 45^m$ - 1 set	$1.87\ m^3/m \times 45^m$ - 1 set
Raw Water Main	$250^{mm} \times 1450^m$ - 1 unit	——	——
Receiving Well	$1.5^m \times 3.4^m \times 1.9^m$ - 1 unit	——	——
Mixing Basin	$1.5^m \times 1.5^m \times 1.5^m$ - 2 sets	——	——
Alum Feeding Equip.	tank & agitator - 2 sets	——	——
Lime Feeding Equip.	E - 1 feeder - 2 sets	——	——
Flocculation Basin	$2.0^m \times 4.0^m \times 2.1^m$ - 2 sets	$2.0^m \times 4.0^m \times 2.1^m$ - 2 sets	$2.2^m \times 5.0^m \times 2.1^m$ - 2 sets
Sedimentation Basin	$4.0^m \times 17.0^m \times 3.0^m$ - 2 sets	$4.0^m \times 17\ 0^m \times 3.0^m$ - 2 sets	$5.0^m \times 18.5^m \times 3.0^m$ - 2 sets
Rapid Sand Filter	$9.18m^2$ - 3 sets	$9.18m^2$ - 2 sets	$9.18m^2$ - 3 sets
Chlorination	1.88 kg/hr. - 1 set	——	1.88 kg/hr. - 1 set
Necessary Pipe & Valves	1 set	1 set	1 set

e) Raw Water Pump

(1) For Storage (available for 1st stage only)
.... 2 sets including one spare

Type: Vertical Mixed Flow

Capacity: 1.87 m^3/m x 10^m x 7.5 KW

(2) For Transmission (available for 1st stages only)
.... 2 sets including one spare

Type: Single suction volute pukp

Capacity? 1.87 m^3/m x 45^m x 30 KW

f) Raw Water Reservoir ... . 1 unit

Capacity: available for three stages

Structure: Reinforced concrete

Dimension: 20.0^m (W) x 27.5^m (L) x 4.0^m (H)

Detension Period: 6.8 hrs. for total capacity in 3rd stage

g) Raw Water Main

Capacity: available for three stages
Material: Asbestos Cement Pipe
Diameter: 250 mm
Length: 1450 m
Flow Velocity: 1.9 m/sec for total capacity in 3rd stage

h) Receiving Well .... 1 unit

Capacity: available for three stages, 9.7 m^3
Structure: Reinforced Concrete

Dimension: 1.5^m (W) x 3.4^m (L) x 1.9^m (H)
Detention Period: 1.7 min. for total capacity in 3rd stage

i) Mixing Basin .... 2 sets

Capacity: available for three stages, 6.75m^3
Structure: Reinforced concrete

Dimension: 1.5^m (W) x 1.5^m (L) x 1.5^m (H)
Detention Period: 1.2 min. for total capacity in 3rd stage
Rapid Mixer: Vertical shaft with 1.5 KW motor

j) Alum Feeding Equipment .... 2 sets

Capacity: available for three stages
Alum solution tank: 1m^3, rubber linning
Agitator: Vertical shaft with 0.75 KW motor, gear drive
Constant Head Tank: 64 liter, rubber linning

k) Lime Feeding Equipment .... 2 sets including one spare

Capacity: available for three stages, 1.4-9.1 kg/hr.
Feeder: dry-feeder (E-1 type)

l) Flocculation Basin .... 2 sets

Capacity per one set: 16.8 m^3, available for 1st stage only
Structure: Reinforced concrete

Dimension: 2.0^m (W) x 4.0^m (L) x 2.1^m (H)
Mixer: Vertical shaft, with 0.75 KW motor.
Detention Time: 20.1 min. for 1st stage

m) Sedimentation Basin .... 2 sets

Capacity per one set: 204 m^3, available for 1st stage only
Structure: Reinforced concrete

Dimension: 4.0^m (W) x 17.0 (L) x 3.0^m (H)
Detention Time: 4.08 hrs. for 1st stage

n) Rapid Sand Filter .... 3 sets

Capacity per one set: 9.18 m^2 x 90 m/d = 800 m^3/d, available for 1st stage only
Structure: Reinforced concrete

Dimension: 2.7^m (W) x 3.4^m (L) x 3.3^m (H)

o) Chlorination .... 1 set

Capacity: available for 1st & 2nd stages
Chlorinator: Vacuum type, 1.88 - 0.09 v.s/h.
Cylinder: 2 - 50 kg chlorine cylinder
Balance: 1 - 50 kg

p) Necessary Pipe & Valves .... 1 set

q) Drainage Pit & Pump .... 1 set

Pit: 4^m (w) x 4^m(L) x 5^m (H)

Pump: 2.0 m^3/min x 10^m x 7.5 KW x 950 rpm

6.2.3 2nd Stage Works

a) Raw Water Pump

(1) For Storage (available for 2nd stage only) .... 1 set

Capacity as well as 1st stage

(2) For Transmission (available for 2nd stage only) .... 1 set

Capacity as well as 1st stage

b) Flocculation Basin .... 2 sets

Capacity as well as 1st stage

c) Sedimentation Basin .... 2 sets

Capacity as well as 1st stage

d) Rapid Sand Filter .... 2 sets

Capacity as well as 1st stage, but rate of filtration is 105 m/d.

e) Necessary Pipes & Valves .... 1 set

6.2.4 3rd Stage Works

a) Raw Water Pump

(1) For Storage (available for 3rd stage only)
.... 1 set

(2) For Transmission (available for 3rd stage only)
.... 1 set

b) Flocculation Basin .... 2 sets

Capacity per one set: 23.1 m^3, available for 3rd stage only

Dimension: 2.2^m (W) x 5.0^m (L) x 2.1^m (H)
Mixer: Vertical shaft with 0.75 KW motor
Detention Time: 20.2 min for 3rd stage

c) Sedimentation Basin .... 2 sets

Capacity per one set: 278 m^3, available for 3rd stage only
Structure: Reinforced concrete
Dimension: 5.0^m (W) x 18.5^m (L) x 3.0^m (H)
Detention Time: 4.04 hrs.

d) Rapid Sand Filter .... 3 sets

Capacity as well as 1st or 2nd stage, but rate of filtration is 119 m/d

e) Chlorination Equipment .... 1 set

Capacity as well as 1st and 2nd stages

f) Necessary Pipes & Valves .... 1 set

Fig 6-8 Location of Intake &. Treatment Plant

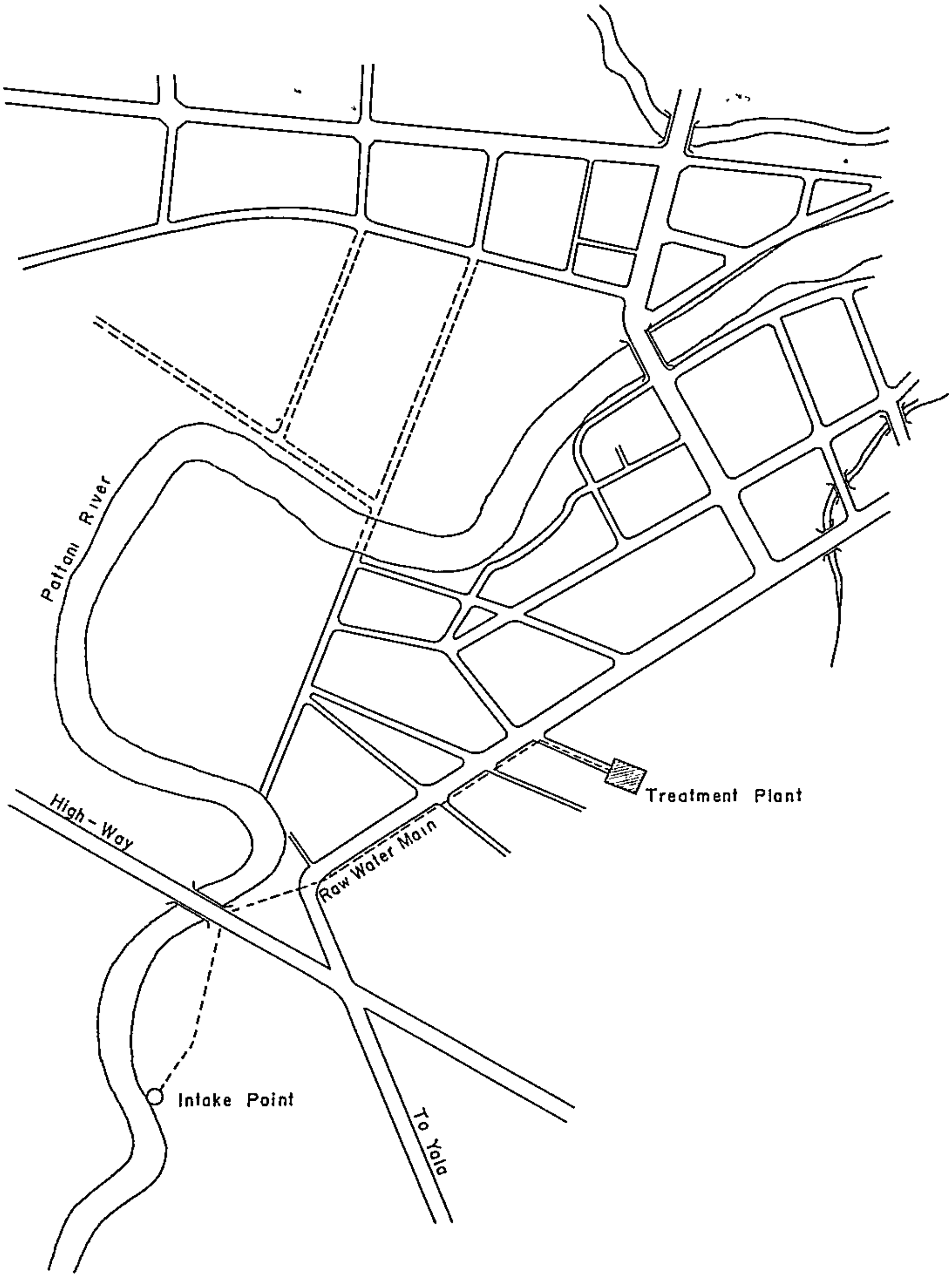


Fig. 6-9 Intake Facility

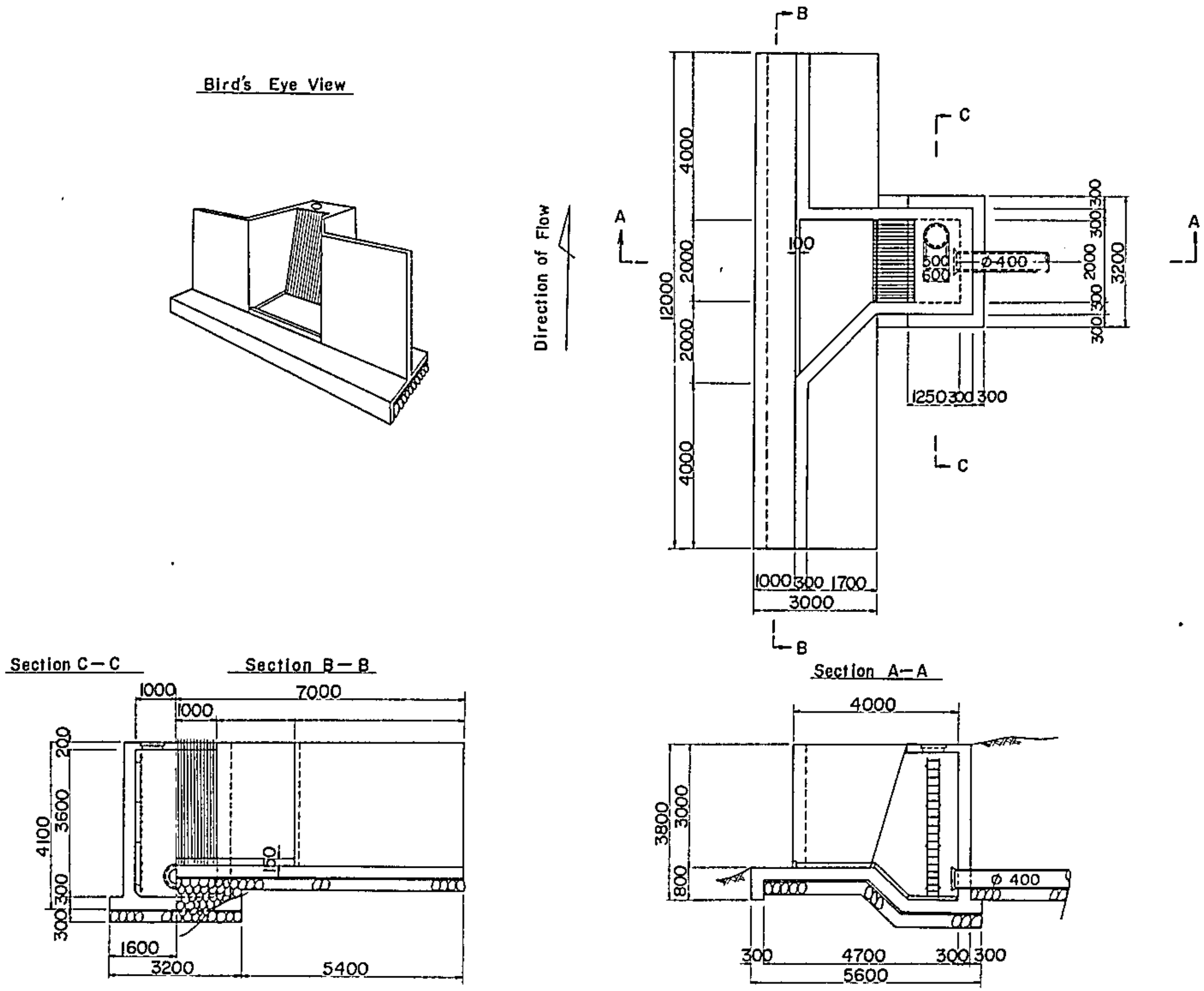


Fig 6-10 Pumping House

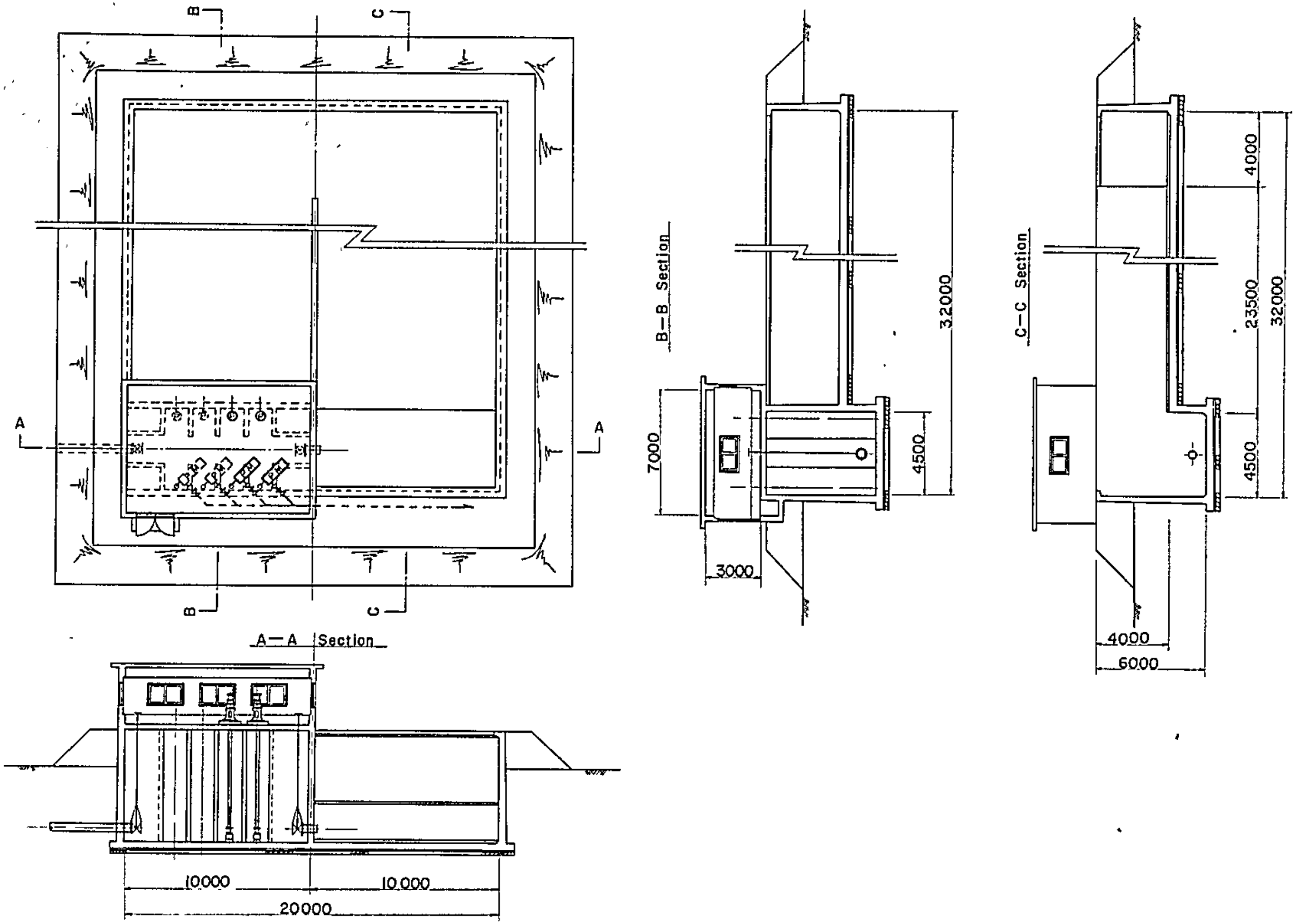


Fig. 6-11 Water Treatment Plant

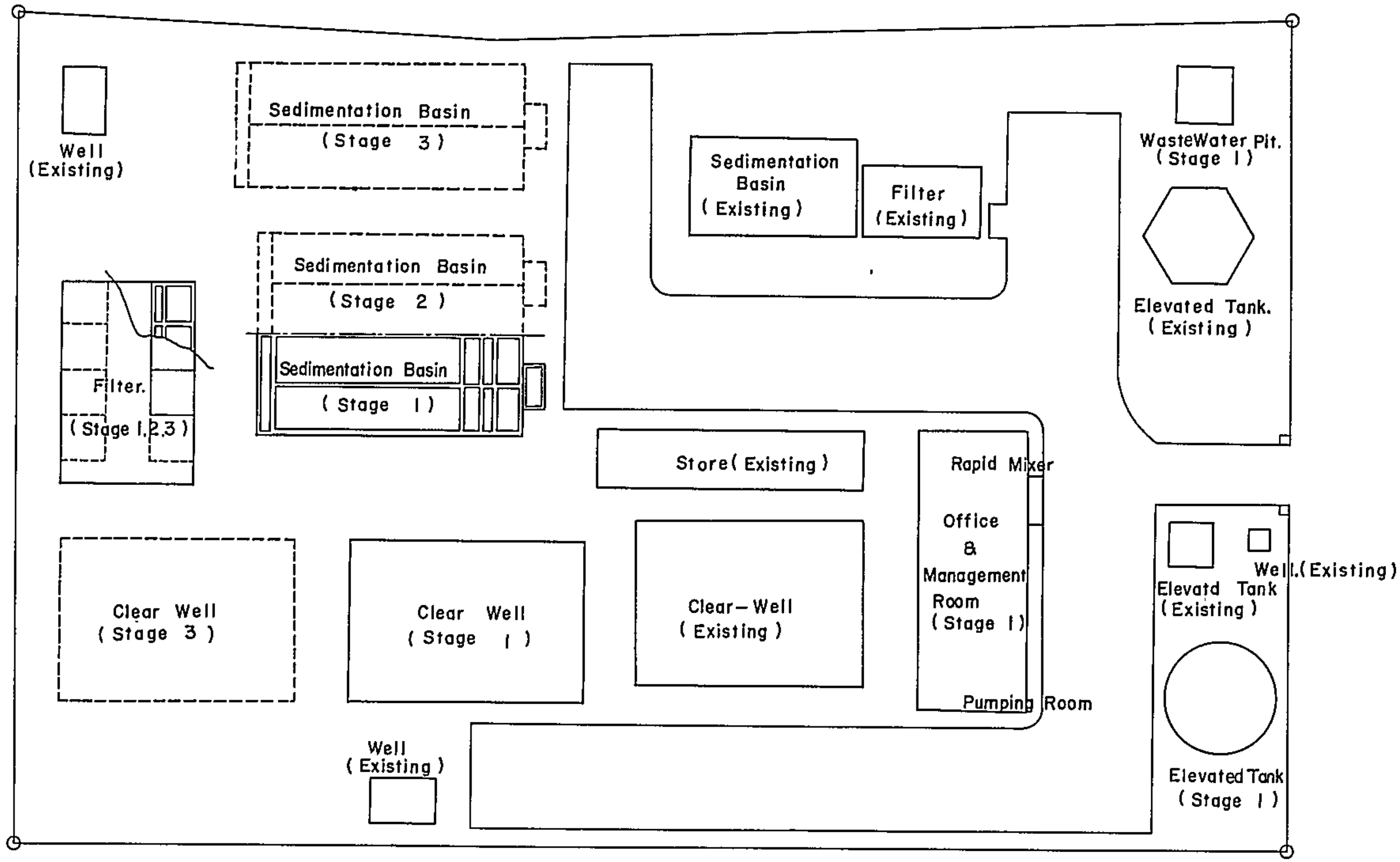


Fig 6-12 Flocculation & Sedimentation

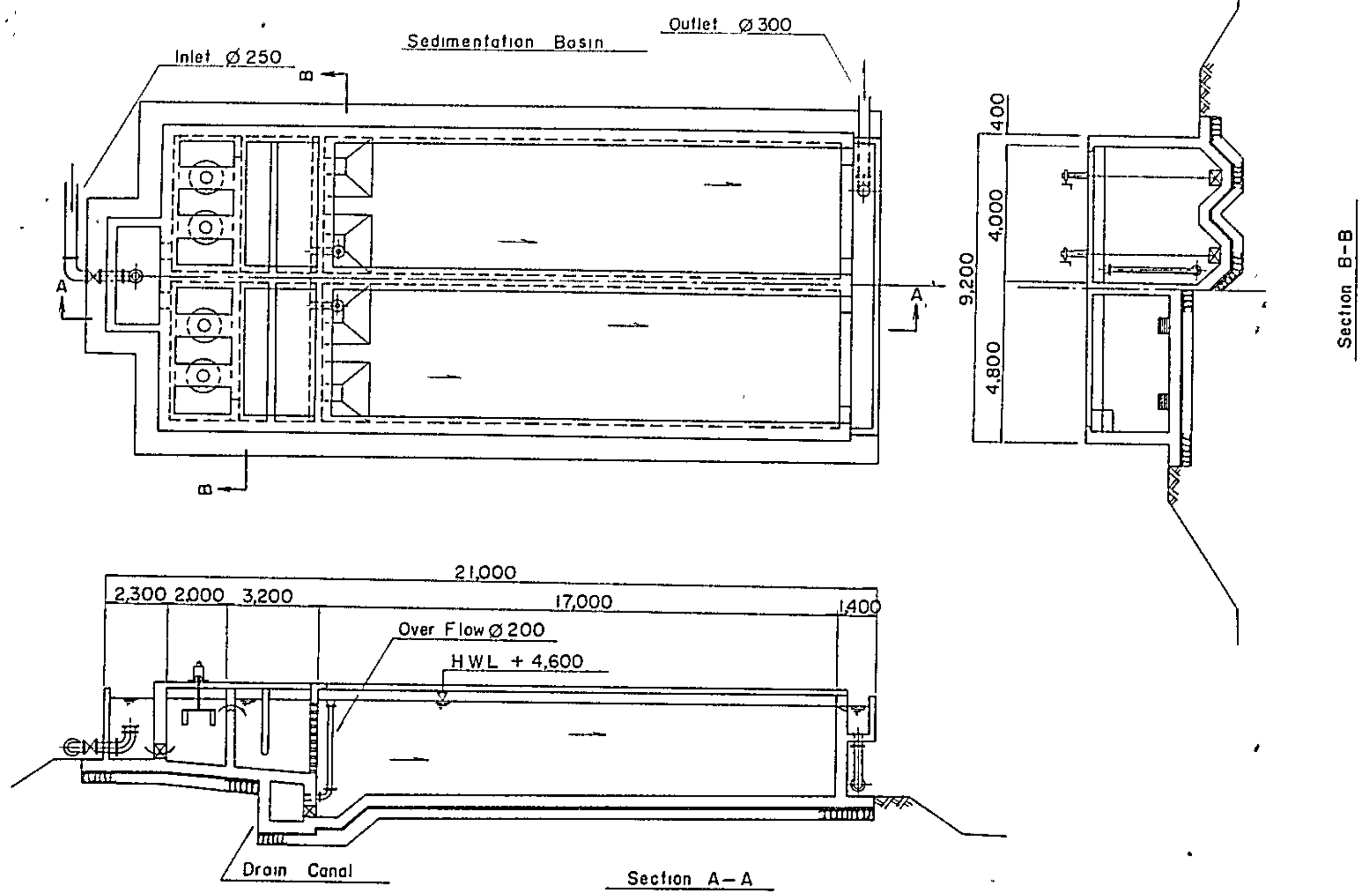


Fig. 6-13 Filtration Basin

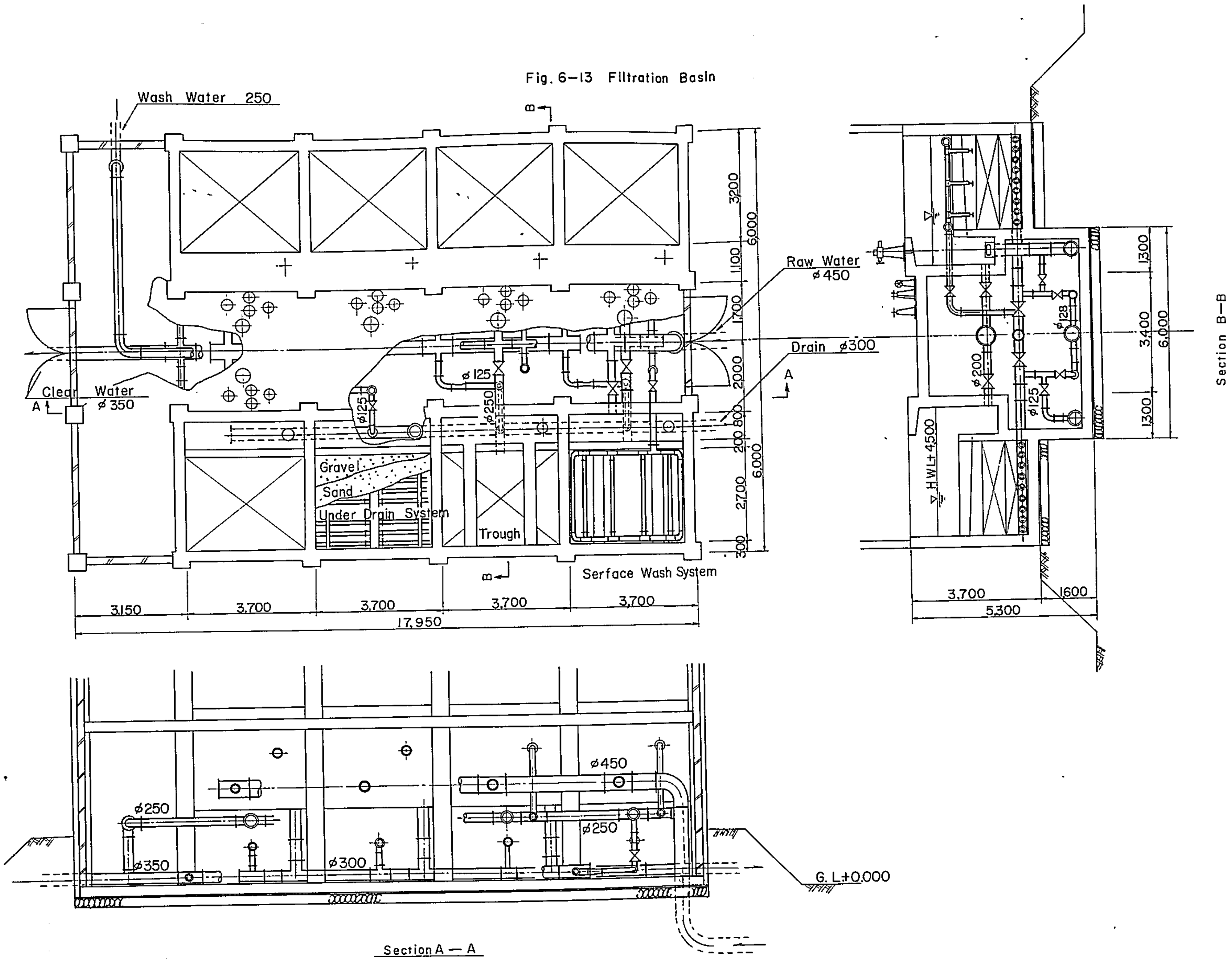
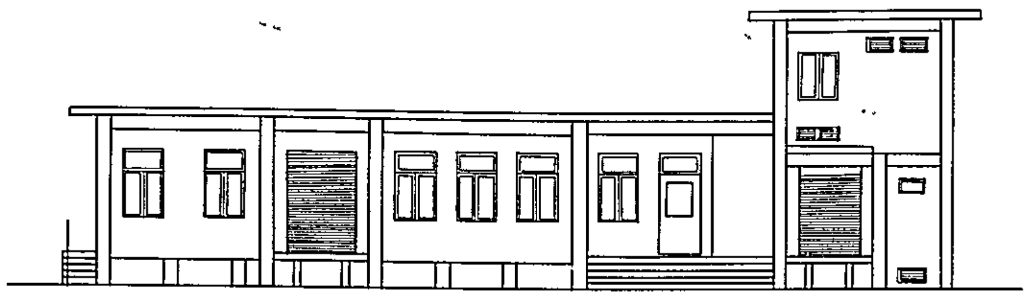
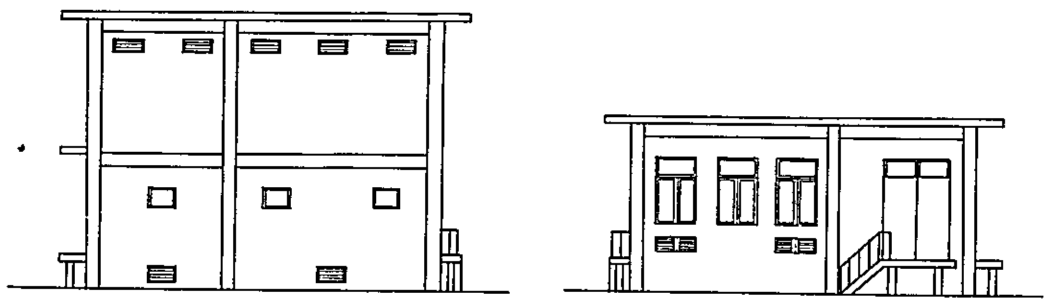


Fig. 6-14 Head House

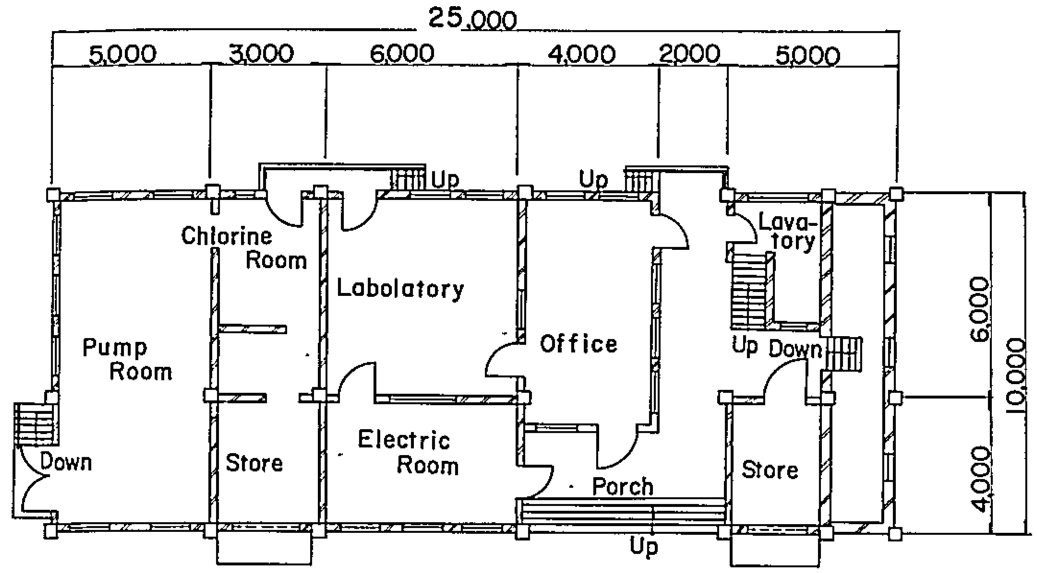


Front Elevation

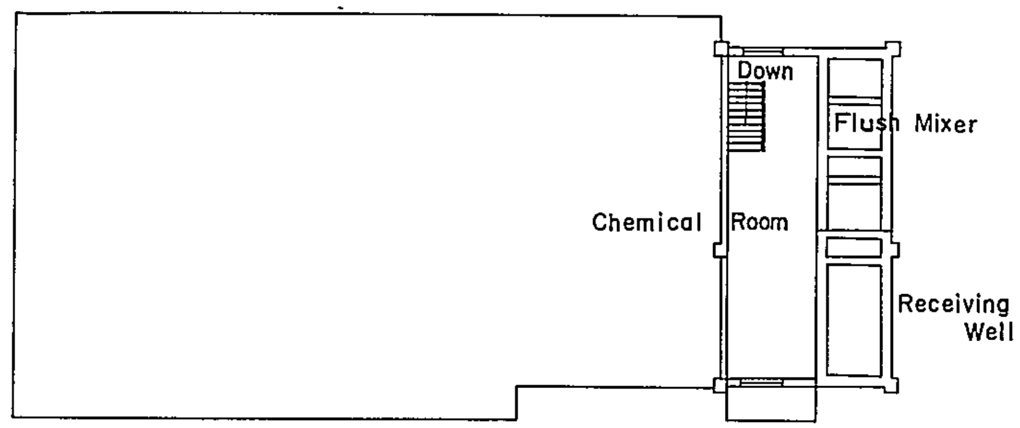
Rear Elevation



Side Elevation



Ground Floor Plan



First Floor Plan

Fig. 6 – 15 Chemical Room

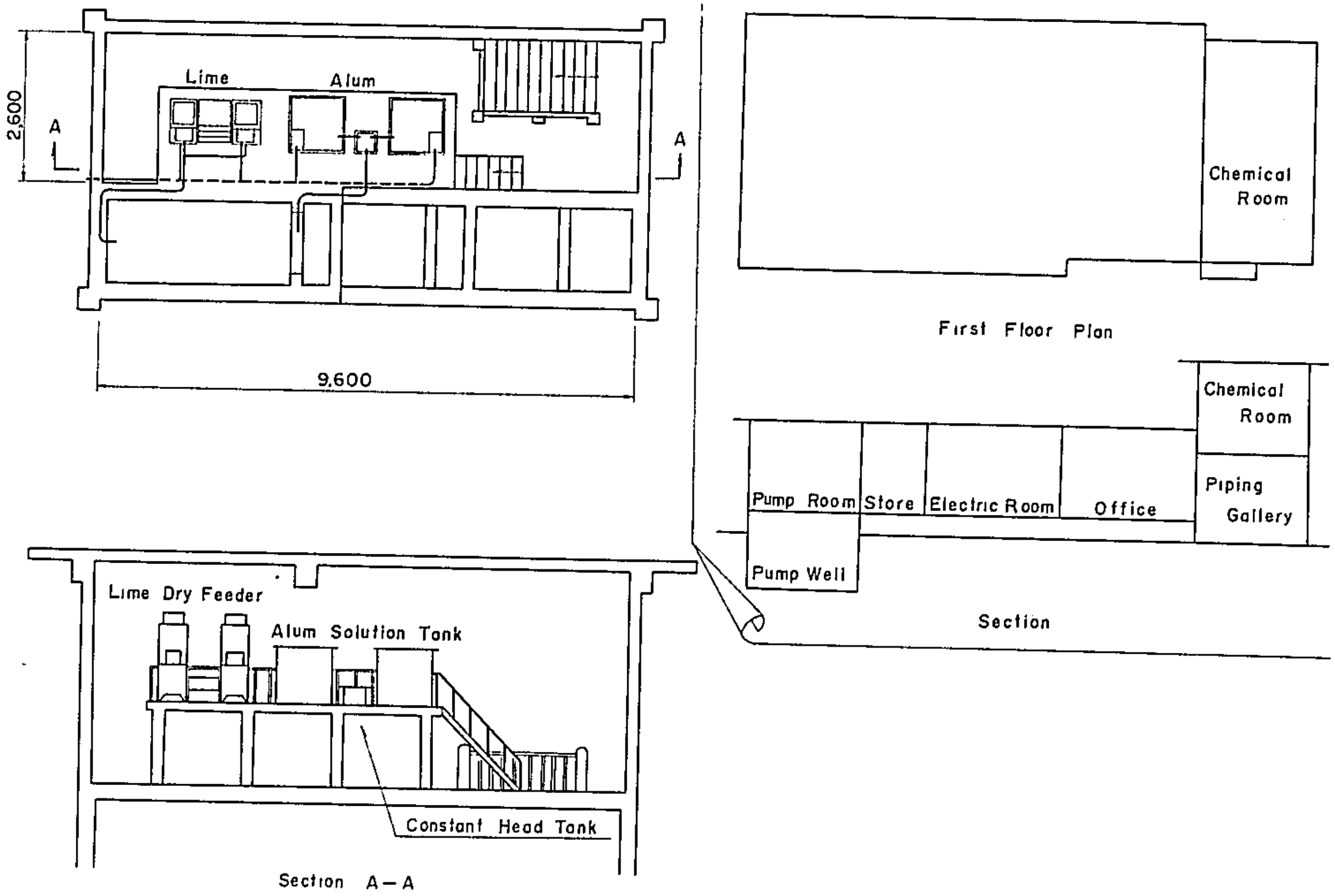


Fig. 6-16 Chlorination Room

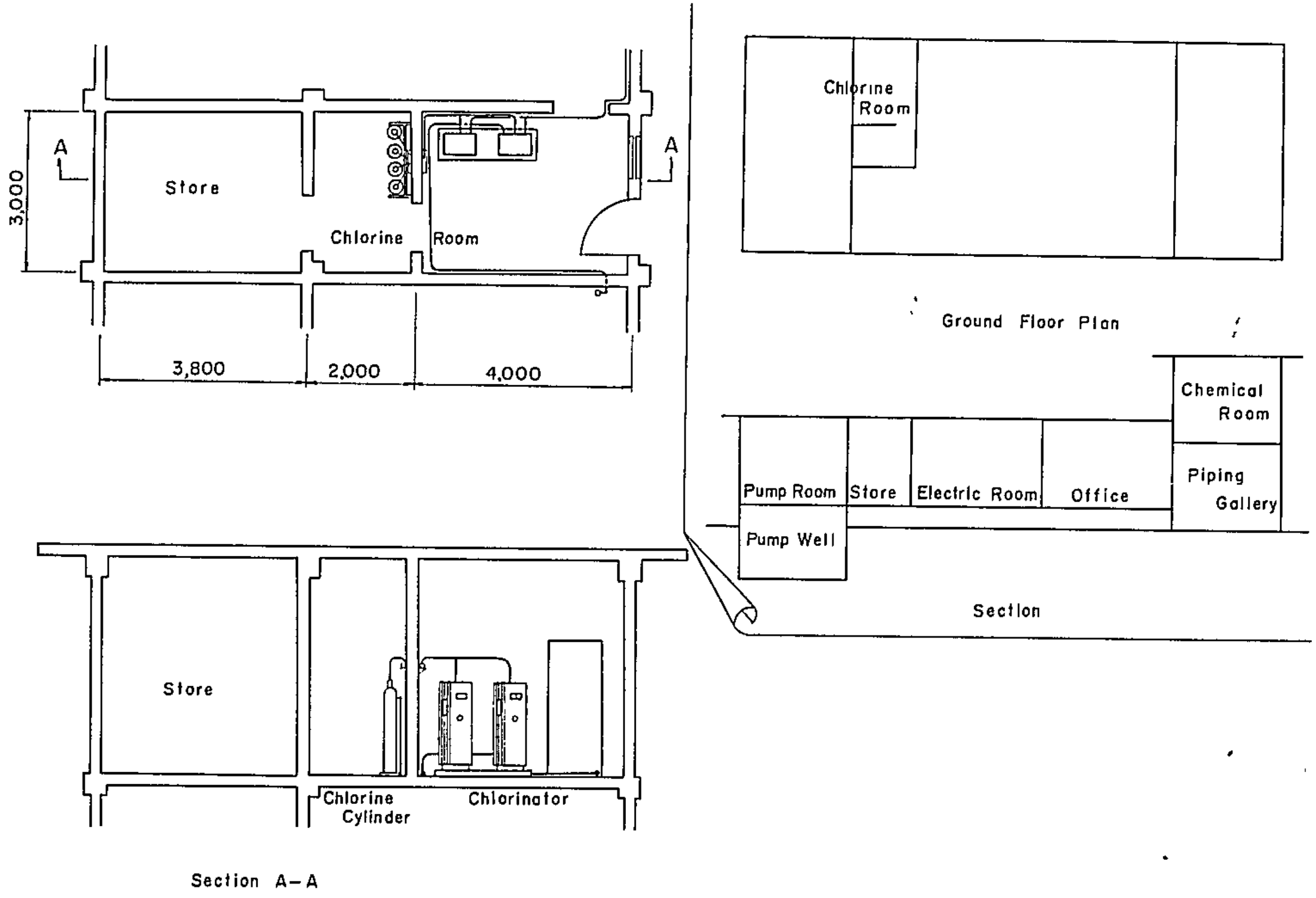
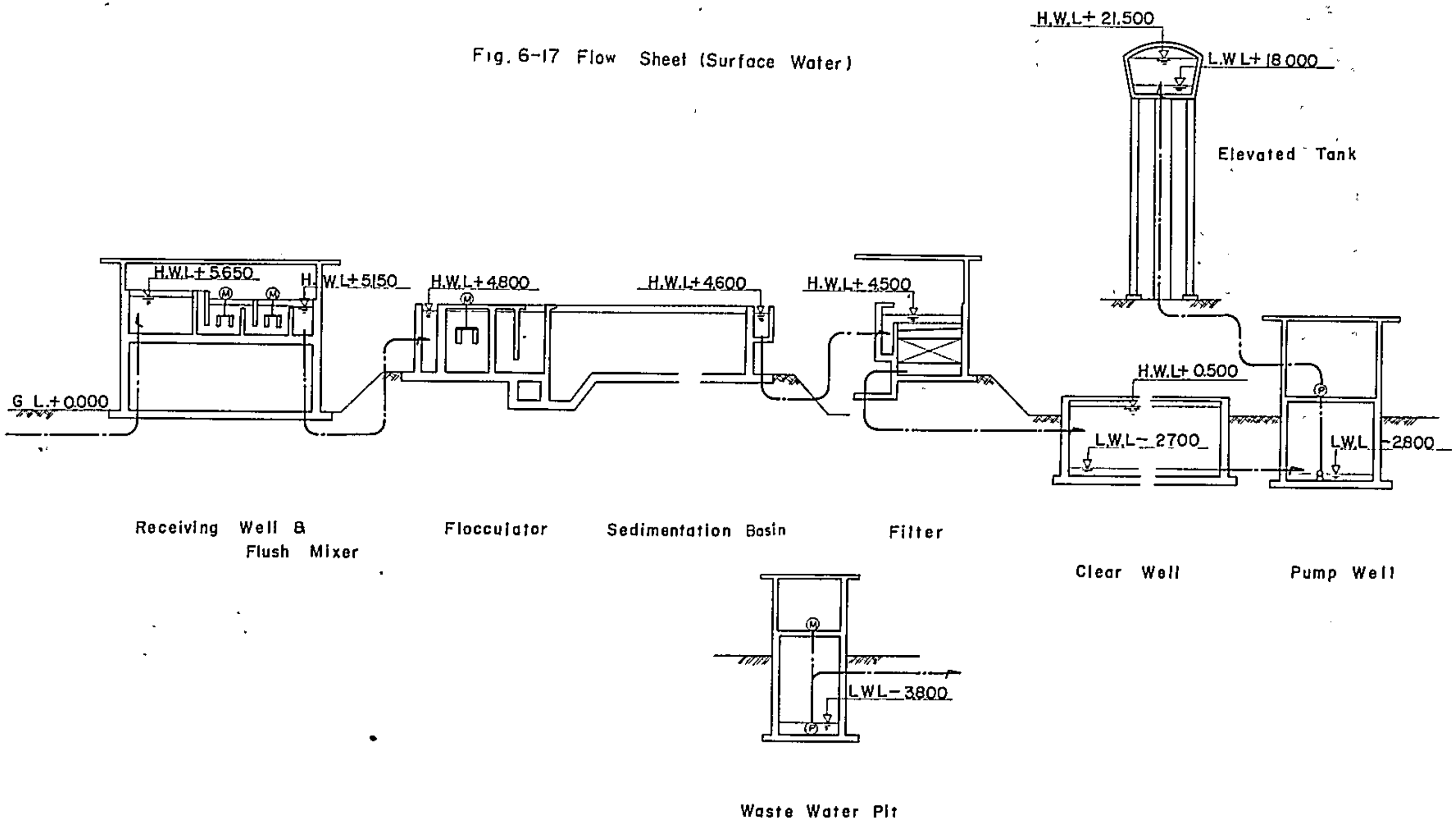


Fig. 6-17 Flow Sheet (Surface Water)



6.3 Distribution System

6.3.1 General

In general, the distribution area is covered by gravity flow from the elevated tanks located respectively in No. 1 and No. 2 distribution network. No. 1 distribution network is composed in right bank of Pattani River in comparison with No. 2 distribution network in left bank of the river.

There is different way between 1st and 2nd stage of extension project as far as distributing method from the elevated tank is concerned. Namely, at 1st stage, any water needed in all distribution system can be covered by an elevated tank located in No. 1 distribution network, but at 2nd stage another elevated tank is essential to built in No. 2 distribution network.

Table 6.5 Summary of Extension of Distribution System

Item	1st Stage	2nd Stage	3rd Stage
Clear Well	1,000 m^3 - 1 unit	—	1,000 m^3 - 1 unit
Pump No. 1	22 KW - 3 sets	22 KW - 1 set	22 KW - 1 set
" No. 2	—	18.5 KW - 3 sets	18.5 KW - 1 set
Elevated Tank " No. 1	120 m^3 - 1 unit	—	—
" No. 2	—	120 m^3 - 1 unit	120 m^3 - 1 unit
Pipe No. 1	ϕ500-ϕ100: 5,800 m	ϕ150-ϕ100: 6,580 m	—
" No. 2	ϕ300-ϕ100: 8,400 m	ϕ150-ϕ100: 12,000 m	—

6.3.2 Clear Well (reservoir)

Any waters filtered are kept in clear wells which are built in water treatment plant as follows in addition to existing one (1,000 m^3)

1st stage: 15^m (W) x 21^m (L) x 3.2^m (D), 1,000 m^3 - 1 unit
2nd stage: none
3rd stage: as well as 1st stage, 1,000 m^3 - 1 unit

The purpose of reservoirs is to adjust the difference between the discharge capacity relying on equipment, that is normally set on the basis of a maximum daily demand and a maximum hourly demand, and store water for emergency use, for example, water for extingwishing a five. If the capacity of a reservoir is insufficient, the relation between water production and demand will be unbalanced causing water shourtage problems. The detention time of reservoir of 8 - 9 hours is a value computed empirically as

2,000 (including existing 1,000 m^3) x $\frac{24}{5,120}$ = 8 - 9 hrs. at 1st stage.

6.3.3 Pumps

Any pumps shall be used to pump up the filtered water inclear well to elevated tank in either No. 1 or No. 2 distribution network, based on following demand of pumping up in hourly maximum.

Table 6.6 Demand of Pumping up in hourly maximum

	Total Area	No. 2 distribution net-work
1st Stage	320 m^3/h (= 5.4 m^3/min.)	——
2nd Stage	477 m^3/h (= 8.0 m^3/min.)	225 m^3/hr (= 3.8 m^3/min.)
3rd Stage	681 m^3/h (= 11.4 m^3/min.)	313 m^3/hr (= 5.2 m^3/min.)

Selection of Pump

a) No. 1 distribution network

Type: Volute Pump (Single Suction)
Capacity: 2.83 m^3/min x 30 m x 2950 rpm x 22 KW
Number: Stage 1 .... 3 sets (Including 1 spare)
Stage 2 .... 1 set
Stage 3 .... 1 set

b) No. 2 distribution network

Type: Volute Pump (Single Suction)
Capacity: 1.90 m^3/min. x 30 m x 2950 rpm x 18.5 KW
Number: Stage 1 .... None
Stage 2 .... 3 sets (Including 1 spare)
Stage 3 .... 1 set

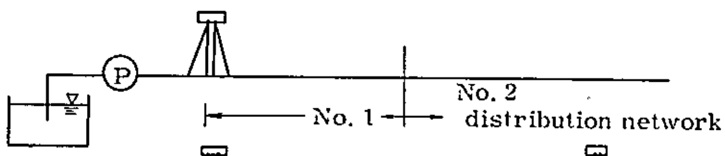
6.3.4 Elevated Tank

In addition to existing elevated tank (120 m^3), other elevated tank is proposed in 1st stage at water treatment plant as following capacity.

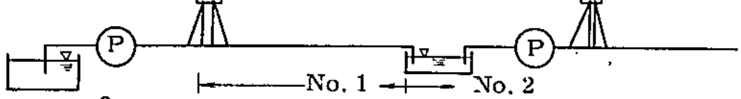
Capacity: 120 m^3 - 1 unit (1st stage)
Structure: Reinforced concrete
Dimension: dia 7.0^m x height 23.5^m
Attachement: inlet pipe 300 mm
outlet pipe 350 mm
drain pipe 100 mm
overflow pipe 150 mm

The elevated tanks including existing one can cover the overall water demand throughout No. 1 and No. 2 distribution networks in 1st stage, but depending upon increasing water demand in 2nd stage the two elevated tanks are not enough to keep water pressure as well as in 1st stage in No. 2 distribution network. There, another elevated tank is installed in No. 2 distribution network to cover the water demand in 2nd stage. That is to say,

1st stage



2nd stage



120 m^3 - 1 unit at municipal office in No. 2 distribution network

3rd stage; 120 m^3 - 1 unit at same location

Existing small elevated tank (50 m^3) is used for only backwashing and other miscellaneous uses in water treatment plant after the completion of 1st stage.

6.3 5 Distribution network

Any distribution networks are designed on the base of 3rd stage's water demand at 2,000 AD. 1st stage, main pipes are laid in No 1 and No. 2 distribution networks in addition to existing network, but smaller pipes are branched from the main pipes in 2nd stage.

Hydraulic study has been made as follows.

No. 1 Distribution Network:

	(0)	200^{m}	(1)	280^{m}	(15)	380^{m}	(14)	230^{m}	(24)	170^{m}	(23)	340^{m}	(22)
		φ500		φ400		φ400		φ250		φ250		φ200	
Q(1/sec)		189.09		163.09		159.66		31.49		26.51		21.53	
I (%)		2.6		6.4		6.0		3.3		6.4		4.4	
h(m)		0.52		1.79		2.28		0.65		0.36		1.36	
G. L(m)	3,491		3,341		2,590		2,891		2,501		2,723		2,327
P/w(m)	21,491		20,971		19,181		16,901		16,251		15,891		14,531
H(m)	18,000		17,630		16,591		14,010		13,750		13,168		12,204

	(22)	180^{m}	(21)	400^{m}	( )
		φ200		φ150	
Q(1/sec)		16.55		3.62	
I (%)		2.7		0.66	
h(m)		0.45		0.26	
G. L(m)	2,327		2,707		2,448
P/w(m)	14,531		14,081		13,821
H(m)	12,204		11,374		11,373

	(21)	230^{m}	(20)	120^{m}	(19)	180^{m}	( )
		φ200		φ150		φ100	
Q(1/sec)		12.33		7.25		3.62	
I (%)		1.6		2.5		4.8	
h(m)		0.37		0.30		0.86	
G. L(m)	2,707		3,188		2,898		2,538
P/w(m)	14,081		13,711		13,411		12,551
H(m)	11,374		10,523		10,513		10,013

No. 2 Distribution Network:

	(0)	50^m	(26)	100^m	(27)	160^m	(28)	180^m	(42)	220^m	(41)	300^m	(40)
		ϕ300		ϕ250		ϕ250		ϕ200		ϕ200		ϕ200	
Q(1/sec)		86.17		45.97		43.37		26.47		23.87		20.45	
I (%)		8.0		6.0		5.4		6.4		5.2		4.0	
h(m)		0.40		0.60		0.86		1.15		1.14		1.20	
G. L(m)	3,100		3,211		3,138		3,590		3,379		3,122		3,759
P/w(m)	21,100		20,700		20,100		19,240		18,090		16,950		15,750
H(m)	18,000		17,489		16,962		15,630		14,711		13,828		11,991

	(40)	440^m	(39)	500^m	( )	(28)	250^m	(29)	180^m	(30)	1800^m	( )
		ϕ200		ϕ200			ϕ150		ϕ150		ϕ150	
Q(1/sec)		17.03		13.89			13.43		8.23		5.90	
I (%)		2.8		1.9			7.5		3.1		1.65	
h(m)		1.23		0.95			1.87		0.56		2.97	
G. L(m)	3,759		3,442		3,323	3,590		3,650		3,628		2,034
P/w(m)	15,750		14,520		13,570	19,240		17,370		16,810		13,840
H(m)	11,991		11,078		10,247	15,650		13,718		13,182		11,806

Consequently, the size and length of pipes is as follows.

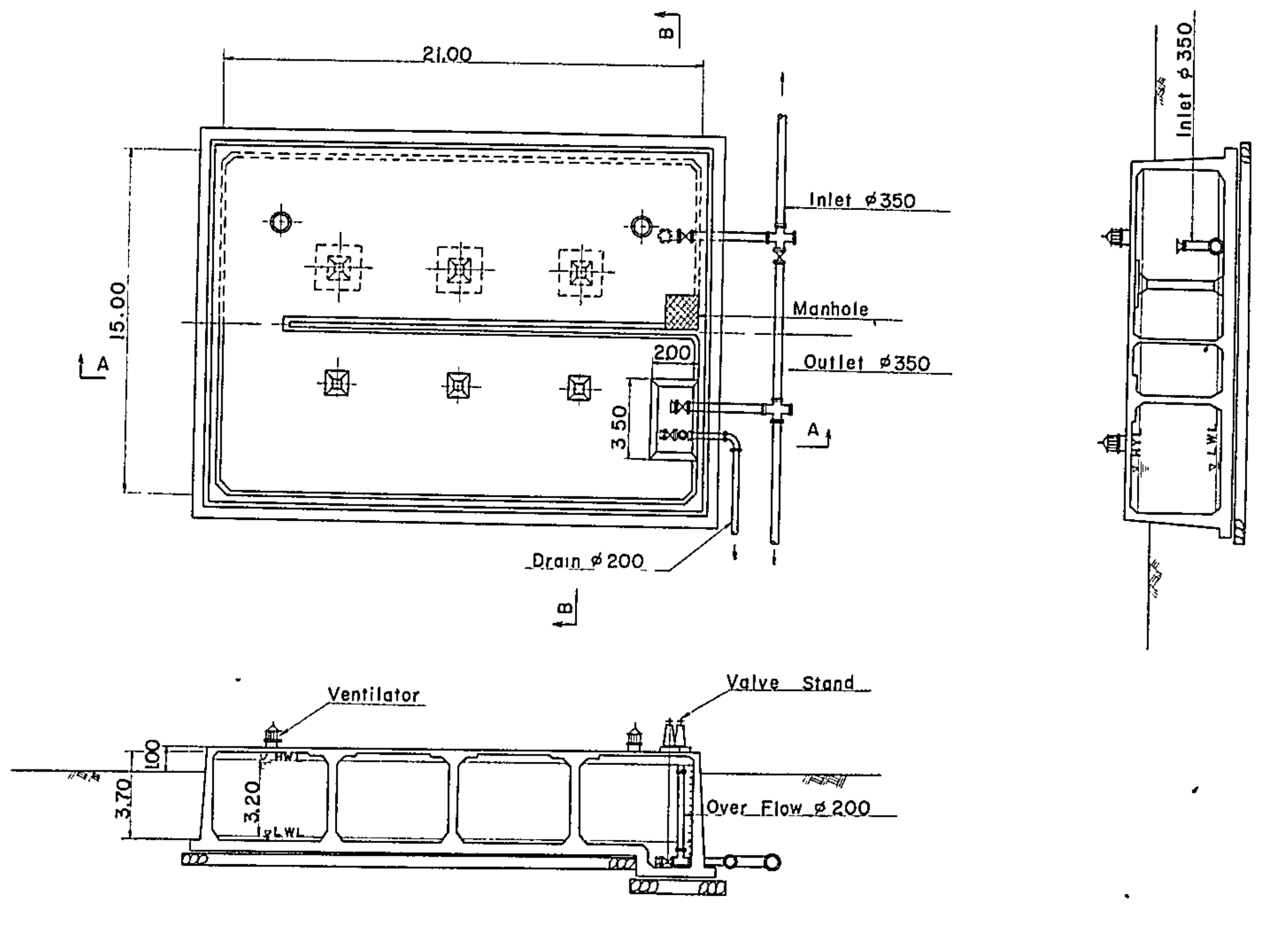
1st stage; total length 14,200 m

Dia. (mm)	No. 1 Network$^{(m)}$	No. 2 Network$^{(m)}$	Total (m)
500	200	-	200
400	1,010	-	1,010
300	140	400	540
250	400	310	710
200	1,500	3,500	5,000
150	1,830	2,970	4,800
100	720	1,170	1,890
Total	5,800	8,350	14,150

2nd stage; total length 12,000$^{(m)}$

Dia. (mm)	No. 1 Network (m)	No. 2 Network$^{(m)}$	Total (m)
150	2,140	480	2,620
100	4,440	4,940	9,380
Total	6,580	5,420	12,000

Fig. 6-18 Clear Well



Section A—A

Section B—B

Fig. 6-19 Pumping Room

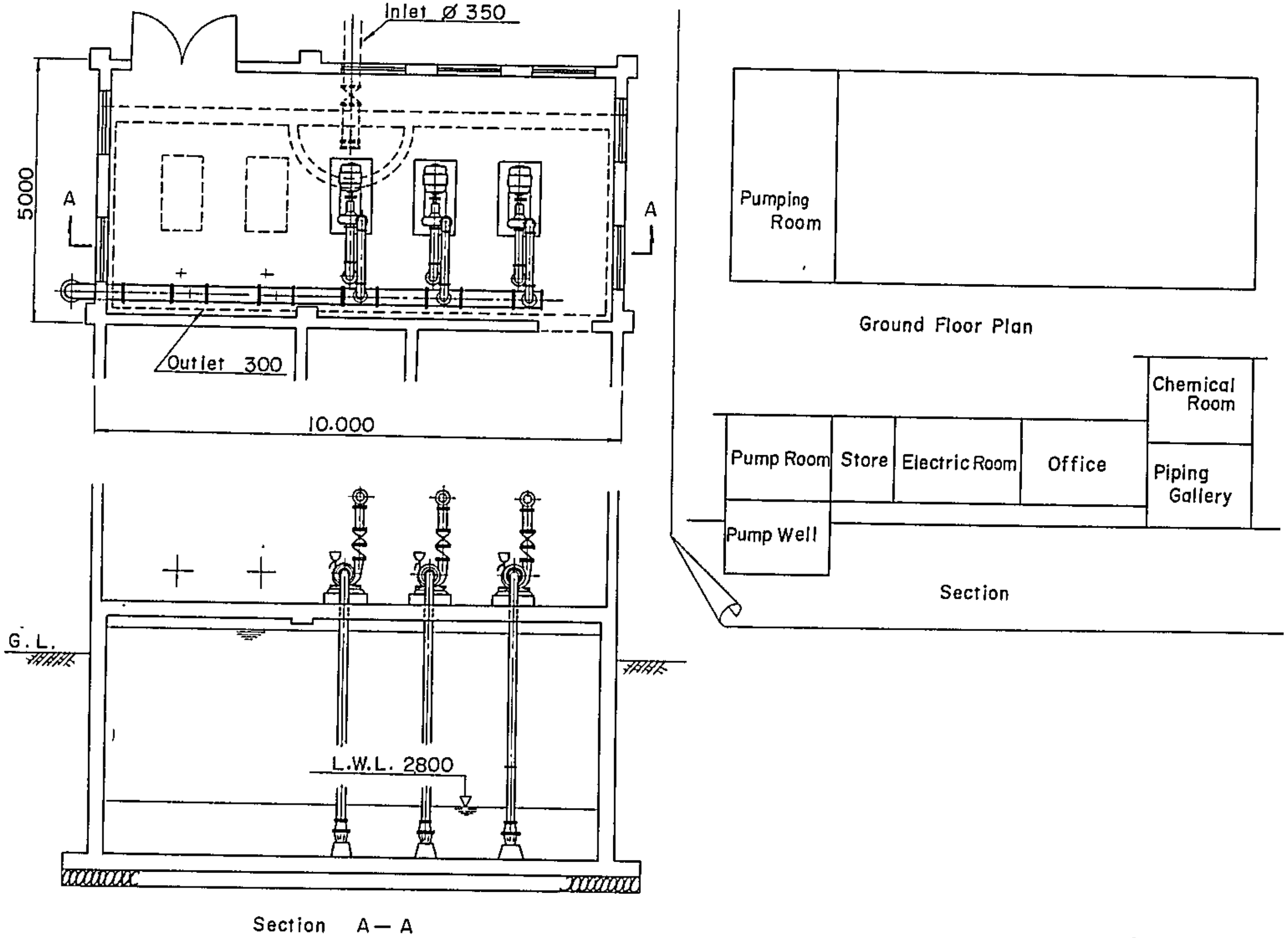


Fig.6-20 Elevated Tank

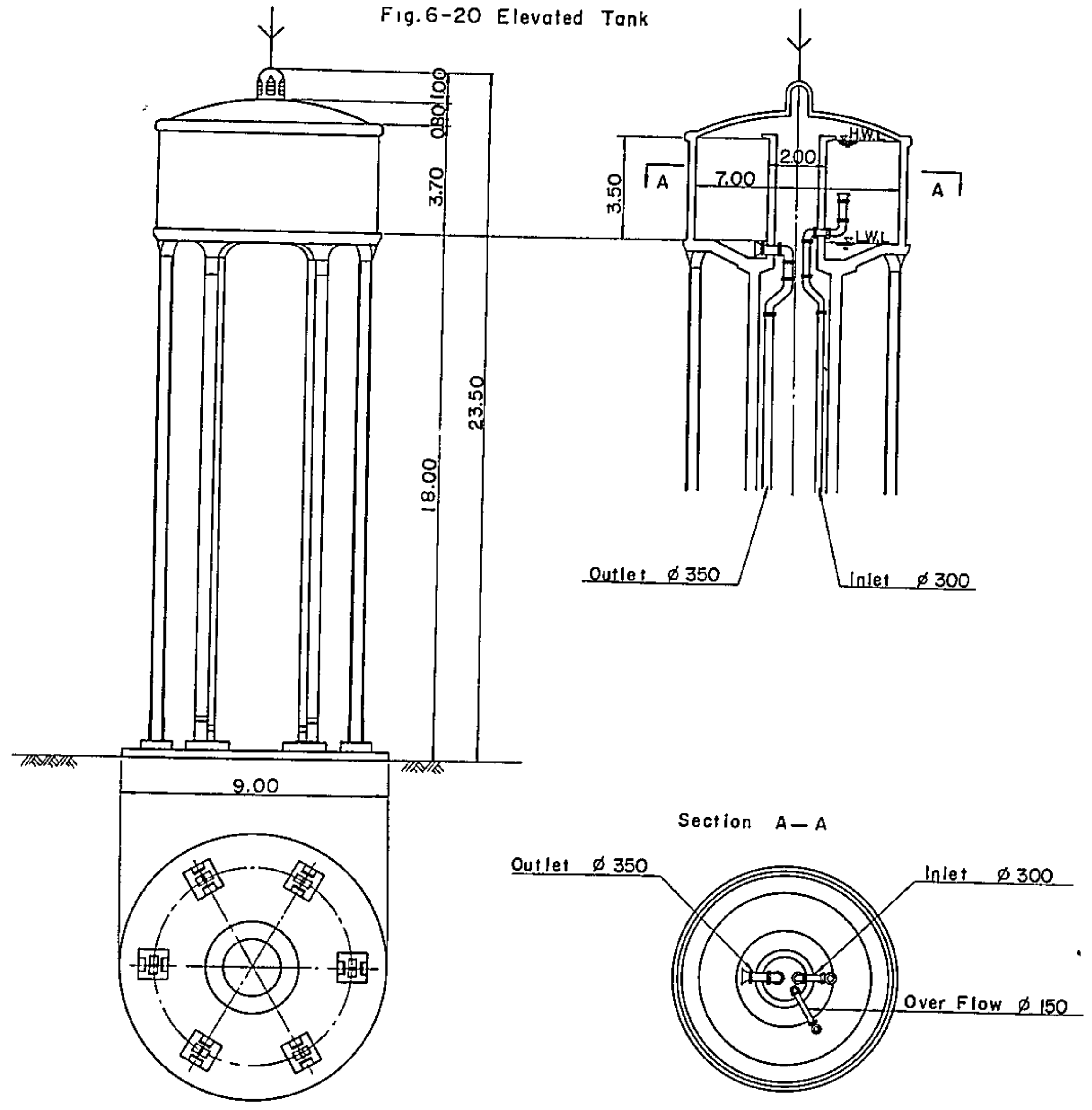
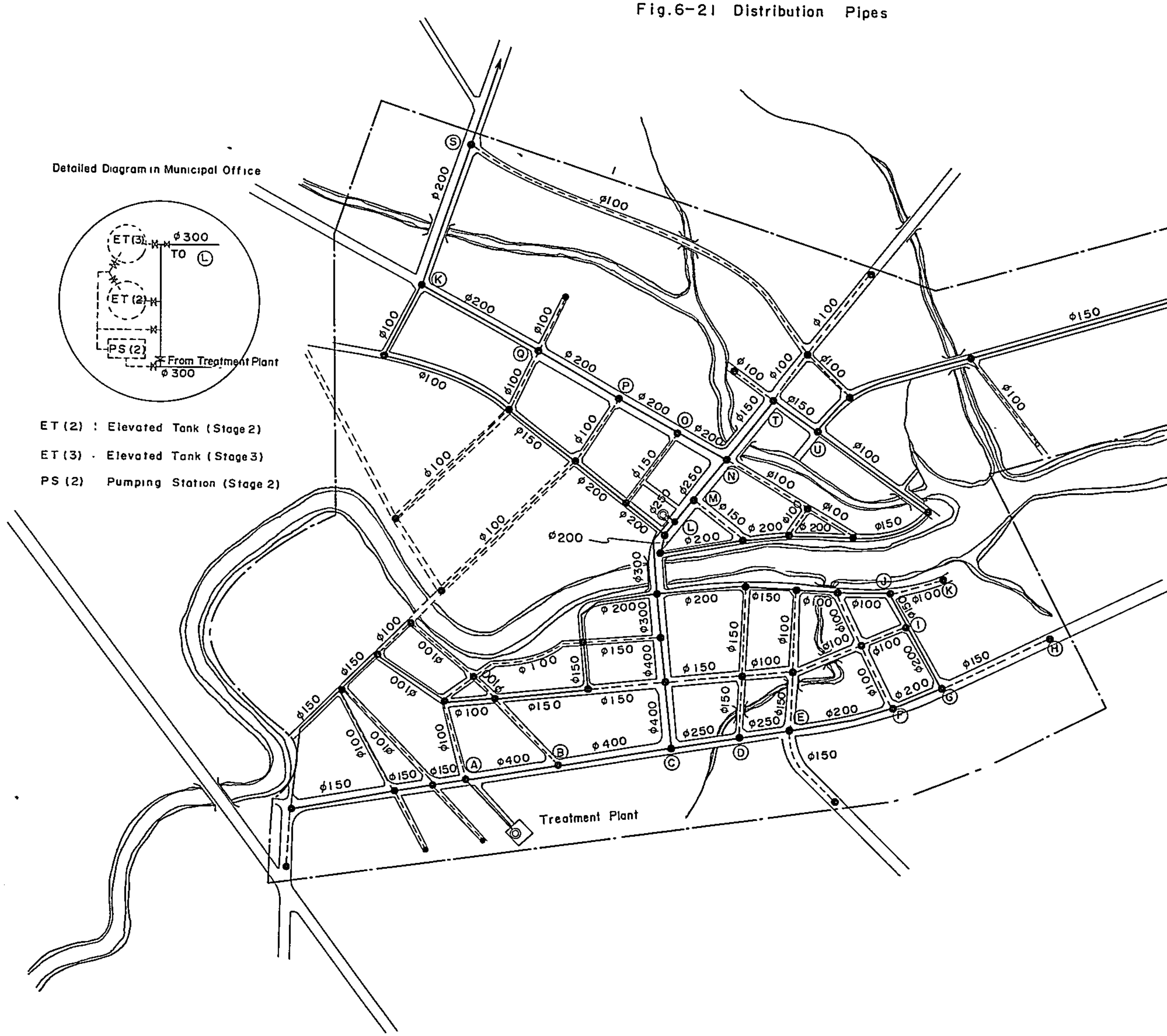


Fig.6-21 Distribution Pipes



Remarks

(1) ●——● Distribution Main (1st-Stage)

●----● Distribution Branch (2nd Stage)

(2) All pipes shall be asbest cement pipe without special application such as river-cross

Fig. 6-22 Main Pipe Line (1)

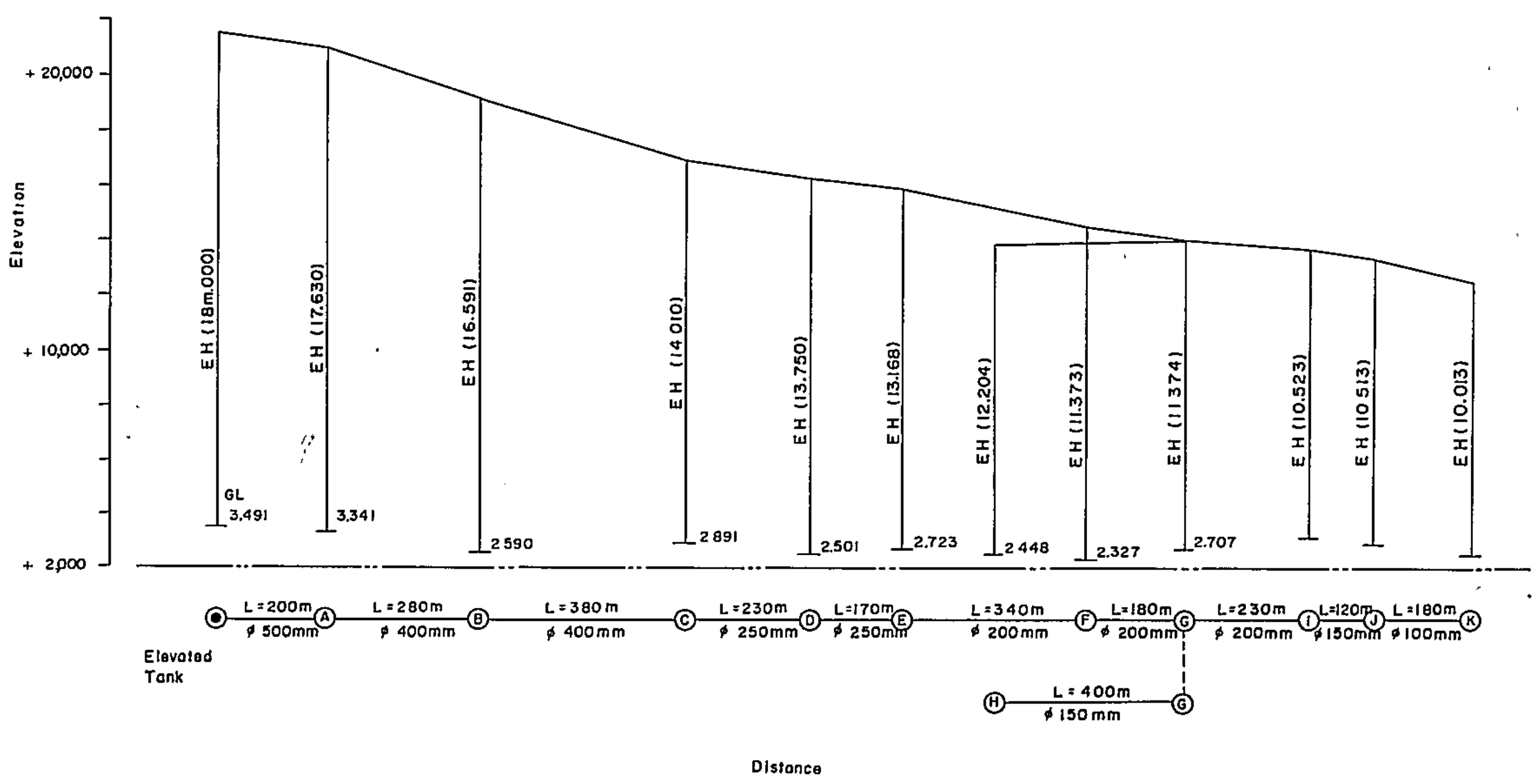
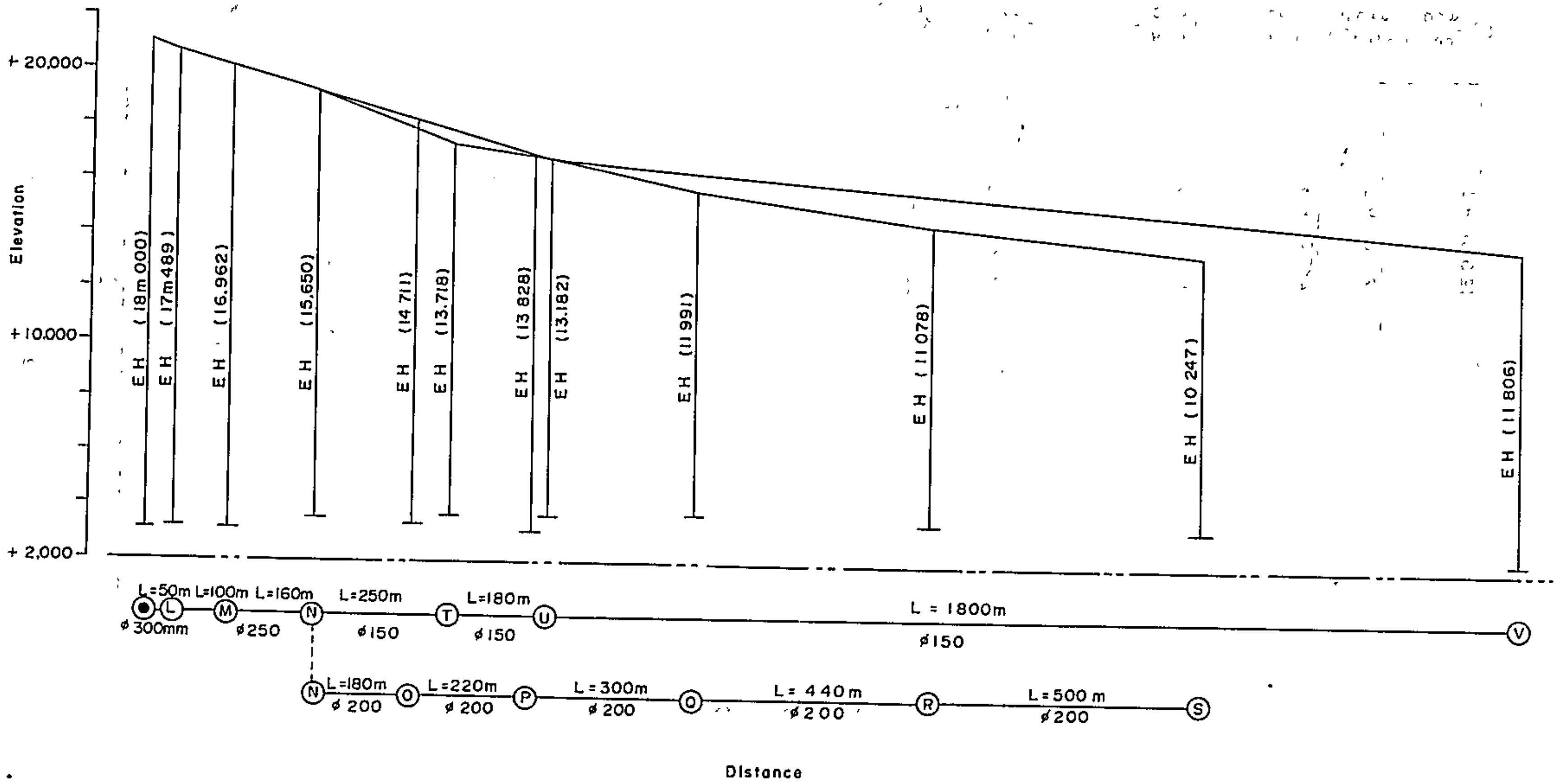


Fig.6-23 Main Pipe Line (2)



Chapter 7 Cost of Construction & Maintenance

7.1 Breakdown of Construction Cost (1st Stage)

I. Underground Water:

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Deep Well Intake				
1-1 Land Cost	432 m^2	9,000	0	9,000
1-2 Deep Well	2 sets	460,000	220,000	240,000
1-3 Pump	2 units	216,000	196,000	20,000
1-4 Pump House	18 m^2	54,000	0	54,000
1-5 Miscellaneous	(5.5%)	41,000	23,000	18,000
Sub-Total		780,000	439,000	341,000
2) Raw Water Main				
2-1 ACP ϕ150 mm	1,500 m	198,000	0	198,000
2-2 Expenditure	≈ 20%	40,000	0	40,000
2-3 Pavement	1,500 m^2	68,000	0	68,000
2-4 Miscellaneous	(4.6%)	14,000	0	14,000
Sub-Total		320,000	0	320,000
3) Water Treatment Plant				
3-1 Land Cost	680 m^2	14,000	0	14,000
3-2 Receiving Well	1 set	98,000	29,000	69,000
3-3 Mixing Basin	2 sets	107,000	26,000	81,000
3-4 Head House	280 m^2	840,000	0	840,000
3-5 Coagulation & Filtration Equip.	22.2 m^2	737,000	660,000	77,000
3-6 Alum Feeding Equip.	1 set	148,000	133,000	15,000
3-7 Lime Feeding Equip.	1 set	120,000	110,000	10,000
3-8 Chlorination	1 set	130,000	120,000	10,000
3-9 Drainage	1 set	44,000	0	44,000
3-10 Pipe in Plant	1 set	604,000	549,000	55,000
3-11 Miscellaneous	(5.55%)	158,000	90,000	68,000
Sub-Total		3,000,000	1,717,000	1,283,000
4) Distribution System				
4-1 Clear Well	1,000 m^3	1,242,000	62,000	1,180,000
4-2 Pump	3 sets	330,000	330,000	0
4-3 Elevated Tank	120 m^3	462,000	30,000	432,000
4-4 Distribution Pipe				
MDCIP ϕ500	200 m	246,000	224,000	22,000
〃 ϕ400	1,010 m	868,600	797,900	70,700
ACP ϕ300	540 m	138,672	0	138,672
〃 ϕ250	710 m	145,905	0	145,905
〃 ϕ200	5,050 m	884,760	0	884,760
〃 ϕ150	4,800 m	504,480	0	504,480
〃 ϕ100	1,890 m	147,042	0	147,042

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
4-5 Valve ϕ500	1	34, 700	34, 700	0
" ϕ400	4	85, 530	85, 530	0
" ϕ300	4	43, 296	43, 296	0
" ϕ250	6	48, 834	48, 834	0
" ϕ200	19	108, 965	108, 965	0
" ϕ150	31	118, 141	118, 141	0
" ϕ100	42	124, 236	124, 236	0
4-6 Hydrant-Double	9	53, 348	53, 348	0
" Single	64	319, 360	319, 360	0
4-7 Pavement	14, 200 m^2	639, 000	0	639, 000
4-8 Expenditure (20% of local)	1	382, 000	0	382, 000
4-9 Miscellaneous	(4. 8%)	333, 131	98, 690	234, 441
Sub-Total		7, 260, 000	2, 479, 000	4, 781, 000
5) Electric Equipment		1, 500, 000	1, 000, 000	500, 000
Sub-Total		1, 500, 000	1, 000, 000	500, 000
Grand - Total		12, 860, 000	5, 635, 000	7, 225, 000
Engineering Fee	5%	640, 000	640, 000	0
Administration Cost	4%	510, 000	0	510, 000
Reserve	10%	1, 290, 000	525, 000	765, 000
Total		15, 300, 000	6, 800, 000	8, 500, 000

II Surface Water:

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Intake Facilities				
1-1 Land Cost	2, 500 m^2	50, 000	0	50, 000
1-2 Intake	1 unit	109, 000	0	109, 000
1-3 Raw Water Pipe ϕ400	10 m	53, 000	51, 000	2, 000
1-4 Pumping Well	1 unit	300, 000	0	300, 000
1-5 Pumping House	1 unit	210, 000	0	210, 000
1-6 Storage Tank	1 unit	1, 496, 000	0	1, 496, 000
1-7 Pump	4 sets	440, 000	400, 000	40, 000
1-8 Miscellaneous	(5. 34%)	142, 000	24, 000	118, 000
Sub-Total		2, 800, 000	475, 000	2, 325, 000
2) Raw Water Main				
2-1 ACP ϕ250 mm	1, 450 m	295, 000	0	295, 000
2-2 Expenditure	= 20%	59, 000	0	59, 000
2-3 Pavement	1, 450 m^2	66, 000	0	66, 000
2-4 Miscellaneous	(4. 76%)	20, 000	0	20, 000
Sub-Total		440, 000	0	440, 000

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
3) Water Treatment Plant				
3-1 Land Cost	1, 500 m^2	30, 000	0	30, 000
3-2 Receiving Well	1 set	98, 000	29, 000	69, 000
3-3 Mixing Basin	2 sets	107, 000	26, 000	81, 000
3-4 Head House	280 m^2	840, 000	0	840, 000
3-5 Sedimentation	2 sets	996, 000	92, 000	904, 000
3-6 Filtration	3 sets	1, 411, 000	495, 000	916, 000
3-7 Alum Feeding	1 set	148, 000	133, 000	15, 000
3-8 Lime Feeding	1 set	120, 000	110, 000	10, 000
3-9 Chlorination	1 set	130, 000	120, 000	10, 000
3-10 Drainage	1 set	97, 000	0	97, 000
3-11 Pipe in plant	1 set	678, 000	618, 000	60, 000
3-12 Miscellaneous	(5. 26%)	245, 000	85, 000	160, 000
Sub-Total		4, 900, 000	1, 708, 000	3, 192, 000
4) Distribution System as well as underground water	1	7, 260, 000	2, 479, 000	4, 781, 000
Sub-Total		7, 260, 000	2, 479, 000	4, 781, 000
5) Electric Equipment & Instrument as well as underground	1	1, 500, 000	1, 000, 000	500, 000
Sub-Total		1, 500, 000	1, 000, 000	500, 000
Grand - Total		16, 900, 000	5, 662, 000	11, 238, 000
Engineering Fee	5%	840, 000	278, 000	562, 000
Administration Cost	4%	670, 000	0	670, 000
Reserve	10%	1, 690, 000	560, 000	1, 130, 000
Total		20, 100, 000	6, 500, 000	13, 600, 000

7.2 Summary of Construction Cost (1st Stage)

I. Underground Water:

(unit: Baht)

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	780,000	439,000	341,000
2) Raw Water Main	320,000	0	320,000
3) Water Treatment Plant	3,000,000	1,717,000	1,283,000
4) Distribution System	7,260,000	2,479,000	4,781,000
5) Electric Equipment	1,500,000	1,000,000	500,000
Sub-Total	12,860,000	5,635,000	7,225,000
Engineering Fee	640,000	640,000	0
Administration Cost	510,000	0	510,000
Reserve	1,290,000	525,000	765,000
Grand Total	15,300,000	6,800,000 (44%)	8,500,000 (56%)

II. Surface Water:

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	2,800,000	475,000	2,325,000
2) Raw Water Main	440,000	0	440,000
3) Water Treatment Plant	4,900,000	1,708,000	3,192,000
4) Distribution System	7,260,000	2,479,000	4,781,000
5) Electric Equipment	1,500,000	1,000,000	500,000
Sub-Total	16,900,000	5,662,000	11,238,000
Engineering Fee	840,000	278,000	562,000
Administration Cost	670,000	0	670,000
Reserve	1,690,000	560,000	1,130,000
Grand Total	20,100,000	6,500,000 (32%)	13,600,000 (68%)

7.3 Breakdown of Construction Cost (2nd Stage)

I. Underground Water:

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Deep Well Intake				
1-1 Land Cost	216 m^2	5,000	0	5,000
1-2 Deep Well	1 set	230,000	110,000	120,000
1-3 Pump	1 set	108,000	98,000	10,000
1-4 Pump House	9 m^2	27,000	0	27,000
1-5 Miscellaneous	5.4%	20.000	10,000	10,000
Sub-Total		390,000	218,000	172,000
2) Raw WaterMain				
2-1 ACP ϕ150 mm	800^m	70,000	0	70,000
2-2 Expenditure	20%	14,000	0	14,000
2-3 Pavement	800 m^2	36,000	0	36,000
2-4 Miscellaneous	8.3%	10,000	0	10,000
Sub-Total		130,000	0	130,000
3) Water Treatment Plant				
3-1 Coagulation & Filtration Equipment	22.2 m^2	737,000	660,000	77,000
3-2 Drainage	1 set	8,000	0	8,000
3-3 Pipe in Plant	1 set	55,000	50,000	5,000
3-4 Miscellaneous	5%	40,000	35,000	5,000
Sub-Total		840,000	745,000	95,000
4) Distribution System				
4-1 Pump No. 1	1 set	95,000	95,000	0
" No. 2	3 sets	240,000	240,000	0
4-2 Elevated Tank	120 m^3	462,000	30,000	432,000
4-3 Distribution Pipe				
ACP ϕ150	2,620 m	275,362	0	275,362
" ϕ100	9,340 m	726,652	0	726,652
4-4 Valve ϕ300	2	21,648	21,648	0
" ϕ150	3	11,433	11,433	0
" ϕ100	18	53,244	53,244	0
4-5 Hydrant - Single	46	229,540	229,540	0
4-6 Pavement	11,960 m^2	538,200	0	538,200
4-7 Expenditure (20% of local)	1	200,000	0	200,000
4-8 Miscellaneous	5.2%	146,921	34,135	112,786
Sub-Total		3,000,000	715,000	2,285,000
5) Electric Equipment	1 set	300,000	200,000	100,000
Sub-Total		300,000	200,000	100,000
Grand-Total		4,660,000	1,878,000	2,782,000
Engineering Fee	5%	230,000	230,000	0
Administration Cost	4%	180,000	0	180,000
Reserve	10%	460,000	182,000	278,000
Total		5,530,000	2,290,000	3,240,000

II. Surface Water:

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Intake Facilities				
1-1 Pump	2 sets	155, 000	140, 000	15, 000
1-2 Miscellaneous	4. 8%	75, 000	65, 000	10, 000
Sub-Total		230, 000	205, 000	25, 000
2) Water Treatment Plant				
2-1 Sedimentation	2 sets	996, 000	92, 000	904, 000
2-2 Filtration	2 sets	40, 000	34, 000	6, 000
2-3 Pipe in plant	1 set	59, 000	54, 000	5, 000
2-4 Miscellaneous	5%	55, 000	10, 000	45, 000
Sub-Total		1, 150, 000	190, 000	960, 000
3) Distribution System as well as underground water	-	3, 000, 000	715, 000	2, 285, 000
Sub-Total		3, 000, 000	715, 000	2, 285, 000
4) Electric Equipment as well as underground water		300, 000	200, 000	100, 000
Sub-Total		300, 000	200, 000	100, 000
Grand Total		4, 680, 000	1, 310, 000	3, 370, 000
Engineering Fee	5%	240, 000	240, 000	0
Administration Cost	4%	180, 000	0	180, 000
Reserve	10%	500, 000	150, 000	350, 000
Total		5, 600, 000	1, 700, 000	3, 900, 000

7.4 Summary of Construction Cost (2nd Stage)

I. Underground Water

(unit: Baht)

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	390,000	218,000	172,000
2) Raw Water Main	130,000	0	130,000
3) Water Treatment Plant	840,000	745,000	95,000
4) Distribution System	3,000,000	715,000	2,285,000
5) Electric Equipment	300,000	200,000	100,000
Sub-Total	4,660,000	1,878,000	2,782,000
Engineering Fee	230,000	230,000	0
Administration Cost	180,000	0	180,000
Reserve	460,000	182,030	278,000
Grand Total	5,530,000	2,290,000	3,240,000

II. Surface Water

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	230,000	205,000	25,000
2) Water Treatment Plant	1,150,000	190,000	960,000
3) Distribution System	3,000,000	715,000	2,285,000
4) Electric Equipment	300,000	200,000	100,000
Sub-Total	4,680,000	1,310,000	3,370,000
Engineering Fee	240,000	240,000	0
Administration Cost	180,000	0	180,000
Reserve	500,000	150,000	350,000
Grand Total	5,600,000	1,700,000	3,900,000

7.5 Breakdown of Construction Cost (3rd Stage)

I. Underground Water

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Deep Well Intake				
1-1 Land Cost	216 m^2	5,000	0	5,000
1-2 Deep Well	1 set	230,000	110,000	120,000
1-3 Pump	1 set	108,000	98,000	10,000
1-4 Pump House	9 m^2	27,000	0	27,000
1-5 Miscellaneous	5.4%	20,000	10,000	10,000
Sub-Total		390,000	218,000	172,000
2) Raw Water Main				
2-1 ACP ϕ150 mm	600 m	53,000	0	53,000
2-2 Expenditure	20%	10,000	0	10,000
2-3 Pavement	600 m^2	27,000	0	27,000
2-4 Miscellaneous	4.5%	40,000	0	40,000
Sub-Total		130,000	0	130,000
3) Water Treatment Plant				
3-1 Coagulation & Filtration Equip.	26.4 m^2	1,029,000	920,000	109,000
3-2 Alum Feeding Equip.	1 set	148,000	133,000	15,000
3-3 Lime Feeding Equip.	1 set	120,000	110,000	10,000
3-4 Chlorination	1 set	130,000	120,000	10,000
3-5 Miscellaneous	5.1%	73,000	57,000	16,000
Sub-Total		1,500,000	1,340,000	160,000
4) Distribution System				
4-1 Pump No. 1	1 set	95,000	95,000	0
" No. 2	1 set	65,000	65,000	0
4-2 Elevated Tank	120 m^3	462,000	30,000	432,000
4-3 Valves ϕ300	2 ca	22,000	22,000	0
4-4 Miscellaneous	5.6%	36,000	18,000	18,000
Sub-Total		680,000	230,000	450,000
5) Electric Equipment	1 set	100,000	70,000	30,000
Sub-Total		100,000	70,000	30,000
Grand Total		2,800,000	1,858,000	942,000
Engineering Fee	5%	140,000	140,000	0
Administration Cost	4%	110,000	0	110,000
Reserve	10%	280,000	182,000	98,000
Total		3,330,000	2,180,000	1,150,000

II. Surface Water

(unit: Baht)

Item	Quantity	Total	Foreign Currency	Local Currency
1) Intake Facilities				
1-1 Pump	2 sets	155,000	140,000	15,000
1-2 Miscellaneous	4.8%	75,000	65,000	10,000
Sub-Total		230,000	205,000	25,000
2) Water Treatment Plant				
2-1 Sedimentation	2 sets	1,127,000	47,000	1,080,000
2-2 Filtration	3 sets	60,000	51,000	9,000
2-3 Chlorination	1 set	130,000	120,000	10,000
2-4 Miscellaneous	4.5%	63,000	12,000	51,000
Sub-Total		1,380,000	230,000	1,150,000
3) Distribution System as well as underground water	1 set	680,000	230,000	450,000
Sub-Total		680,000	230,000	450,000
4) Electric Equipment as well as Underground water	1 set	100,000	70,000	30,000
Sub-Total		100,000	70,000	30,000
Grand Total		2,390,000	735,000	1,655,000
Engineering Fee	5%	120,000	120,000	0
Administration Cost	4%	96,000	0	96,000
Reserve	10%	234,000	75,000	159,000
Total		2,840,000	930,000	1,910,000

7.6 Summary of Construction Cost (3rd Stage)

I Underground Water

(Unit: Baht)

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	390, 000	218, 000	172, 000
2) Raw Water Main	130, 000	0	130, 000
3) Water Treatment Plant	1, 500, 000	1, 340, 000	160, 000
4) Distribution System	680, 000	230, 000	450, 000
5) Electric Equipment	100, 000	70, 000	30, 000
Sub-Total	2, 800, 000	1, 858, 000	942, 000
Engineering Fee	140, 000	140, 000	0
Administrative Cost	110, 000	0	110, 000
Reserve	280, 000	182, 000	98, 000
Grand Total	3, 330, 000	2, 180, 000	1, 150, 000

II. Surface Water

Item	Total Cost	Foreign Currency	Local Currency
1) Intake	230, 000	205, 000	25, 000
2) Water Treatment Plant	1, 380, 000	230, 000	1, 150, 000
3) Distribution System	680, 000	230, 000	450, 000
4) Electric Equipment	100, 000	70, 000	30, 000
Sub-Total	2, 390, 000	735, 000	1, 655, 000
Engineering Fee	120, 000	120, 000	0
Administration Cost	96, 000	0	96, 000
Reserve	234, 000	75, 000	159, 000
Grand Total	2, 840, 000	930, 000	1, 910, 000

Chapter 8 Financial Plan & Water Rate

8 1 Financial Prospect

In the preceding Chapters, existing facilities and future water demand were studied and analyzed. And as a result of these studies and analyzes, a construction program was drawn out. The construction of new facilities require a sizable amount of funds in both local and foreign currencies. Availability of funds may necessitate modification of construction schedule. For one thing, the amount of available funds must be made sure before everything. For second, terms and conditions of funds must be acceptable to this project. When these are not satisfied, the schedule of construction must be accordingly changed.

In general, it is desirable for water supply projects with slow speed of capital rotation to be carried out using long-term, low-interest fund. However, since it is difficult to procure such long-term, low-interest funds in domestic financial market of a developing country, it was assumed that for this water supply system foreign currency fund requirements would be met by foreign loans and local currency requirements by Thai Government funds.

Example of loans obtained from foreign countries of this type in Thailand currently are the World Bank loan, Asian Development Bank, Yen-Credit and so forth. However, this project of water supply in Pattani municipality is probably financed by loan from Japanese Government Economic Cooperation Fund with the terms contemplated to be an interest rate of 4% and amortization periods 20 years including a grace of 5 years, but it cannot be concluded that loans in the future can be secured at such terms.

8. 2 Past Data of Revenue

According to Pattani Municipality, following data has been reported,

Table 8. 1 Past Revenue

Budget Year	Amount of Water Supply (m^3)	Revenue (Bahts)
1961	263, 400	220, 687. 50
1962	282, 000	295, 101. 88
1963	293, 000	332, 404. 36
1964	326, 000	476, 072. 95
1965	346, 751	512, 339. 37
1966	358, 000	461, 916. 23
1967	410, 000	509, 673. 03
1968	438, 000	351, 218. 78
1969	458, 000	462, 972. 36
1970	584, 000	393, 260. 56

8. 3 General Consideration & Water Rate

Financial plan has been made by following conditions partially as stated as before.

(1) Limited for 1st stage project only, which is made for 7 years from 1974 to 1980.

(2) Amortization schedule as Table 8. 2,

Table 8. 2 Amortization Schedule

	Foreign Fund	Local Fund
Interest	4%	8%
Term of Loan	20 years	30 years
Grace Period	5 years	5 years
Annual Rate of Amortization	0. 0899414	0. 0936787

(3) Cost of Maintenance and Management as Table 8. 3 and 8. 4

Table 8. 3 Cost of Maintenance & Management

(Underground Water)

Year	Personnel cost	Miscellaneous (1) x 20%	Repair Expenditure (1) x 10%	Chemical cost	Power cost	Total
1973	—	—	—	—	—	—
74	98, 540	19, 708	9, 854	80, 983	216, 026	425, 111
75	102, 490	20, 498	10, 249	88, 457	223, 884	445, 578
76	106, 590	21, 318	10, 659	97, 323	233, 766	469, 656
77	110, 850	22, 170	11, 085	105, 741	241, 832	491, 678
78	115, 280	23, 056	11, 528	114, 379	250, 920	515, 163
79	119, 890	23, 978	11, 989	124, 898	261, 177	541, 932
80	124, 690	24, 938	12, 469	135, 639	270, 594	568, 330
	778, 330	155, 666	77, 833	747, 420	1, 698, 199	3, 457, 448

Table 8. 4 Cost of Maintenance of Management

(Surface Water)

Year	Personnel cost	Miscellaneous (1) x 20%	Repair Expenditure (1) x 10%	Chemical cost	Power cost	Total
1973	—	—	—	—	—	—
74	98, 540	19, 708	9, 854	59, 153	278, 624	465, 879
75	102, 490	20, 498	10, 249	64, 633	287, 467	485, 337
76	106, 590	21, 318	10, 659	71, 049	299, 283	508, 899
77	110, 850	22, 170	11, 085	77, 228	308, 335	529, 668
78	115, 280	23, 056	11, 528	83, 566	318, 372	551, 802
79	119, 890	23, 978	11, 989	91, 198	330, 600	577, 655
80	124, 690	24, 938	12, 469	99, 089	340, 966	602, 152
	778, 330	155, 666	77, 833	545, 916	2, 163, 647	3, 721, 392

(4) Water Rate is based on Table 8. 5,

Table 8. 5 Effective Rate of Water Supply

Year	Popu-lation	House Connec-tion %	Daily Mean Demand					Effective Quantity for Revenul		
			municipality		Outside need	Total		Effective Ratio %		
			(l/c/d)	(m^3/d)	(m^3/d)	(m^3/d)	(m^3/yr.)		(m^3/d)	(m^3/yr.)
1974	28, 200	62. 0	132	2, 308	267	2, 575	937, 875	70	1, 803	657, 913
75	29, 100	62 5	133	2, 419	278	2, 697	984, 405	70	1, 888	689, 120
76	30, 100	63. 0	135	2, 561	289	2, 850	1, 040, 250	70	1, 995	728, 175
77	31, 000	63. 5	136	2, 678	300	2, 978	1, 086, 970	70	2, 085	760, 879
78	31, 900	64. 0	137	2, 797	310	3, 107	1, 134, 055	70	2, 175	793, 839
79	32, 900	64. 5	139	2, 950	321	3, 271	1, 193, 915	70	2, 290	835, 741
80	33, 800	65. 0	140	3, 076	334	3, 410	1, 244, 650	70	2, 387	871, 255
—	—	—	—	—	—	—	—	—	—	5, 336, 922

a: Underground Water

Management Cost	:	3, 457, 448. 00	Baht
Amorization	:	8, 983, 611. 41	"
Total		12, 441, 059. 41	"

$$\text{Water Charge} = \frac{12,441,059.41 \text{ Baht}}{5,336,922 \text{ m}^3} = 2.3311 = 2.35 \text{ Baht}$$

b: Surface Water

Management Cost	:	3, 721, 392. 00
Amorization	:	12, 315, 948. 26
Total		16, 037, 340. 26

$$\text{Water Charge} = \frac{16,037,340.26}{5,336,922} = 3.0048 = 3.00 \text{ Baht}$$

3.4 Financial Plan

Table 8.6 Financial Plan onUnderground Water

(Water Charge 2.35 Bants/m^3)

Year	Income			Expenditure				Balance
	Borrowing	Water Charge	Total	Construction Cost	Management Cost	Amotization	Total	
1973	15,300,000,00		15,300,000.00	15,300,000.00		952,000.00	16,252,000.00	△ 952,000.00
74		1,546,096.00	1,546,096.00		425,111.00	952,000.00	1,377,111.00	168,985.00
75		1,619,432.00	1,619,432.00		445,578.00	952,000.00	1,397,578.00	221,854.00
76		1,711,211.00	1,711,211.00		469,656.00	952,000.00	1,421,656.00	289,555 00
77		1,788,066.00	1,788,066.00		491,678.00	952,000.00	1,443,678.00	344,388.00
78		1,865,522.00	1,865,522.00		515,163.00	1,407,870.47	1,923,033.47	△ 57,511.47
79		1,963,991.00	1,963,991.00		541,932.00	1,407,870.47	1,949,802.47	14,188.53
80		2,047,449.00	2,047,449.00		568,330.00	1,407,870.47	1,976,200.47	71,248 53
Total	15,300,000.00	12,541,767.00	27,841,767.00	15,300,000.00	3,457,448.00	8,983,611.41	27,741,059.41	100,707.59

Remarks : Signal △ in Balance ---------- deficit

Table 8.7 Financial Plan on Surface Water

(Water Charge 3.0 Bahts/m^3)

Year	Income			Expenditure				Balance
	Borrowing	Water Charge	Total	Construction Cost	Management Cost	Amotization	Total	
1973	20,100,000.00		20,100,000.00	20,100,000.00		1,348,000.00	21,448,000.00	△ 1,348,000.00
74		1,973,739	1,973,739.00		465,879.00	1,348,000.00	1,813,879.00	159,860.00
75		2,067,360	2,067,360.00		485,337.00	1,348,000.00	1,833,337.00	234,023.00
76		2,184,525	2,184,525.00		508,899.00	1,348,000.00	1,856,899.00	327,626.00
77		2,282,637	2,282,637.00		529,668.00	1,348,000.00	1,877,668.00	404,969.00
78		2,381,517	2,381,517.00		551,802.00	1,858,649.42	2,410,451.42	△ 28,934.42
79		2,507,223	2,507,223.00		577,655.00	1,858,649.42	2,436,304.42	70,918.58
80		2,613,765	2,613,765.00		602,152.00	1,858,649.42	2,460,801.42	152,963.58
Total	20,100,000.00	16,010,766	36,110,766,00	20,100,000.00	3,721,392.00	12,315,948.26	36,137,340.26	△ 26,574.26

Remarks : Signal △ in Balance ---------- deficit

Appendix-1

THE ENGAGEMENT OF CONSULTING ENGINEERS

PART I

IMPROVEMENT OF EXISTING WATER WORKS FACILITIES AND EMERGENCY SUPPLY OF WATER

1. IMPROVEMENT OF EXISTING WATER TREATMENT PLANTS, PUMPING PLANTS AND OTHER ASSOCIATED FACILITIES INCLUDING RAW WATER SUPPLY FACILITIES TO INCREASE TREATED WATER OUTPUT

Consulting engineers are required to study existing water treatment plants, pumping plants and other associated facilities including raw water supply facilities and to find the most economical and suitable means to increase treated water supply output by improving existing facilities.

Detailed designs, drawings and specifications ready for tender are required.

2. IMPROVEMENT OF EXISTING WATER DISTRIBUTION NETWORKS

2.1 LEAKAGE STUDY

Consulting engineers are required to make leakage study of the distribution networks and submit a report showing means of reduction and other necessary recommendations.

Consulting engineers are required to train water works personnel on leakage survey of the distribution networks and on the use of the equipment for leakage study.

2.2 DISTRIBUTION NETWORKS STUDY

Consulting engineers are required to study existing distribution networks including all house connections and submit a report for their improvement. Report should contain designs and specifications to facilitate efficient operation (including fire fighting) and maintenance of the distribution networks. Consulting engineers are required to supervise construction and to train water works personnel in the design, operation and maintenance of distribution systems.

3. EMERGENCY SUPPLY OF WATER DURING THE PLANNING AND EXECUTION OF WORKS IN THE FIRST PHASE OF MASTER PLAN

Consulting engineers are required to review the emergency water supply projects proposed by the Department of Publics and Municipal Works and submit a report on their feasibility and recommendation on actions to be taken.

Consulting engineers are required to study the problems of emergency supply of water during the planning and execution of works in the first phase of Master Plan. Detailed designs, drawings and specifications ready for tender are required.

PART II

MASTER PLAN

Consulting engineers are required to prepare a long range Master Plan of water supply for Pattani municipality including bordering areas must likely to come under the Water Works in the future, for the next 30 years with recommendations with regard to the number of phases and the extent and cost of development and data of commencement and completion of each phase.

Report must include for each phase of the Master Plan, estimates of future population, future demands, source of water of adequate quantity and quality, necessary storage, raw water systems, treatment facilities, distribution systems, distribution reservoirs, pumping stations, elevated storage, provision for the establishment of an organization capable of maintaining and operation the supply, distribution and treatment facilities, and cost estimates for each of the various phases of the Master Plan.

Detailed designs, drawings and specifications of the first phase of Master Plan, ready for tender, are required.