REPUBLIC OF INDONESIA
MINISTRY OF PUBLIC WORKS(DPU)
DIRECTORATE GENERAL OF
HUMAN SETTLEMENT (CIPTA KARYA)

UJUNG PANDANG

WATER SUPPLY DEVELOPMENT PROJECT

VOLUME V

SUPPORTNG REPORTS
FOR
FEASIBILITY STUDY

NOVEMBER 1985

JAPAN INTERNATIONAL COOPERATION AGENCY

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I. COMPARISON OF ALTERNATIVES (II)

1. COMPARATIVE STUDY OF CONSTRUCTION SCHEDULE

1.1 General

In the Master Plan Report alternative studies were made on such subjects as the method of raw water transmission, the location of the treatment plant and type of distribution systems. Each subject was studied as an integral part of a total system of water supply. (See V. "COMPARISON OF ALTERNATIVES" in Supporting Reports (I).)

The present Study relates to the Stage I Project with a target year, 1995, which is further divided into Phases 1 and 2.

The Master Plan Report proposes that the project be implemented according to the following schedule:

- Treatment Plant:
  Construction will be done in two steps for Phases 1 and 2. Because of a limited time for construction, however, the two steps are expected to take place in succession.

and,

- Raw Water Transmission Pipeline:
  Pipeline with capacity to meet demand for both Phases 1 and 2 will be constructed by the target year of Phase 1.

In this connection, a rationale of the Master Plan for proposing two phases of construction may be required to be verified.

1.2 Alternative Construction Schedule

The subject to be dealt with in this paper is the construction schedules of:

1) Treatment plant, and
2) Raw water transmission pipeline.

1) Construction of Treatment Plant

Alternative schedules for the construction of a new treatment plant are:

- Alternative 1-1: One-step construction to meet the total capacity of 1.0 m3/sec within Phase 1 (1987 - 1991)

- Alternative 1-2: Two-step construction:
  Phase 1 (Construction period: 1987 - 1991) of 0.5 m3/sec plant, and
  Phase 2 (Construction period: 1991 - 1993) of additional 0.5 m3/sec.

2) Construction of Raw Water Transmission Pipeline

Alternative plans for construction of the raw water transmission pipeline/s are:

- Alternative 2-1: One-step construction of a single line of 1,100 mm pipe which covers the total capacity of 1.0 m3/sec within Phase 1

- Alternative 2-2: Two-step construction:
  A pipeline of 900 mm in Phase 1, and an additional pipeline of 800 mm in Phase 2

(Note): - Raw water requirement includes 0.04 m3/sec for

Sungguminasa City and system losses; and is projected to amount to:

Phase 1 = 0.55 m3/sec
Phase 2 = 0.55 m3/sec
Total = 1.10 m3/sec

- Length of pipeline: 19,500 m
- Potential head between grit chamber and receiving well = (+37.5 m) - (+9.0 m) = 28.5 m

1.3 Cost Comparison

As the discount rates applicable to the study of cost comparison, three rates, i.e., i) 10 %, ii) 15 %, and iii) 20 %, were adopted, as they are most frequently used for the studies of other similar projects and are also considered suitable to the current conditions in Indonesia.

1) Treatment Plant

Comparison was made only of construction costs, exclusive of land costs and O/M costs which were found almost same in the two alternatives. The results of comparison are shown in Table 1.1, and illustrated in Figure 1.1 (1).

The results of comparison show that the cost of the two-step construction (Alternative 1-2)is more economical than that of one-step construction (Alternative 1-1) by about Rp. 500 million. This economical advantage becomes more remarkable when higher discount rates are applied.

2) Raw Water Transmission Pipeline

Similar cost comparison was made for the raw water transmission pipeline and shown in Table 1.2 and Figure 1.1 (2). In this case, the two-step construction (Alternative 2-1) was verified to be more economical than the one-step construction (Alternative 2-2).

The results of comparison show that when higher discount rates are applied, the difference of costs between the two alternatives narrows; however, even at the discount rate of 20 %, Alternative 2-1 still maintains an economical advantage over Alternative 2-2. The reason is considered to be that 1) the basic construction cost of Alternative 2-2 is 20 % more expensive than that of Alternative 2-1, and 2) Phase 2 construction is expected to start immediately after completion of Phase 1.

1.4 Conclusion

From the above study, it is concluded that Alternative 1-2 (Two-step construction) for the treatment plant, and Alternative 2-1 (One-step construction) for the raw water transmission pipeline are more advantageous.

In addition to the economical advantage, these alternatives will have the following advantages as well:

<u>For Treatment Plant:</u>

- Amount of pre-investment can be reduced.
- Facility expansion can be adjusted to the real pace of increase in future water demand.
- Construction period can be shortened and the date of the new system commisioning can be made in an earlier stage.

For Raw Water Transmission Pipeline:

- As construction is one-step, the impact of construction on traffic and other facilities along the pipe route can be minimized.
- As the pipeline can be single, more space along the route can be left for the Stage II pipeline construction.

TABLE 1.1 COST COMPARISON FOR TREATMENT PLANT

Unit : F/C : x 1,000 US$
: L/C : million Rp.

Year	Alternative 1-1					Alternative 1-2				
	Disbursement		Discount Rate			Disbursement		Discount Rate		
	F/C	L/C	10%	15%	20%	F/C	L/C	10%	15%	20%
1985										
1987		1,132	935	856	786	-	774	640	585	537
1988		1,133	851	745	656	-	774	582	509	448
1989	693	1,133	1,302	1,090	919	480	774	894	749	631
1990	2,957	1,133	2,751	2,203	1,780	2,049	775	1,900	1,521	1,230
1991	1,824	1,133	1,788	1,369	1,061	1,264	774	1,233	944	731
Phase 1	5,474	5,664	7,627	6,262	5,202	3,793	3,871	5,248	4,308	3,578
						(52%)	(48%)			
1991						-	618	349	267	207
1992						1,223	742	1,081	792	588
1993						458	433	440	308	219
Phase 2						1,681	1,793	1,870	1,367	1,014
						(51%)	(49%)			
GRAND TOTAL			7,627	6,262	5,202			7,118	5,675	4,592

Note; 1 thousand US$ = 1.115 million Rp.

TABLE 1.2 COST COMPARISON OF RAW WATER TRANSMISSION PIPELINE

Unit: F/C : x1,000 US$
L/C : million Rp.

Year	Alternative 2-1					Alternative 2-2				
	Disbursement		Discount Rate			Disbursement		Discount Rate		
	F/C	L/C	10%	15%	20%	F/C	L/C	10%	15%	20%
1988	-	813	611	535	470	-	554	416	364	321
1989	1,606	1,219	2,056	1,721	1,452	1,094	830	1,400	1,172	989
1990	1,606	813	1,617	1,295	1,046	1,094	554	1,101	882	713
Phase 1			4,283	3,550	2,968			2,918	2,418	2,022
1991						899	796	1,015	777	602
1992						899	796	923	676	502
Phase 2								1,938	1,453	1,104
Total			4,284	3,550	2,968			4,856	3,871	3,126

Note ; 1 thousand US$ = 1.115 million Rp.

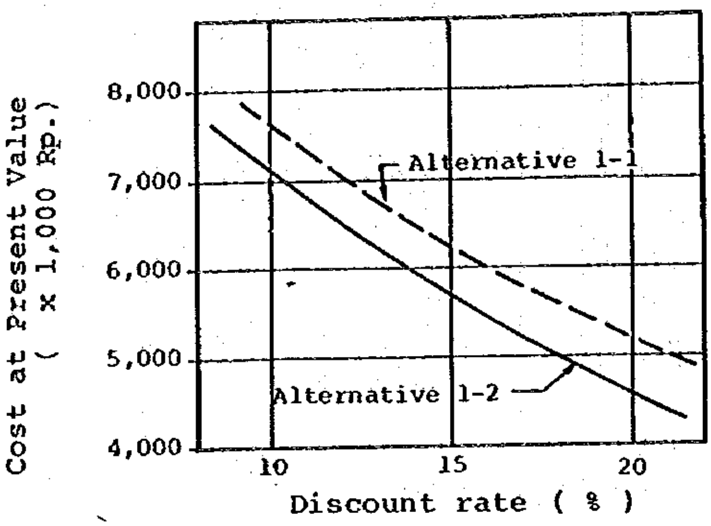


FIGURE 1.1(1) COST COMPARISON OF ALTERNATIVES (TREATMENT PLANT)

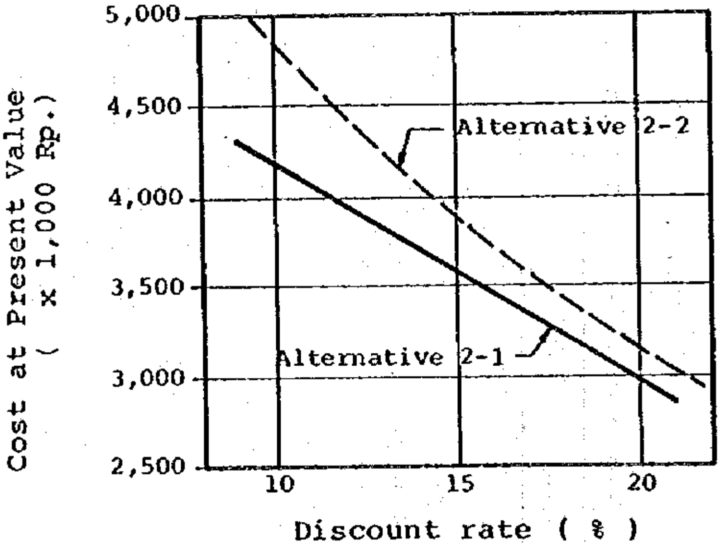


FIGURE 1.1(2) COST COMPARISON OF ALTERNATIVES (TRANSMISSION PIPELINE)

FIGURE 1.1	COST COMPARISON OF ALTERNATIVES

2. CONSIDERATION ON PIPING MATERIALS

2.1 General

This section intends to provide technical information on piping materials for distribution and transmission pipelines, keeping in mind that further discussions are to be made in the course of detailed design and tendering stages. Many kinds of pipes are currently manufactured, which are made of steel, concrete, cast iron, asbestos cement and plastic. Piping materials discussed in this section are listed below:

1) Steel pipe
2) Corrugated steel pipe
3) Ductile iron pipe
4) Polyvinyl chloride pipe
5) Asbestos cement pipe
6) Prestressed concrete pipe
7) Fiberglass reinforced plastic pipe

2.2 Characteristics of Pipes

Mechanical and physical characteristics of the above pipes are summarized in Table 2.1 and briefed hereunder.

1) Steel Pipe

Steel pipes, despite their high mechanical strength, has the nature of flexibility. They can be fabricated to any configuration required. Internal and external surfaces of the steel pipes are usually protected by bituminous coating reinforced with woven glass cloth and by cement mortar lining respectively.

Welding is in wide use to connect the pipes. Damaged parts due to the heat of welding should be repaired after installation by coating the external and internal surfaces. This jointing work is frequently disturbed by moisture such as raining and groundwater.

2) Corrugated Steel Pipe

Corrugated steel pipes have a thin thickness of the pipe wall and their surfaces are usually protected by zinc coating. Physical and mechanical nature of the pipes are the same as those of steel pipes. The pipes are not much tolerable of high internal and external pressure due to their thin walls. Pipe walls are folded solely to connect the pipes.

3) Ductile Iron Pipe

Composition of raw materials used for ductile iron, although similar to those of cast iron, yields ductility and impact resistance. Their corrosive nature is similar to that of cast iron. Therefore, the internal surface is usually coated by a thin layer of cement mortar and the external surface by tar epoxy. Materials used for jointing are ductile iron. Typical types of jointing are mechanical or push-on. Work force to be required for pipe connection is rather smaller than that for steel pipes.

4) Polyvinyl Chloride Pipe

Polyvinyl chloride pipes have an excellent nature of resistivity to chemical corrosion. In addition, handling to transport and jointing to connect pipes are relatively simple because of their lightness in weight.

Polyvinyl chloride pipes show unfavourable behavior in thermal condition. Their strength suddenly drops when temperature increases.

Due to their poor resistivity to heavy load, they should be laid on the sand bed and backfilled by sand. Push-on types of polyvinyl chloride pipes make jointing works easy. Diameter of the pipes manufactured ranges from 50 mm to 300 mm.

5) Asbestos Cement Pipe

Asbestos cement pipes have a nature of brittleness. For this reason, uncautious transportation and handling often damage the pipes. Further, they have a corrosive nature, frequently attacked by acid or sulfides. Sleeve type couplings with rubber rings are usually utilized for connection.

6) Prestressed Concrete Pipe

Prestressed concrete pipes are produced in providing prestressed wires in the concrete pipes during manufacturing. Although stronger than normal concrete pipes, they have a poor resistivity to heavy load. Furthermore, they are relatively heavy in weight. Therefore, utmost cares and attentions should be given to the handling and transportation of the pipes.

Since pipes consist of porous materials of concrete, ground water with high content of chloride can easily attack the highly tensioned wire. Once the wires are corroded, the pipes lose their own strength. Usual method of jointing is push-on type with o-shaped rubber ring.

7) Fiberglass Reinforced Plastic Pipe

Fiberglass reinforced plastic pipes have an excellent resistivity to chemical corrosion and are not heavy. In consideration of their flexibility and poor resistivity to internal and external pressure, it is desirable to install pipes with sand bed and backfilling. Push-on, sleeve coupling or flange type are usual jointing method.

2.3 Soil Condition of the Site

The climate of the project area is characterized by two typical rainy and dry seasons. It should be noted that the groundwater table is located high, ranging from 1.0 m to a few centimeter below the ground all the year.

As specified in the Master Plan Report, the area where electrical conductivity exceeds 1,000 micromho/cm spreads to the outskirts of the urban area.

To clarify further the soil condition of the area, an investigation was conducted during the period of the Feasibility Study. The survey points (No.1 - No.13) were selected as shown on Figure 2.1. The Team examined the soil condition on the following:

a) Resistivity
b) Redox potential
c) pH
d) Contents of sulfide
e) Wet/dry

Items listed above are all related to the aggressiveness of the soil. The results of the soil testing are summarized in Table 2.2. Paying particular attentions to the resistivity in the Table, soils characterized as corrosive are found to distribute in the fringe area of densely populated districts. The distribution is almost similar to that of electrical conductivity.

2.4 Consideration on Piping Materials

The above results should be considered in selecting piping materials of transmission and distribution pipelines.

Transmission pipeline, of which diameter is ϕ1,100, conveys raw water from the grit chamber to the Mangngasa treatment plant by gravity. It will be installed along Jl. Malino, where the soil condition is less corrosive.

The distribution pipelines form the network covering all the project area, through which the treated water will be distributed by pump from the treatment plant. The pipelines, ranging from 50 mm to 1,000 mm in diameter are installed along/across the roads. Most area where pipelines will be laid are covered by soils of aggressive nature.

The Team proposes following piping materials to be installed:

1) Raw water transmission pipelines with a low water pressure
   - Steel pipes/ Ductile iron pipes/Prestressed concrete pipes/Fiberglass reinforced plastic pipes

2) Raw water transmission pipeline except the above
   - Steel pipes/Ductile iron pipe/Fiberglass reinforced plastic pipes

3) Distribution pipelines at the center of the Municipality where soil conditions are relatively good,
   - Steel pipes/Ductile iron pipes/Fiberglass reinforced plastic pipes/ Polryvinyl chloride pipes/Asbestos cement pipes

   Distribution pipelines at the other area
   - Steel pipes coated by cathodic protection/Ductile iron pipe with polyethylene sleeve/Fiberglass reinforced plastic pipes/Polyvinyl chlorine pipes.

4) Pipebridge

- Steel pipes/Ductile iron pipes

The further discussion regarding piping materials listed above should be made on the basis of the cost, strength and materials to be studied in the stage of detailed design.

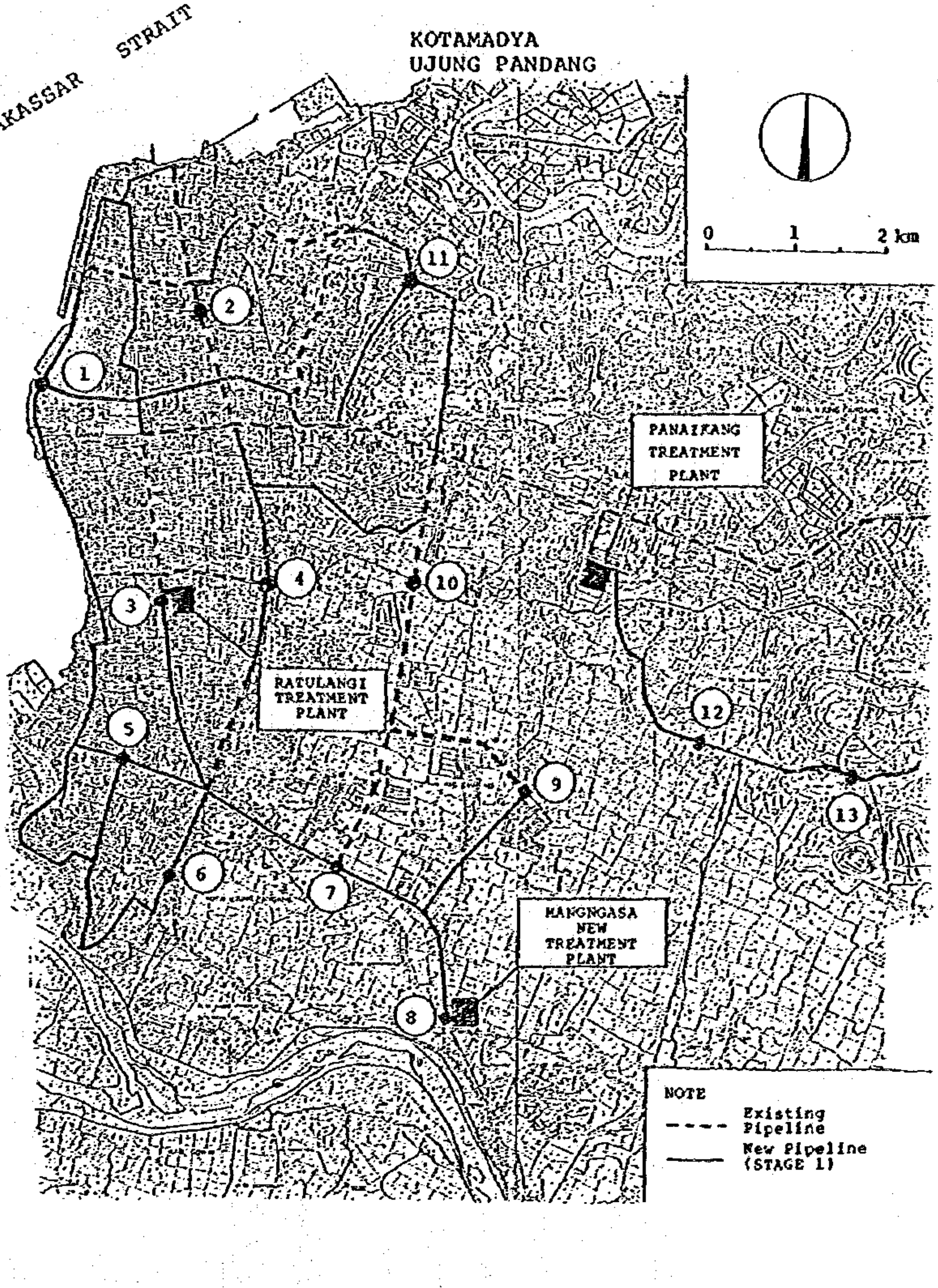


FIGURE	POINTS OF SOIL INVESTIGATION
2.1	

Table 2.1 CHARACTERISTICS OF PIPING MATERIALS

	Piping Materials	Corrosive/ non-corrosive	Handling and Workability	Water-tightness	Strength
1)	Steel Pipe	Corrosive	Welding and X-ray test is required	tight	strong
2)	Corrugated Steel Pipe	"	easy	low	weak
3)	Ductile Iron Pipe	"	easy	tight	strong
4)	Polyvinyl Chloride Pipe	Non-corrosive	easy	tight	weak
5)	Asbestos Cement Pipe	Corrosive	difficult because of its brittleness	tight	weak
6)	Prestressed Concrete Pipe	Corrosive	difficult	low	strong
7)	Fiberglass Reinforced Plastic Pipes	Non-corrosive	easy	tight	ordinary

TABLE 2.2 RESULTS OF SOIL INVESTIGATION

Survey Point	Depth (m)	Resistivity (ohm-cm)		Redox Potential (mV)	pH	Sulfide
		At the Site	At the Labo			
1	1.0	18,000	3,600	470	7	Nil
2	1.0	48,000	3,700	407	7	Nil
3	1.0	8,000	4,200	297	7	Nil
4	-	2,800	3,000	297	7	Nil
5	0.8	2,000	2,000	217	6	Objective
6	0.8	3,400	1,800	307	7	Nil
7	-	11,000	5,400	307	7	Nil
8	0.8	4,200	2,200	367	7	Nil
9	0.9	10,000	5,800	327	6	Nil
10	-	1,200	1,100	397	7	Nil
11	-	1,600	1,800	437	7	Nil
12	1.0	-	2,200	-	7	Nil
13	0.9	-	3,300	-	7	Nil

II. HYDRAULIC CALCULATION FOR PRELIMINARY DESIGN

II. HYDRAULIC CALCULATION FOR PRELIMINARY DESIGN

1. GENERAL

The outline of the facilities to be constructed up to the year 2005 is presented in the Master Plan report. In this paper the hydraulic calculation for the facilities to be constructed in the Stage I is made in order to provide the basis for preliminary design of the Stage I program.

2. BASIC FACTOR OF DESIGN

2.1 Design Flow

The design flow of the facilities in the Stage 1 is as follows:

- Intake Water Transmission Pipeline	: 1.06 m3/sec
- Raw Water Transmission Pipeline	: 1.06 m3/sec
- Treatment Facilities	: 1.03 m3/sec
- Distribution Facilities (Peak hour)	
Mangngasa system	: 1.17 m3/sec
Panaikang system	: 1.53 m3/sec

2.2 Water Level

Gravity flow is basically employed for the system. Water levels of the facilities to be taken for design are subject to topographical restrictions at their locations. Considering all these circumstances, the following water levels are employed for the design,

- Bili Bili Intake	: +41.0 m above sea level
- Grit chamber (outlet)	: +3.75 m "
- Receiving well	: + 9.0 m "
- Clear water reservoir	: H.W.L + 4.0 m "
	L.W.L. - 1.0 m "

3. INTAKE FACILITIES

3.1 Bili Bili Intake

Two intake gates by the size 1.4 x 1.4 m are provided at the irrigation intake.

The water level after the gates is calculated as below assuming that total flow of about 6.9 m3/sec, including 5.8 m3/sec (Max.) for the irrigation and 1.06 m3/sec for water supply, passes through the intake gates.

$$h = 1/C^2 \times v^2/2g = 0.44 \text{ m}$$

where,

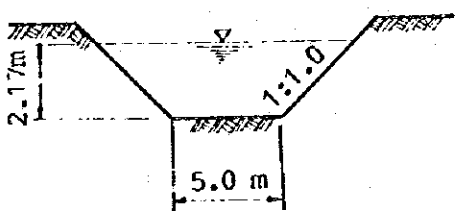
v : Mean velocity, Q/A = 6.9/(1.4x1.4x2) = 1.76 m/sec

C : Flow coefficient = 0.6

The water level at the initial point of the canal after the gates is calculated as +40.6 m by reducing the head loss about 40 cm, from the intke water level of +41.0 m.

3.2 Raw Water Canal

Required depth and velocity at the canal are computed employing Manning formula. As the results, the existing canal is to be expanded as shown in the sketch.



H = 2.17 m
V = 1.12 m/sec

A gate will be installed at the diversion into the irrigation canal to control flow rate. Since there is 3 m difference between the elevations of the starting point of the canal and the grit chamber, this figure is taken as the gradient of the canal flow. Accordingly, the water level at the inlet of the grit chamber is taken as EL. +37.6 m (=40.6 - 3.0).

3.3 Grit Chamber

The water diverged from the canal flows into the grit chamber passing through the coarse screen, inlet channel and gate. The loss of water is briefly calculated as below.

Coarse screen: $hs = k1 \times k2 \times k3(V^2/2g) = 0.02$ m

Inlet channel: $hc = I \times L = 0.001 \times 30 = 0.03$ m

Others : = 0.05 m

Total Loss = 0.10 m

The raw water at thr grit chamber has, therefore, a level of El.+37.5 m (=37.6 - 0.1). Perforated walls will be installed at the inlet and outlet of the grit chamber. Head loss at the inlet is computed as follows:

$$ho = (1/C^2) \times (V^2/2g) \times 2 = 0.06 \text{ m}$$

where,

Area of opening: $A = 2.5 \text{ m} \times 6.0 \text{ m} \times 0.08 = 1.2 \text{ m2}$

Velocity : $V = Q/A = (1.10/2)/1.2 = 0.46$ m/sec

As the loss due to perform wall is negligible the water levels at outlet is assumed at EL +37.5 m.

The length of the chamber is obtained by the following equation:

$$L = K \times H/U \times V = 1.5 \times (2.5/0.008) \times 0.04$$
$$= 18.8 \quad 19.0 \text{ m}$$

where,

L : Length of chamber (m)
H : Water depth (2.5 m)
U : Settling flow rate of the particles to be removed
V : Average flow rate in the chamber (4 cm/sec)
K : Safety coefficient (1.5)

4. RAW WATER TRANSMISSION PIPELINE

The topographic profile of the transmission route from the grit chamber to the Mangngasa Plant is shown on Figure 2.1. For the gently sloping portion from the grit chamber to Point No.2, 1,200 mm pipe is employed and for the steeper portion from Point No.2 to the plant, 1,100 mm pipe. These diameters are determined so as to accommodate the increased flow in the dry season.

4.1 Head Loss between Grit Chamber and Point No.5

The following calculation is made by Hazen - Williams Formula assuming that the piping 1,200 ø and 1,100 mm ø are laid in the span of 3,860 m.

$$H = 10.666 \times C^{-1.85} \times D^{-4.87} \times Q^{1.85} \times L$$
$$= 4.9 \text{ m}$$

where :

H : Head loss

C : Flow coefficient (C=120)

D : Diameter (mm)

Q : Flow rate

L : Length of pipeline

The water level at point No.5, the elevation which is +32.03 m is calculated at EL.+32.6 m (=37.5 - 4.9).

4.2 Head Loss between Point No.5 and Mangngasa Plant

Pipe with a diameter of 1,100 mm, laid between point No.5 and the Mangngasa treatment plant, has a length of 14,440 m. Employing Hazen-William's formula, head loss is computed as 15.4 m. The remaining head (17.2 m = 32.6 m - 15.4) is sufficient to convey water to the receiving point EL.+9.0 m, and the pipeline can accommodate the increased flow in the dry season.

5. MANGNGASA TREATMENT PLANT

The hydraulic calculation is made hereinafter employing flow rates of 0.5 m3/sec.

5.1 Receiving Well and Mixing Well

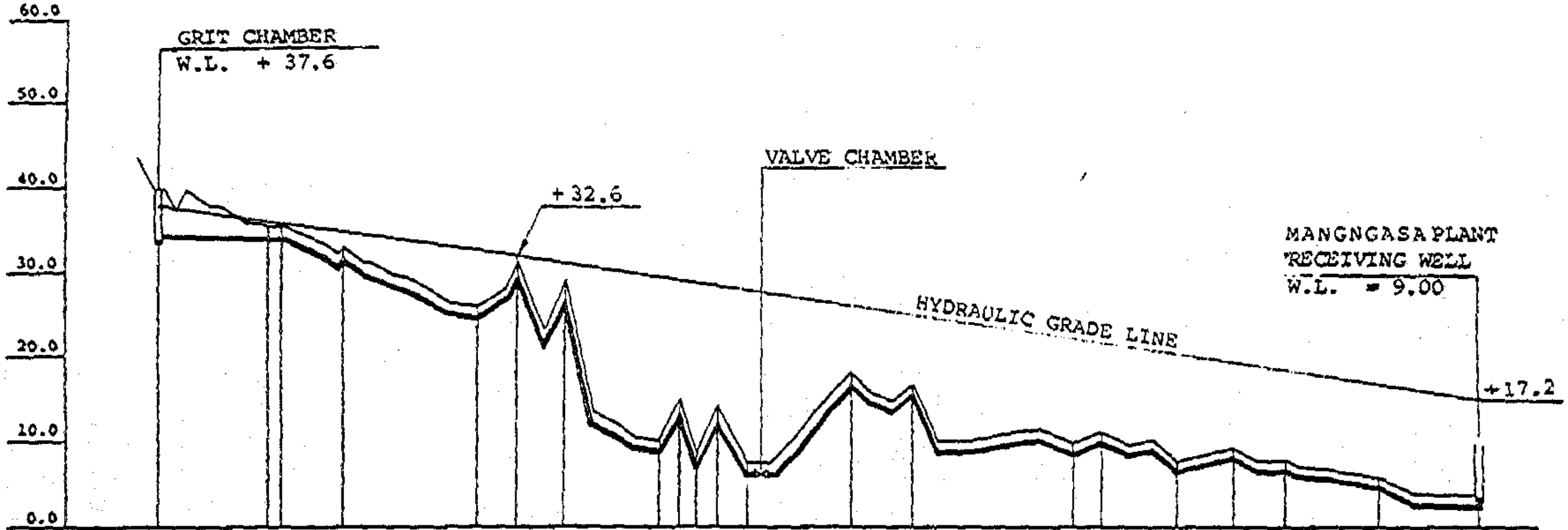
The receiving well is divided into two for easy and efficient maintenance. Stop logs and perforated walls are provided at the receiving well to control water flow. The head loss is obtained as below:

$$ho = 1/C^2 \times V^2/2g = 0.04 \text{ m}$$

where,

A : Area of the inlet, A = 2.5 x 5.0 x 0.08 = 1.0 m^2

V : Velocity, V = Q/A = (1.03/2)/1.0 = 0.52 m/sec



No.	Elevation	Distance (m)
0	44.00	0
1	36.00	1,140
2	35.70	1,490
3	33.68	2,450
4	26.64	4,400
5	32.03	5,060
6	30.00	5,710
7	10.33	7,210
8	15.70	7,490
9	9.12	7,740
10	14.77	8,080
11	8.02	8,520
12	18.44	10,100
13	17.20	11,060
14	10.08	13,440
15	11.48	13,880
16	8.09	14,270
17	9.55	15,830
18	8.10	16,640
19	5.93	18,060
20	4.00	19,500

FIGURE 2.1 PROFILE OF RAW WATER TRANSMISSION PIPELINE

Water level at the inlet of the mixing well reduces to EL.+8.95 m from EL.+9.0 m, water level at the inlet of receiving well. The rapid mixing of chemicals is designed to utilize the energy of 80 cm fall of water at the weir between the receiving well and the mixing well, of which the water level is EL.+8.10 m.

5.2 Flocculation and Sedimentation Basin

The raw water flows through the pipeline with a diameter of 700 mm to be installed between the mixing well and the flocculation basin. The loss at the pipeline is calculated as below:

$$hf = 124.5 \times n^2 \times v^2/2g \times d^{\;4/3} \times L$$
$$= 0.01 \text{ m}$$

Where,

L : Length, 20 m

n : Roughness coefficient, n = 0.014

v : Velocity, v = (1.03/4)/(0.785 x 0.7) = 0.70 m/sec

The water level at the inlet of the flocculation basin is computed at EL.+8.05 m taking into account 0.04 m the loss at the elbows and the outlet. Flocculation basin is divided into two. Vertical-flow baffles create turbulence in the water channel of basin. The head loss and G value at the flocculation basin are calculated, dividing into three steps as follow:

- First setp : Volume 12.5 m x 1.0 m x 3.5 m x 2 channels = 87 m3

  Overflow: $ho = fo \times V^2/2g \times 5 \times 2 = 114$ m

  Function: $ho = n^2 \times v^2/R^{(4/3)} \times 2 = 0.12$ m

  Bending : $hb = fb \times V^2/2g \times 6 \times 2 = 0.248$ m

  ---

  Total Head Loss = 0.374 m

Velocity Gradient = 97 sec^{-1}

- Second step : Volume 12.5 m x 1.2 m x 3.0 m
  x 2 channels = 93 m3
  Total Head Loss = 0.212 m
  Velocity Gradient = 71 sec^{-1}

- Third setp : Volume 12.5 m x 1.8 m x 2.9 m
  x 2 channels = 131 m3
  Total Head Loss = 0.092 m
  Velocity Gradient = 39 sec^{-1}

Considering total head loss in the floccuation basin 0.678 m as calculated above and other minor losses, the water level at the outlet is estimated to be EL.+7.20 m. Hence, the water level in the sedimentation basin is assumed at EL. +7.10 m. Two perforated walls are provided in the sedimentation basin. Loss by the perforated walls is negligible as shown below.

$$ho = 1/C^2 \times V^2/2g = 0.00 \text{ m}$$

where,

A : The area of the section, A = 12.5 m x 3.0 m x 0.08
= 3.0 m2

V : Velocity, V = Q/A = (1.03/4)/3.0 = 0.09 m/sec

Since head loss in the basin is small, the water level at the outlet of the basin is considered nearly equal to EL.+7.10 m. The other relevant factors for the sedimentation basin are computed as follows.

Overflow rate : Q/A = (1.03/4)/(12.5 x 64.0)
= 29 m/day

Froude number : $V^2/g.h = 0.00606^2/9.8 \times 3.5$
$= 1.07 \times 10^{-6}$

Weir loading	: WL = (1.03/4)/(7 x 4 x 2)	
	= 0.0046 m3/sec/m (=397 m3/day/m)	
Ave. flow rate	: V = (1.03/4)/(12.5 x 3.5)	

All the above are deemed appropriate for the design of sedimentation basin. Allowing for free fall of water from effluent launders to the outlet channel, the water level at the latter is determined at EL.+6.40 m.

5.3 Filters

Interfilter backwashing will be applied to the filter which consists of six beds. Filteration rate employed is 120 m/day.

Inlet gate loss	: $h1 = 1/0.6^2 \times V^2/2g = 0.01$ m
Inlet weir loss	: $h2 = (Q/1.838 \times B)^{2/3} = 0.08$ m
Secondary mixing	: $h3 = 1/0.6^2 \times V^2/2g \times 2$
	$= 0.01$ m
Others	: $h4 = 0.20$ m
Total head loss	$= 0.30$ m

The water level of the filter bed, hence, is EL.+6.10 m (=6.40 - 0.30). The low water level is taken as EL.+4.60 m which is 1.5 m lower than the high water level. This is required for varying water level filtration.

The head loss of the media is calculated by Fair-Hatch formula.

Head loss at the media	: $hf = 0.198$ m
Outlet gate loss	: $h2 = 1/0.6^2 \times V^2/2g$
	$= 0.01$ m
Under drain loss	: $h3 = 0.00$ m
Others	: $h4 = 0.09$ m
Total head loss	$= 0.30$ m

From the above, the water level at the clear water channel is EL.+4.03 m (=4.60 - 0.30).

5.4 Clear Water Reservoir

The filtered water flows into the clear water reservoir through the channel.

The head loss is calculated as below.

Clear water channel:

$$h1 = n^2 \times L \times V^2/R^{4/3} = 0.01 \text{ m}$$

where,

N : Roughness coefficient, N = 0.014

L : Length, L = 30 m

V : Velocity, V = 0.515/(1.5 x 1.0) = 0.34 m

R : Hydraulic radius, R = (1.5 +1.0) x 2/1.0 x 1.5
= 3.33

Inlet gate :

$$h2 = 1/0.6^2 \times (0.515/1.0)^2/19.6 = 0.04 \text{ m}$$

Filter outlet weir :

$$h3 = (Q/1.838 \times B)^{2/3}$$

$$= (0.515/1.838 \times 3.0)^{2/3}$$

$$= 0.21 \text{ m}$$

Others : h4 = 0.06 m

Total head loss = 0.30 m

Therefore, the high water level of the clear water reservoir is calculated as EL.+4.00 m and the low water level is fixed at EL.-1.00 m providing the effective water depth 5.0 m. The effective capacity is determined as five hour consumption, considering future expansion of the water supply system and the examples of the other cities.

6. DISTRIBUTION FACILITIES

To prepare preliminary design of the distribution system, the zoning of the service area will be focussed on in Subsection 6.1. The service area will be divided into plural zones for convenience of operation, leak detection and others. The pipe network analysis made in Subsection 6.2 intends to develop the distribution network in compliance with the design criteria, aiming at the economical design. The distribution network thus designed is also checked with its appropriateness for emergency water distribution when one of the supply sources surffers water shortage, in order to secure continuous water supply to all consumers. In Subsection 6.3, the location and capacity of the distribution tower are studied so that the treated water may be supplied effectively and continuously.

6.1 Zoning

The new water treatment plant is planned to be constructed under Stage I at Mangngasa in the southern part of the city so as to meet the expansion of the service area and increase of water demand. It is necessary to expand the distribution network along with this system development. Zoning to be studied hereinafter is to achieve independent water supply in each zone. This zone aims:

1) To make easy the flexible operation of distribution facilities in accordance with hourly variation of water demand.
2) To shorten the retention time of the water in the pipe network and to minimize the deterioration of the water quality,

3) To minimixe the occurrence of turbid water due to changes of flow direction in the pipes,
4) To minimize the influence of the accidents to the other area, and
5) To identify leaked water by zone.

6.1.1 Zoning Approach and Results

The supply zone is planned as follows.

1) Target Year

The two supply zone with different target year of each 1992 and 1995 are planned taking into account the phasing of the Stage I Project.

2) Number of Supply Zones

The total service area is divided largely in two supply zones, considering the following points.

i) There wil be two major supply sources, i.e., Panaikang and Mangngasa treatment plants.

ii) The smaller size of the supply zone is desirable from the technical points of view, but the initial cost and O&M cost of the distribution system will increase as the number of supply zones increases.

3) Boundaries

The boundary of the supply zone is to be determined taking into account (1) the flat service area, (2) water quality, and (3) cost for maintenance and operation.

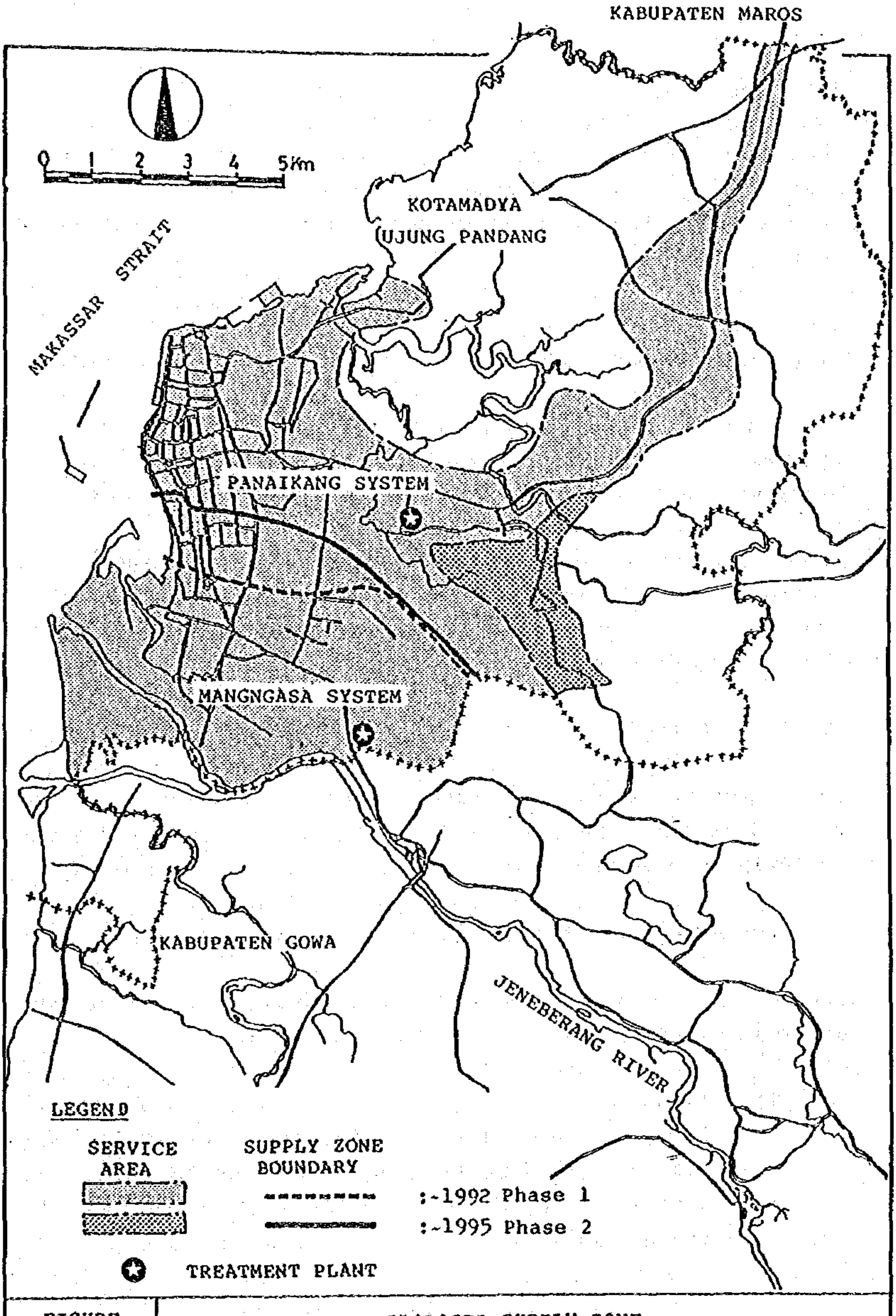


FIGURE	PROPOSED SUPPLY ZONE
6.1	

The supply zones as planned are shown in Figure 6.1. The two supply zones, being connected by trunk mains, are separated by the valves under normal condition. By this measure, any area of a zone will be supplied from the other zone through the connecting trunk mains, in case of water shortage.

6.2 Pipe Network Analysis

The objective of the analysis is to design the pipe network in compliance with the design criteria. The water pressure distribution in the pipe network is to be also discussed.

The distribution network thus designed is examined with regard to its appropriateness or capability in coping with emergency cases such as accidents in the water system, water shortage in drought periods and others.

6.2.1 Method of Analysis

To supply water to meet future water requirements in the planned service area, the network analysis was made by the following steps.

1) Formation of network skelton

The routes of pipelines and their connection points are determined taking into account the following.

i) Among the existing pipelines as shown in Figure 6.2, the new pipelines installed in 1970's and 1980's will be utilized continuously, while old pipelines installed in 1920's will be replaced by the new pipes.

ii) The trunk and secondary mains will be laid along the existing major roads and those to be constructed in future.

2) Distribution of the water demand over the network

Water demand by area for hydraulic calculation is estimated on the basis of the magnitude of area, the water demand in each district, and hourly maximum coefficient (K=1.3) as defined in the design criteria.

3) Collection of relevant data and assumption of flow rates in the pipeline and a pump head

The data to be required for the pipe network analysis are elevation of the area, diameter and length of existing pipeline. The flow rates and direction in the pipeline are provisionally given prior to the hydraulic calculation by trial and error method.

4) Calculation by trial and error method

Hazen-William's formula is utilized to calculate the friction loss of the pipeline. The C value including bend loss for existing new pipeline taken is C=110, and C=120 for pipeline to be newly constructed as described in the design criteria.

The minimum dynamic water pressure at the terminal point of the trunk main adopted is 15 m (1.5 kg/cm2) and the maximum hydrostatic pressure is 50 m (5.0 kg/cm2).

The diameter of each pipe of the proposed distribution network is calculated utilizing Energy Level Method. The results of pipe network analysis ensure that water pressure at all points are higher than the minimum dynamic water pressure at terminal points.

6.2.2 Results of Calculation

The figures calculated by the steps mentioned in the previous paragraph are shown in Figures 6.3 - 6.6 and Tables 6.1 - 6.4. Figure 6.7 shows a planned network which consists of trunk and secondary mains. Followings are recommended from the water pressure contour line (Figure 6.8).

The water pressure in the main distribution pipe adjacent to the treatment plant is about twice as high as that at the terminal point of the pipe. From the relation between water pressure and water loss by leakage, it can be assumed that the leaked water at the area adjacent to the plant is about 140% of that at the terminal point. Therefore the leakage prevention works are to be effectively executed in the area where the water pressure is comparatively high and where water leaks are detected.

Meanwhile, the water supply condition in case of raw water shortage was simulated as shown on Figure 6.9 and Table 6.5, supposing that Panaikang treatment plant decrease its water production to 765 l/sec. These results ensure that all customers in the service area can receive sufficient water all the time.

6.3 Distribution Tower

In Kotamadya Ujung Pandang, the water supply to the consumers is impeded by the power suspension for about one hour which is caused two or three times monthly by the accident mainly of transmission cable. The power supply situation is yearly improved but the power suspension is anticipated once a month even in the future. It is

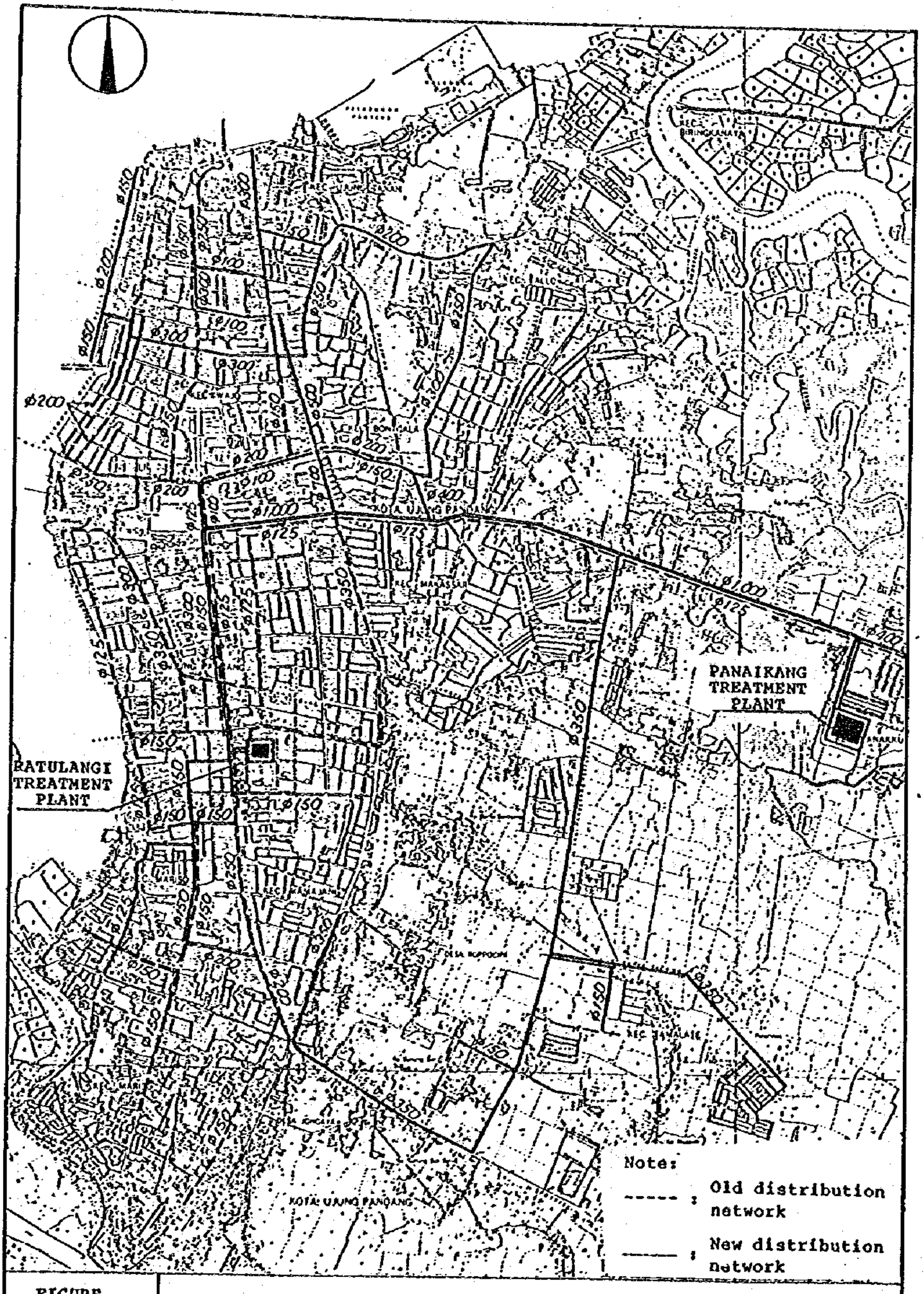


FIGURE	EXISTING DISTRIBUTION NETWORKS
6.2	

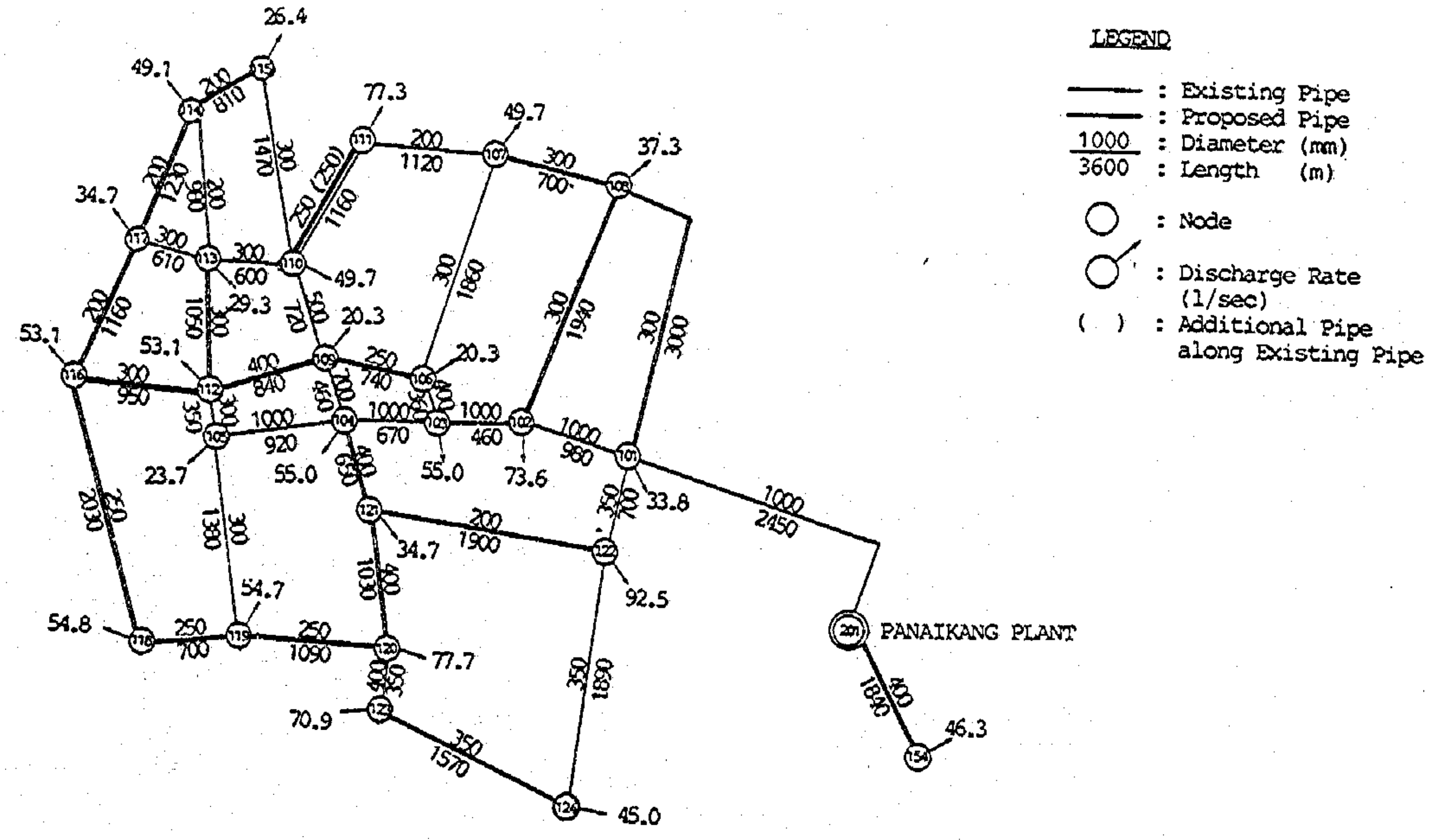


FIGURE	NETWORK DIAGRAM OF PANAIKANG SYSTEM (1992)
6.3	

TABLE 6.1 FLOW IN NETWORK OF PANAIKANG SYSTEM (1992)

Node - Node	D(mm)	L(m)	C	Q(l/sec)	V(m/sec)	i(o/oo)	dH(m)	H(m)	GL(m)	He(m)
201 - 101	1000	2450	110	1171.700	1.492	2.39	5.86	29.14	2.50	26.64
101 - 102	1000	980	110	947.057	1.206	1.61	1.58	27.56	2.50	25.06
102 - 103	1000	460	110	837.727	1.067	1.29	0.59	26.97	2.50	24.47
103 - 104	1000	670	110	710.559	0.905	0.95	0.64	26.33	2.50	23.83
104 - 105	1000	920	110	180.597	0.230	0.08	0.07	26.26	2.50	23.76
103 - 106	400	360	110	72.168	0.574	1.19	0.43	26.54	2.50	24.04
106 - 107	300	1860	110	31.725	0.449	1.06	1.97	24.57	2.50	22.07
108 - 107	300	700	120	36.329	0.514	1.16	0.81	24.57	2.50	22.07
101 - 108	300	3000	120	37.900	0.536	1.25	3.76	25.38	2.50	22.88
102 - 108	300	1940	120	35.730	0.505	1.12	2.18	25.38	2.50	22.88
104 - 109	700	460	110	297.650	0.773	1.08	0.50	25.84	2.50	23.34
106 - 109	250	740	120	20.142	0.410	0.95	0.70	25.84	2.50	23.34
109 - 110	500	720	110	203.656	1.037	2.75	1.98	23.86	2.50	21.36
110 - 111	250	1160	110	28.191	0.574	2.07	2.40	21.46	2.50	18.96
110 - 111	250	1160	120	30.754	0.627	2.07	2.40	21.46	2.50	18.96
107 - 111	200	1120	110	18.355	0.584	2.78	3.11	21.46	2.50	18.96
105 - 112	300	350	120	81.624	1.155	5.19	1.81	24.45	2.50	21.95
109 - 112	400	840	120	93.836	0.747	1.65	1.39	24.45	2.50	21.95
112 - 113	300	1050	120	49.520	0.701	2.06	2.16	22.29	2.50	19.79
110 - 113	300	600	110	51.712	0.732	2.62	1.57	22.29	2.50	19.79
113 - 114	200	980	120	20.507	0.653	2.90	2.84	19.45	2.50	16.95
115 - 114	200	810	120	16.899	0.538	2.03	1.64	19.45	2.50	16.95
110 - 115	300	1470	110	43.299	0.613	1.89	2.77	21.09	2.50	18.59
112 - 116	300	950	120	72.841	1.030	4.20	3.99	20.46	2.50	17.96
117 - 116	200	1160	120	5.031	0.160	0.22	0.25	20.46	2.50	17.96
113 - 117	300	610	110	51.424	0.728	2.59	1.58	20.71	2.50	18.21
117 - 114	200	1230	120	11.694	0.372	1.03	1.26	19.45	2.50	16.95
104 - 121	400	650	120	177.313	1.411	5.37	3.49	22.84	2.50	20.34
116 - 118	250	2030	120	24.771	0.505	1.39	2.82	17.64	2.50	15.14
119 - 118	250	700	120	30.029	0.612	1.98	1.39	17.64	2.50	15.14
120 - 119	250	1090	120	9.456	0.193	0.23	0.25	19.03	2.50	16.53
121 - 120	400	1030	120	139.845	1.113	3.46	3.56	19.28	2.50	16.78
121 - 122	200	1900	120	2.767	0.088	0.07	0.14	22.71	2.50	20.21
101 - 122	350	700	110	152.944	1.590	9.19	6.43	22.71	2.50	20.21
122 - 124	350	1890	110	63.211	0.657	1.79	3.39	19.32	2.50	16.82
124 - 123	350	1570	120	18.211	0.189	0.15	0.24	19.08	2.50	16.58
120 - 123	400	350	120	52.689	0.419	0.57	0.20	19.08	2.50	16.58
105 - 119	300	1380	110	75.273	1.065	5.24	7.24	19.03	2.50	16.53
201 - 154	400	1840	120	46.300	0.368	0.45	0.82	34.18	2.50	31.68

Note :

Node : Supply Source / Discharge point
D : Pipe diameter (mm)
L : Pipe length (m)
C : Friction loss coefficient of Haizen William's formula
Q : Flow in pipe (l/sec)
V : Average flow rate (m/sec)
i : Hydraulic gradient (‰)
dH : Head loss (m)
H : Hydraulic gradient level (m)
GL : Ground height (m)
He : Residual Pressure (m)

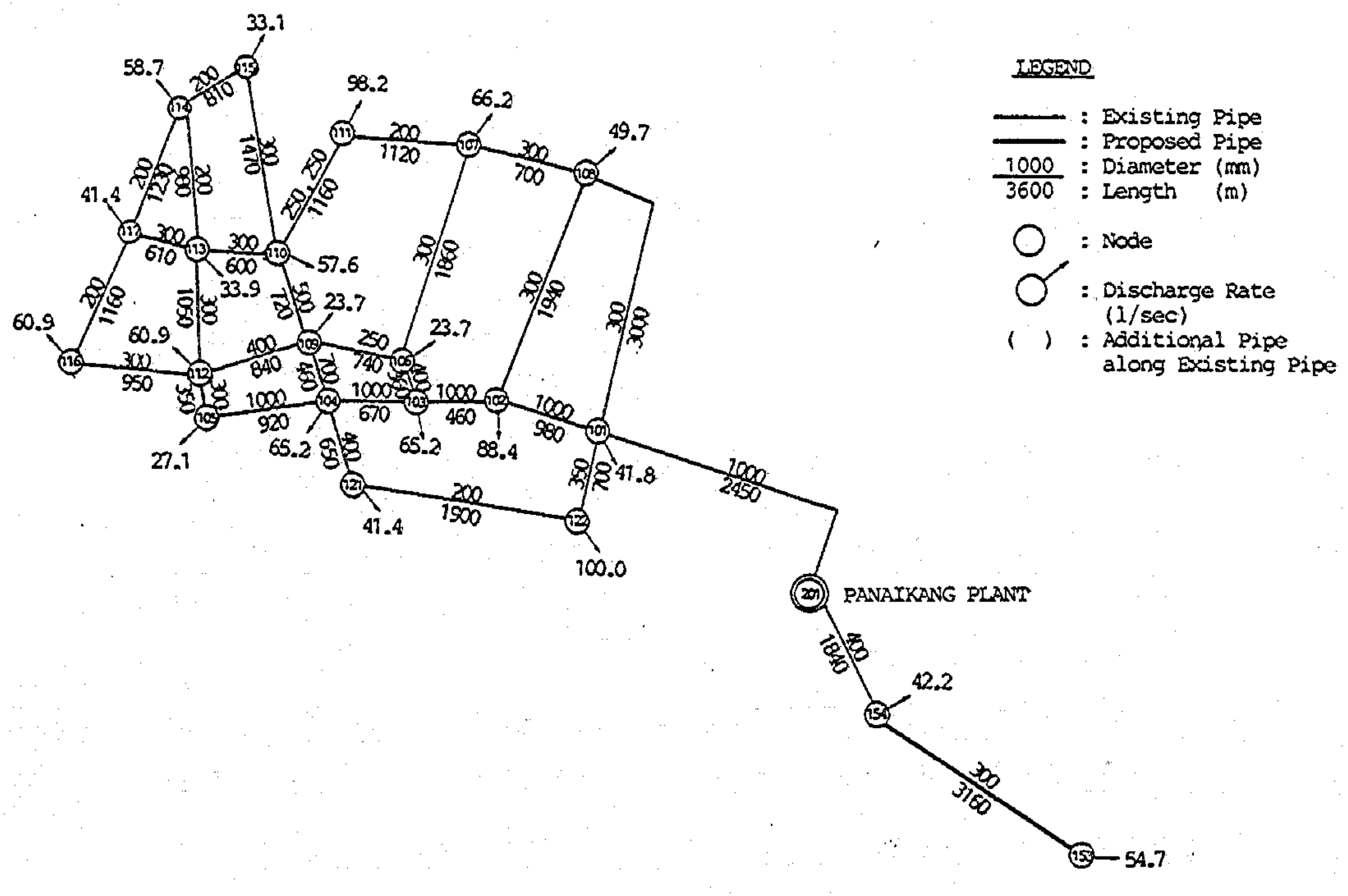


FIGURE	NETWORK DIAGRAM OF PANAIKANG SYSTEM (1995)
6.4	

TABLE 6.2 FLOW IN NETWORK OF MANGNGASA SYSTEM (1995)

Node - Node	D(mm)	L(m)	C	Q(1/sec)	V(m/sec)	i(o/oo)	dH(m)	H(m)	GL(m)	He(m)
202 - 132	1000	1600	120	1149.559	1.464	1.97	3.14	29.86	2.50	27.36
202 - 134	200	3600	120	20.441	0.651	2.88	10.38	22.62	2.50	20.12
132 - 126	900	1340	120	792.966	1.246	1.65	2.21	27.64	2.50	25.14
126 - 127	350	1510	110	65.204	0.678	1.90	2.87	24.78	2.50	22.28
126 - 127	700	1510	120	441.063	1.146	1.90	2.87	24.78	2.50	22.28
127 - 133	350	900	110	57.759	0.600	1.52	1.36	23.41	2.50	20.91
127 - 133	350	900	120	63.009	0.655	1.52	1.36	23.41	2.50	20.91
133 - 123	400	920	120	110.768	0.881	2.25	2.07	21.34	2.50	18.84
123 - 120	400	350	120	116.960	0.931	2.49	0.87	20.47	2.50	17.97
120 - 119	250	1090	120	19.360	0.394	0.88	0.96	19.52	2.50	17.02
127 - 119	350	2240	120	79.820	0.830	2.35	5.26	19.52	2.50	17.02
119 - 118	250	700	120	35.380	0.721	2.68	1.88	17.64	2.50	15.14
127 - 129	500	880	120	185.218	0.943	1.96	1.73	23.05	2.50	20.55
129 - 128	300	490	120	74.681	1.057	4.40	2.16	20.89	2.50	18.39
128 - 118	250	1820	120	28.420	0.579	1.79	3.26	17.64	2.50	15.14
127 - 134	200	1300	120	15.162	0.483	1.66	2.16	22.62	2.50	20.12
134 - 130	200	1400	120	17.603	0.560	2.19	3.06	19.56	2.50	17.06
129 - 130	200	1090	120	21.637	0.689	3.20	3.49	19.56	2.50	17.06
128 - 130	200	1890	120	9.560	0.304	0.71	1.34	19.56	2.50	17.06
132 - 131	600	1750	120	255.293	0.903	1.46	2.56	27.30	2.50	24.80
131 - 155	350	650	110	52.562	0.546	1.27	0.83	26.47	2.50	23.97
131 - 155	500	650	120	146.632	0.747	1.27	0.83	26.47	2.50	23.97
155 - 125	350	950	110	67.093	0.697	2.00	1.90	24.57	2.50	22.07
126 - 125	350	1510	110	67.721	0.704	2.04	3.07	24.57	2.50	22.07
126 - 125	350	1510	120	73.877	0.768	2.04	3.07	24.57	2.50	22.07
125 - 124	350	210	110	72.979	0.759	2.34	0.49	24.08	2.50	21.58
125 - 124	350	210	120	79.613	0.827	2.34	0.49	24.08	2.50	21.58
124 - 123	400	1570	120	96.492	0.768	1.74	2.73	21.34	2.50	18.84

Note :

- Node : Supply Source / Discharge point
- D : Pipe diameter (mm)
- L : Pipe length (m)
- C : Friction loss coefficiient of Haizen William's formula
- Q : Flow in pipe (l/sec)
- v : Average flow rate (m/sec)
- i : Hydraulic gradient (‰)
- dH : Head loss (m)
- H : Hydraulic gradient level (m)
- GL : Ground height (m)
- He : Residual Pressure (m)

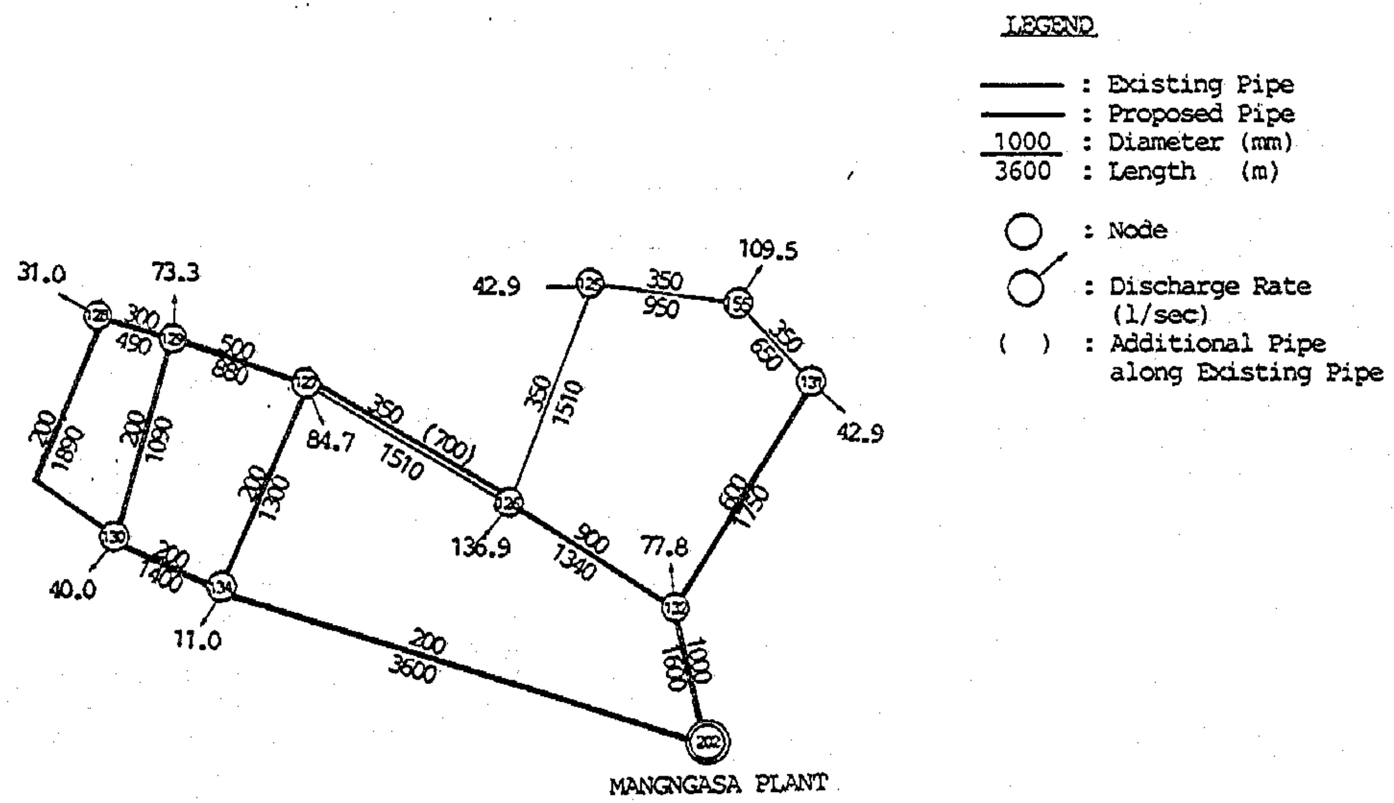


FIGURE	NETWORK DIAGRAM OF MANGNGASA SYSTEM (1992)
6.5	

TABLE 6.3 FLOW IN NETWORK OF MANGNGASA SYSTEM (1992)

Node - Node	D(mm)	L(m)	C	Q(1/sec)	V(m/sec)	i(o/oo)	dH(m)	H(m)	GL(m)	He(m)
202 - 132	1000	1600	120	638.333	0.813	0.66	1.06	31.94	2.50	29.44
202 - 134	200	3600	120	11.667	0.371	1.02	3.68	29.32	2.50	26.82
132 - 126	900	1340	120	365.233	0.574	0.39	0.53	31.41	2.50	28.91
126 - 127	350	1510	110	29.408	0.306	0.43	0.66	30.76	2.50	28.26
126 - 127	700	1510	120	198.925	0.517	0.43	0.66	30.76	2.50	28.26
127 - 129	500	880	120	131.468	0.670	1.04	0.92	29.84	2.50	27.34
129 - 128	300	490	120	41.206	0.583	1.46	0.72	29.12	2.50	26.62
127 - 134	200	1300	120	12.165	0.387	1.10	1.44	29.32	2.50	26.82
134 - 130	200	1400	120	12.832	0.408	1.22	1.71	27.62	2.50	25.12
129 - 130	200	1090	120	16.961	0.540	2.04	2.23	27.62	2.50	25.12
128 - 130	200	1890	120	10.206	0.325	0.80	1.51	27.62	2.50	25.12
132 - 131	600	1750	120	195.300	0.691	0.89	1.56	30.38	2.50	27.88
131 - 155	350	650	110	152.400	1.584	9.13	5.93	24.45	2.50	21.95
155 - 125	350	950	110	42.900	0.446	0.87	0.83	23.62	2.50	21.12

Note :

- Node : Supply Source / Discharge point
- D : Pipe diameter (mm)
- L : Pipe length (m)
- C : Friction loss coefficiient of Haizen William's formula
- Q : Flow in pipe (l/sec)
- v : Average flow rate (m/sec)
- i : Hydraulic gradient (‰)
- dH : Head loss (m)
- H : Hydraulic gradient level (m)
- GL : Ground height (m)
- He : Residual Pressure (m)

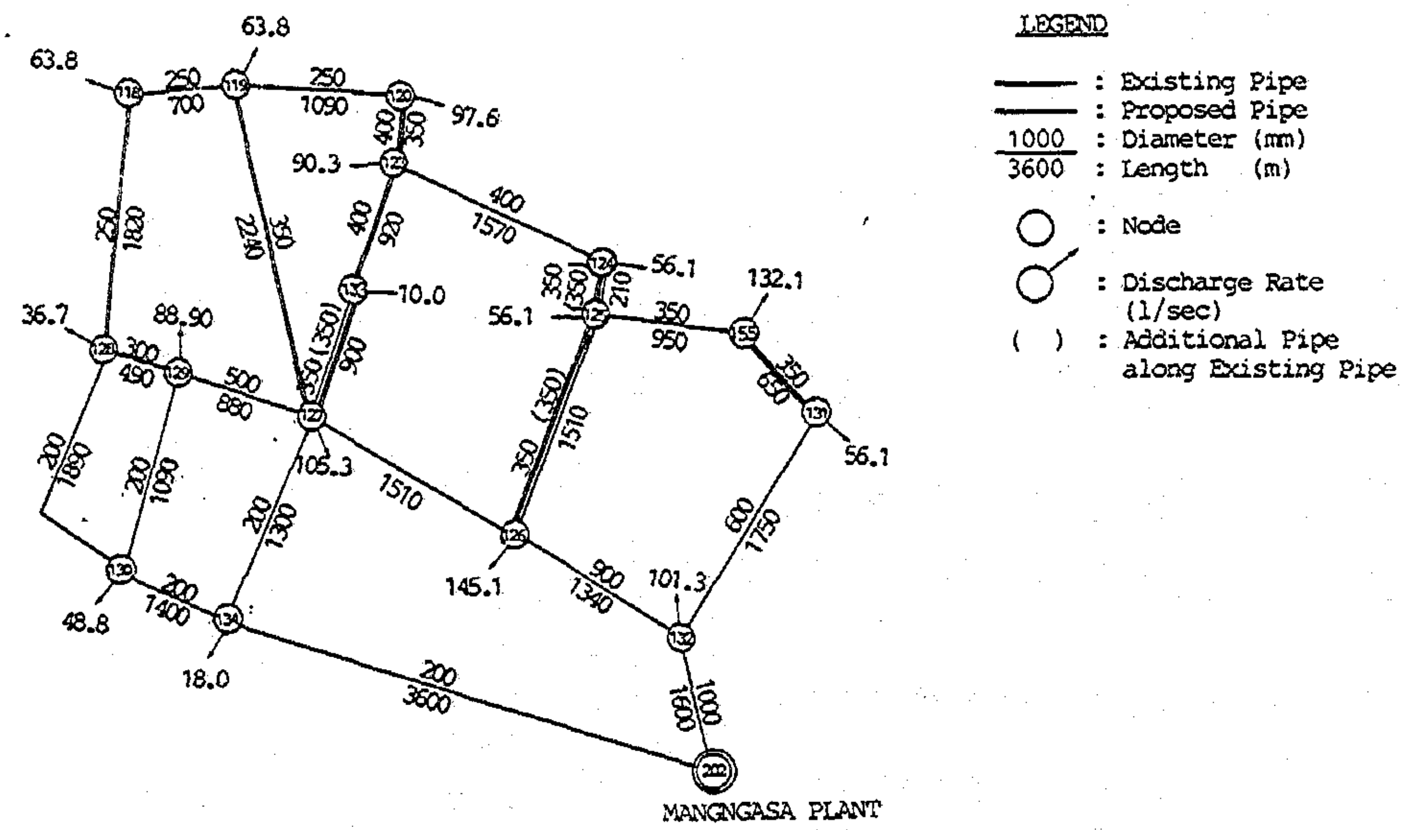


FIGURE	NETWORK DIAGRAM OF MANGNGASA SYSTEM (1995)
6.6	

TABLE 6.4 FLOW IN NETWORK OF PANAIKANG SYSTEM (1995)

Node - Node	D(mm)	L(m)	C	Q(1/sec)	V(m/sec)	i(o/oo)	dH(m)	H(m)	GL(m)	He(m)
201 - 101	1000	2450	110	1037.100	1.320	1.91	4.68	30.32	2.50	27.82
101 - 102	1000	980	110	855.230	1.089	1.34	1.31	29.01	2.50	26.51
102 - 103	1000	460	110	719.610	0.916	0.97	0.45	28.57	2.50	26.07
103 - 104	1000	670	110	569.728	0.725	0.63	0.42	28.15	2.50	25.65
104 - 105	1000	920	110	115.679	0.147	0.03	0.03	28.12	2.50	25.62
103 - 106	400	360	110	84.682	0.674	1.61	0.58	27.99	2.50	25.49
106 - 107	300	1860	110	44.384	0.628	1.97	3.67	24.32	2.50	21.82
108 - 107	300	700	120	41.541	0.588	1.49	1.04	24.32	2.50	21.82
101 - 108	300	3000	120	44.021	0.623	1.65	4.96	25.36	2.50	22.86
102 - 108	300	1940	120	47.220	0.668	1.88	3.65	25.36	2.50	22.86
104 - 109	700	460	110	343.498	0.893	1.40	0.65	27.50	2.50	25.00
106 - 109	250	740	120	16.597	0.338	0.66	0.49	27.50	2.50	25.00
109 - 110	500	720	110	238.711	1.216	3.69	2.65	24.85	2.50	22.35
110 - 111	250	1160	110	37.531	0.765	3.52'	4.08	20.77	2.50	18.27
110 - 111	250	1160	120	40.943	0.834	3.52	4.08	20.77	2.50	18.27
107 - 111	200	1120	110	19.725	0.628	3.17	3.56	20.77	2.50	18.27
105 - 112	300	350	120	88.579	1.253	6.03	2.11	26.00	2.50	23.50
109 - 112	400	840	120	97.685	0.777	1.78	1.50	26.00	2.50	23.50
112 - 113	300	1050	120	56.289	0.796	2.61	2.74	23.27	2.50	20.77
110 - 113	300	600	110	51.879	0.734	2.63	1.58	23.27	2.50	20.77
113 - 114	200	980	120	24.395	0.777	4.00	3.92	19.35	2.50	16.85
115 - 114	200	810	120	17.657	0.562	2.20	1.78	19.35	2.50	16.85
110 - 115	300	1470	110	50.757	0.718	2.53	3.72	21.13	2.50	18.63
112 - 116	300	950	120	69.075	0.977	3.81	3.62	22.39	2.50	19.89
116 - 117	200	1160	120	8.175	0.260	0.53	0.61	21.77	2.50	19.27
113 - 117	300	610	110	49.872	0.706	2.45	1.49	21.77	2.50	19.27
117 - 114	200	1230	120	16.647	0.530	1.97	2.43	19.35	2.50	16.85
104 - 121	400	650	120	45.351	0.361	0.43	0.28	27.87	2.50	25.37
121 - 122	200	1900	120	3.951	0.126	0.14	0.26	27.60	2.50	25.10
101 - 122	350	700	110	96.049	0.998	3.89	2.72	27.60	2.50	25.10
201 - 154	400	1840	120	96.900	0.771	1.75	3.23	31.77	2.50	29.27
154 - 153	300	3160	120	54.700	0.774	2.47	7.81	23.96	2.50	21.46

Note :

Node : Supply Source / Discharge point
D : Pipe diameter (mm)
L : Pipe length (m)
C : Friction loss coefficiient of Haizen William's formula
Q : Flow in pipe (l/sec)
V : Average flow rate (m/sec)
i : Hydraulic gradient (‰)
dH : Head loss (m)
H : Hydraulic gradient level (m)
GL : Ground height (m)
He : Residual Pressure (m)

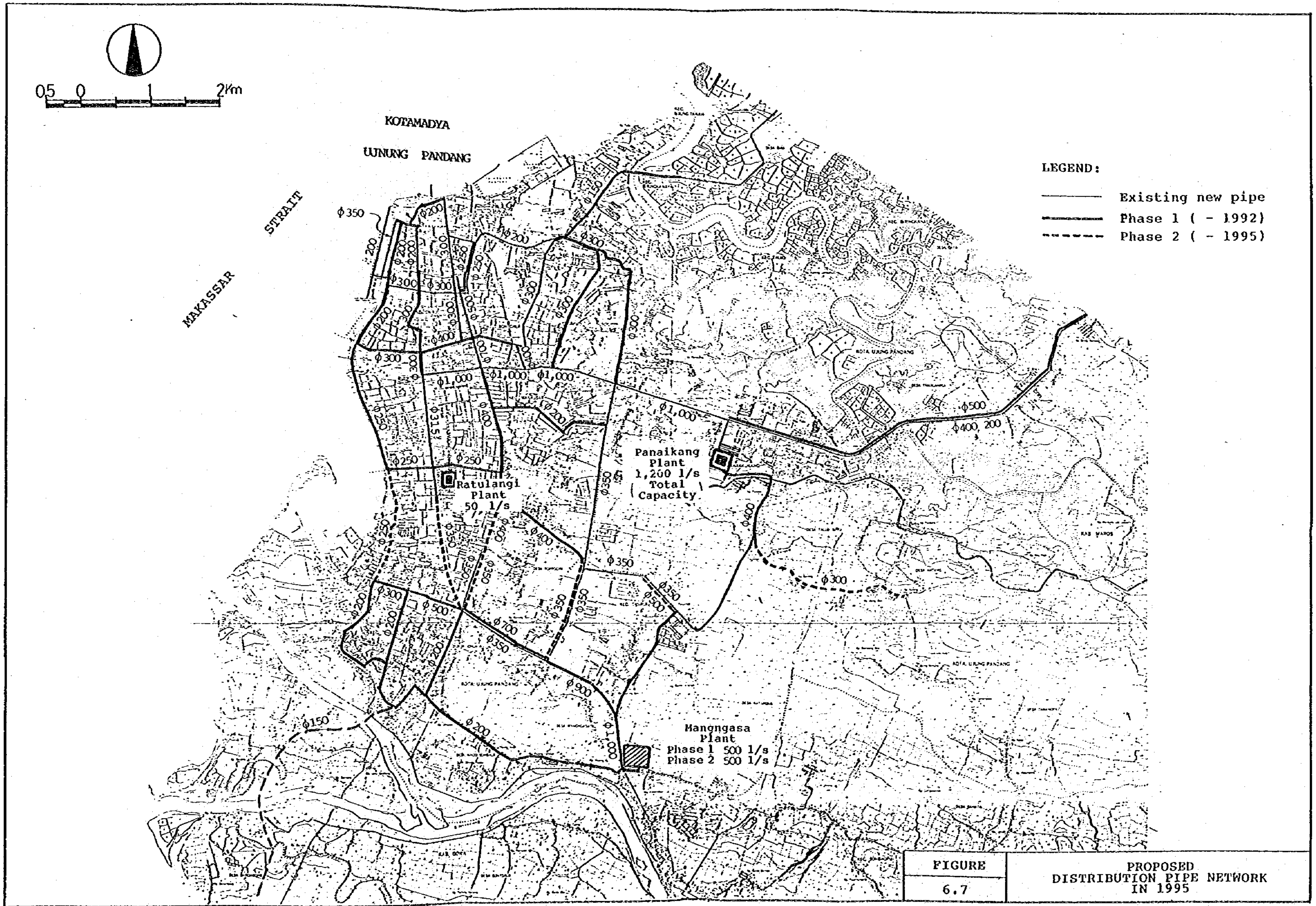


FIGURE 6.7

PROPOSED DISTRIBUTION PIPE NETWORK IN 1995

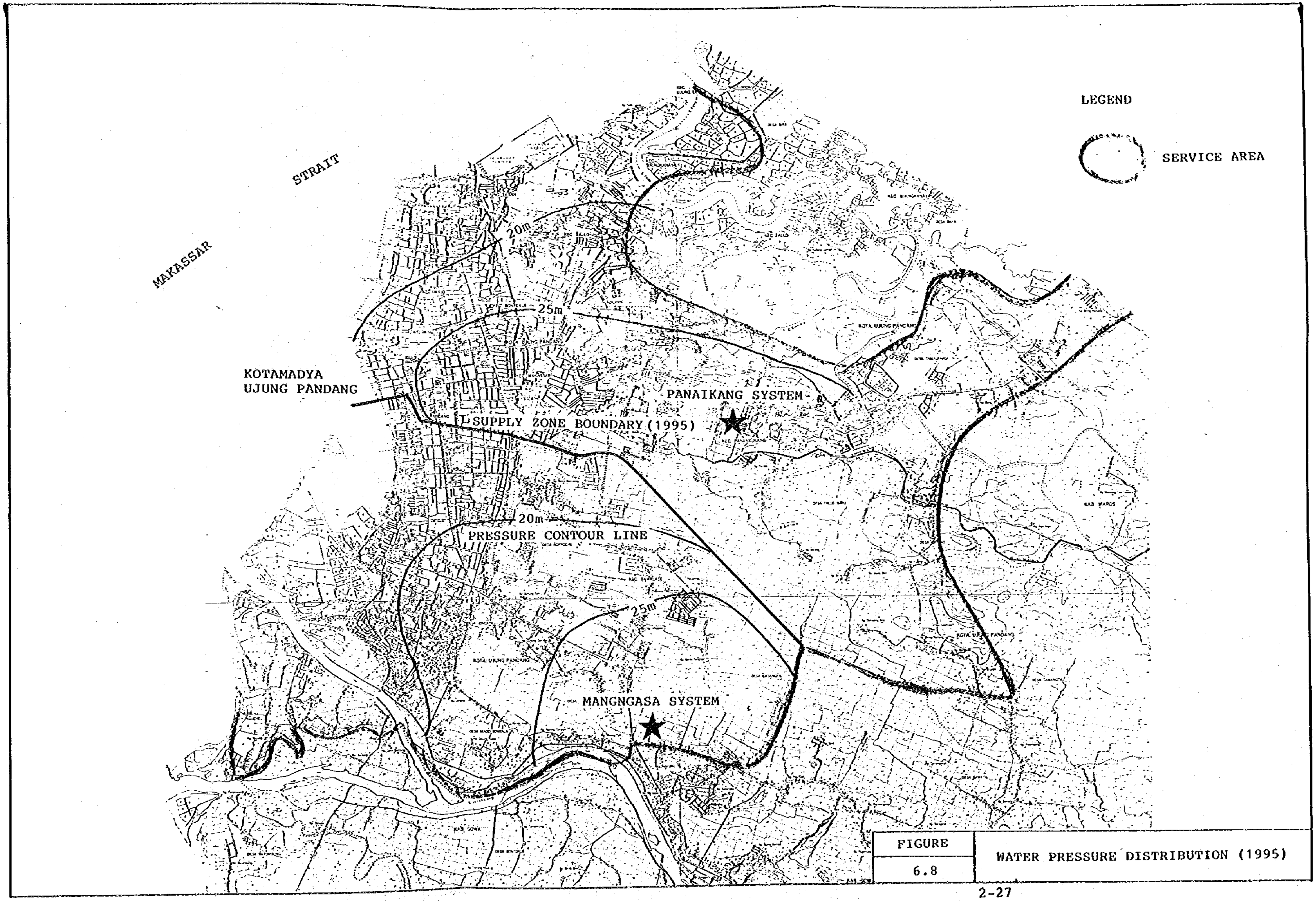


FIGURE	WATER PRESSURE DISTRIBUTION (1995)
6.8	

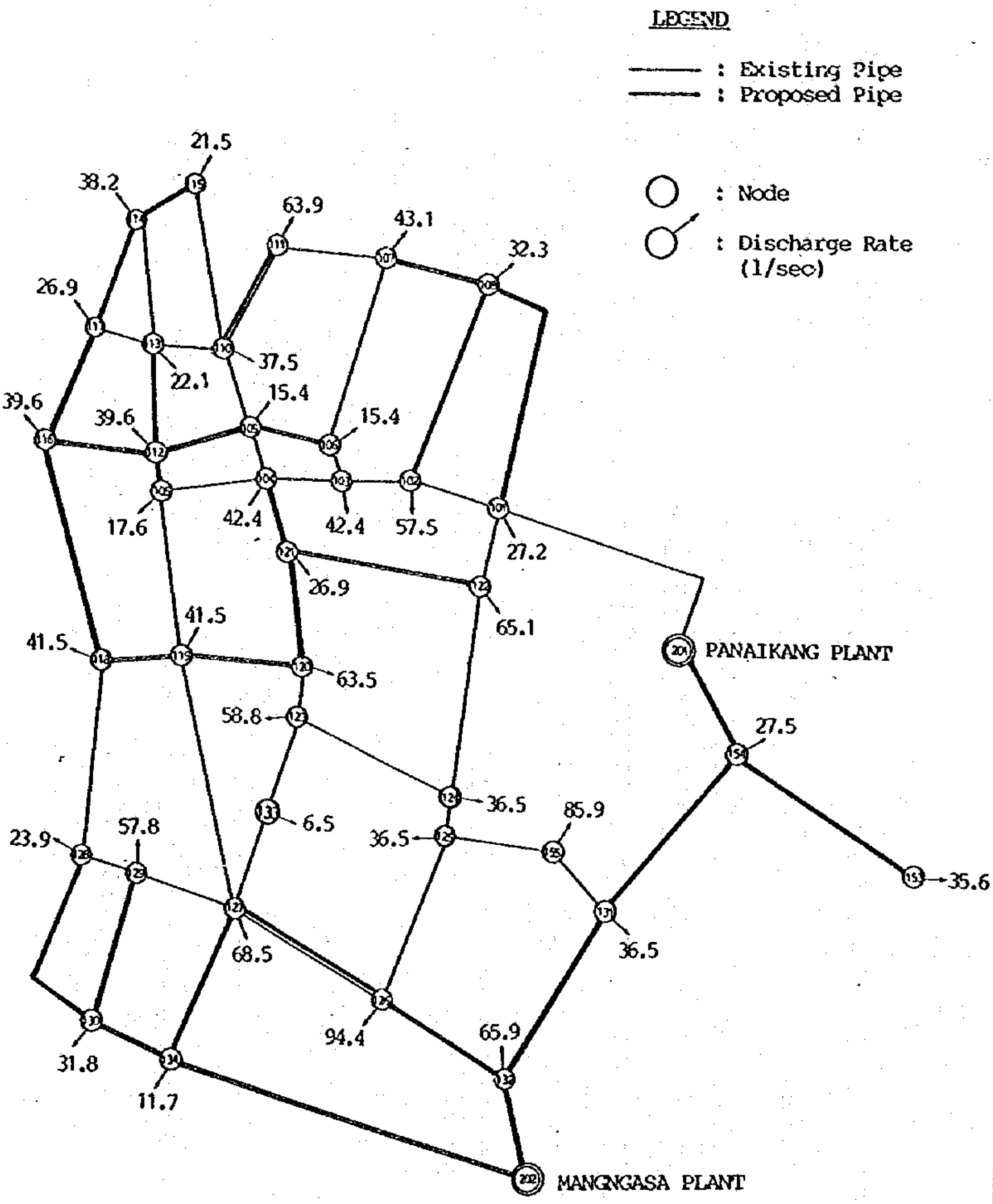


FIGURE	NETWORK DIAGRAM UNDER EMERGENCY OPERATION IN SEVERE DRY SEASON
6.9	

TABLE 6.5 FLOW IN NETWORK IN CASE OF SEVERE DRY SEASON

Node - Node	D(mm)	L(m)	C	Q(l/sec)	V(m/sec)	i(o/oo)	dH(m)	H(m)	GL(m)	He(m)
201 - 101	1000	2450	110	530.863	0.676	0.55	1.35	22.55	2.50	20.05
101 - 102	1000	980	110	462.631	0.589	0.43	0.42	22.13	2.50	19.63
102 - 103	1000	460	110	374.343	0.477	0.29	0.13	22.00	2.50	19.50
103 - 104	1000	670	110	278.664	0.355	0.17	0.11	21.89	2.50	19.39
104 - 105	1000	920	110	52.692	0.067	0.01	0.01	21.88	2.50	19.38
103 - 106	400	360	110	53.260	0.424	0.68	0.25	21.75	2.50	19.25
106 - 107	300	1860	110	29.384	0.416	0.92	1.71	20.04	2.50	17.54
108 - 107	300	700	120	25.915	0.367	0.62	0.43	20.04	2.50	17.54
101 - 108	300	3000	120	27.475	0.389	0.69	2.08	20.48	2.50	17.98
102 - 108	300	1940	120	30.774	0.435	0.85	1.66	20.48	2.50	17.98
104 - 109	700	460	110	215.865	0.561	0.59	0.27	21.61	2.50	19.11
106 - 109	250	740	120	8.456	0.172	0.19	0.14	21.61	2.50	19.11
109 - 110	500	720	110	152.276	0.776	1.60	1.16	20.46	2.50	17.96
110 - 111	250	1160	110	24.708	0.503	1.62	1.88	18.58	2.50	16.08
110 - 111	250	1160	120	26.954	0.549	1.62	1.88	18.58	2.50	16.08
107 - 111	200	1120	110	12.228	0.389	1.31	1.47	18.58	2.50	16.08
105 - 112	300	350	120	52.840	0.748	2.32	0.81	21.07	2.50	18.57
109 - 112	400	840	120	56.626	0.451	0.65	0.55	21.07	2.50	18.57
112 - 113	300	1050	120	36.165	0.512	1.15	1.21	19.86	2.50	17.36
110 - 113	300	600	110	30.684	0.434	1.00	0.60	19.86	2.50	17.36
113 - 114	200	980	120	15.824	0.504	1.80	1.76	18.10	2.50	15.60
115 - 114	200	810	120	10.920	0.348	0.90	0.73	18.10	2.50	15.60
110 - 115	300	1470	110	32.455	0.459	1.11	1.63	18.83	2.50	16.33
112 - 116	300	950	120	33.679	0.476	1.01	0.96	20.11	2.50	17.61
116 - 117	200	1160	120	9.412	0.300	0.69	0.80	19.31	2.50	16.81
113 - 117	300	610	110	28.969	0.410	0.90	0.55	19.31	2.50	16.81
117 - 114	200	1230	120	11.446	0.364	0.99	1.21	18.10	2.50	15.60
121 - 104	400	650	120	32.313	0.257	0.23	0.15	21.89	2.50	19.39
122 - 121	200	1900	120	5.252	0.167	0.23	0.44	22.04	2.50	19.54
101 - 122	350	700	110	13.561	0.141	0.10	0.07	22.48	2.50	19.98
154 - 201	400	1840	120	38.863	0.309	0.32	0.60	23.91	17.00	6.91
154 - 153	300	3160	120	35.588	0.503	1.12	3.53	20.97	2.50	18.47
202 - 132	1000	1600	120	991.015	1.262	1.49	2.39	30.61	2.50	28.11
202 - 134	200	3600	120	15.986	0.509	1.83	6.59	26.41	2.50	23.91
132 - 126	900	1340	120	645.571	1.015	1.13	1.51	29.10	2.50	26.60
126 - 127	350	1510	110	52.114	0.542	1.25	1.89	27.20	2.50	24.70
126 - 127	700	1510	120	352.518	0.916	1.25	1.89	27.20	2.50	24.70
127 - 133	350	900	110	57.630	0.599	1.51	1.36	25.84	2.50	23.34
127 - 133	350	900	120	62.869	0.653	1.51	1.36	25.84	2.50	23.34
133 - 123	400	920	120	113.993	0.907	2.37	2.18	23.66	2.50	21.16
123 - 120	400	350	120	127.214	1.012	2.90	1.02	22.65	2.50	20.15
120 - 119	250	1090	120	9.719	0.198	0.25	0.27	22.38	2.50	19.88
127 - 119	350	2240	120	76.161	0.792	2.15	4.82	22.38	2.50	19.88
119 - 118	250	700	120	26.592	0.542	1.58	1.11	21.27	2.50	18.77
127 - 129	500	880	120	130.645	0.665	1.03	0.91	26.30	2.50	23.80
129 - 128	300	490	120	58.346	0.825	2.79	1.37	24.93	2.50	22.43
128 - 118	250	1820	120	30.272	0.617	2.01	3.66	21.27	2.50	18.77
127 - 134	200	1300	120	8.818	0.281	0.61	0.79	26.41	2.50	23.91
134 - 130	200	1400	120	13.093	0.417	1.26	1.77	24.64	2.50	22.14
129 - 130	200	1090	120	14.460	0.460	1.52	1.66	24.64	2.50	22.14
128 - 130	200	1890	120	4.197	0.134	0.15	0.29	24.64	2.50	22.14
132 - 131	600	1750	120	279.537	0.989	1.73	3.03	27.58	2.50	25.08
131 - 155	350	650	110	37.241	0.387	0.67	0.44	27.15	2.50	24.65
131 - 155	500	650	120	103.890	0.529	0.67	0.44	27.15	2.50	24.65
155 - 125	350	960	110	55.186	0.574	1.39	1.32	25.82	2.50	23.32
126 - 125	350	1510	110	70.082	0.728	2.17	3.27	25.82	2.50	23.32
126 - 125	350	1510	120	76.454	0.795	2.17	3.27	25.82	2.50	23.32
125 - 124	350	210	110	79.019	0.821	2.71	0.57	25.25	2.50	22.75
125 - 124	350	210	120	86.203	0.896	2.71	0.57	25.25	2.50	22.75
124 - 123	400	1570	120	71.971	0.573	1.01	1.59	23.66	2.50	21.16
118 - 116	250	2030	120	15.356	0.313	0.57	1.16	20.11	2.50	17.61
119 - 105	300	1380	110	17.779	0.252	0.36	0.50	21.88	2.50	19.38
120 - 121	400	1030	120	53.936	0.430	0.59	0.61	22.04	2.50	19.54
124 - 122	350	1890	110	56.752	0.590	1.47	2.77	22.48	2.50	19.98
131 - 154	400	1600	120	101.907	0.811	1.93	3.08	24.50	2.50	22.00

therefore recommended to provide a distribution tower on the route of distribution pipe of the Panaikang water supply system as a countermeasure to the power suspension. This measure to supply by the tower is almost maintenance-free and very reliable compared with that by pump and engine.

The following advantages are derived from the provision of the distribution tower.

i) The low water level of the existing distribution reservoir of the Panaikang plant is EL.+14m and the urban service area is flat with average ground level of +2.5m above the sea. Hence, in the case of the power suspension, the water is distributed to the area around the treatment plant from the distribution reservoir of the plant by the gravity flow and the clear water stored in the distribution tower is distributed also by the gravity to the downstream of the tower.

ii) The tower functions not only as a key supply source to downtown of the City in power failure, but also as a regulating reservoir for hourly fluctuation, including fire fighting water, in water demand at other times.

iii) In case the main transmission pipe is damaged by accident, the interconnection valves of the trunk mains connecting two zones are opened so that the clear water be supplied to the zone where water is unavailable due to the accident. The distribution tower plays a vital role to supply clear water while such operation of the opening of valve is performed.

It is considered most adequate to provide the distribution tower at the Karebosi square, taking into account following points to maximize the above advantages:

i) The location is favorable from the hydraulic point of view, and the existing pipeline can be effectively utilized.

ii) The location is well known by the residents and can be seen easily from the surroundings and used preferably as a place of refuge in case of emergency.

iii) In the vicinity of the Karebosi square, there are many municipal offices and Sulawesi provincial offices as well as banks and business offices. Therefore the surroundings of the square is considered as the nucleus of the administrative and financial activities and the most important area for the fire protection.

The capacity of the distribution tower is designed to be 3,000 m3 equivalent to distribution flow for one hour to be directed to the urban area of Panaikang system.

The high water level of this tower is designed to be +27 m above the sea taking into account the results of pipe network calculation and above mentioned location of the tower. The prestressed concrete structure is preferable for the distribution tower owing to its height.

III. OPERATION AND MAINTENANCE

1. Introduction

The principal objective of the water supply works is to supply sufficiently and constantly the clear and safe water of reasonable price to the consumers. In order to achieve above objective, all the water supply facilities should function satisfactorily and be operated and managed with due emphasis on economy.

In this study, the existing situation of the water supply system is reviewed taking into account the data collected as a result of the field survey, hearings including operation records, and an improvement plan is recommended.

On the other hand, the basic methodology and plan for the maintenance and operation of the water supply system is prepared with due consideration on above improvement plan and future system development plan. The flow of the work above mentioned is indicated in Figure 3.1.

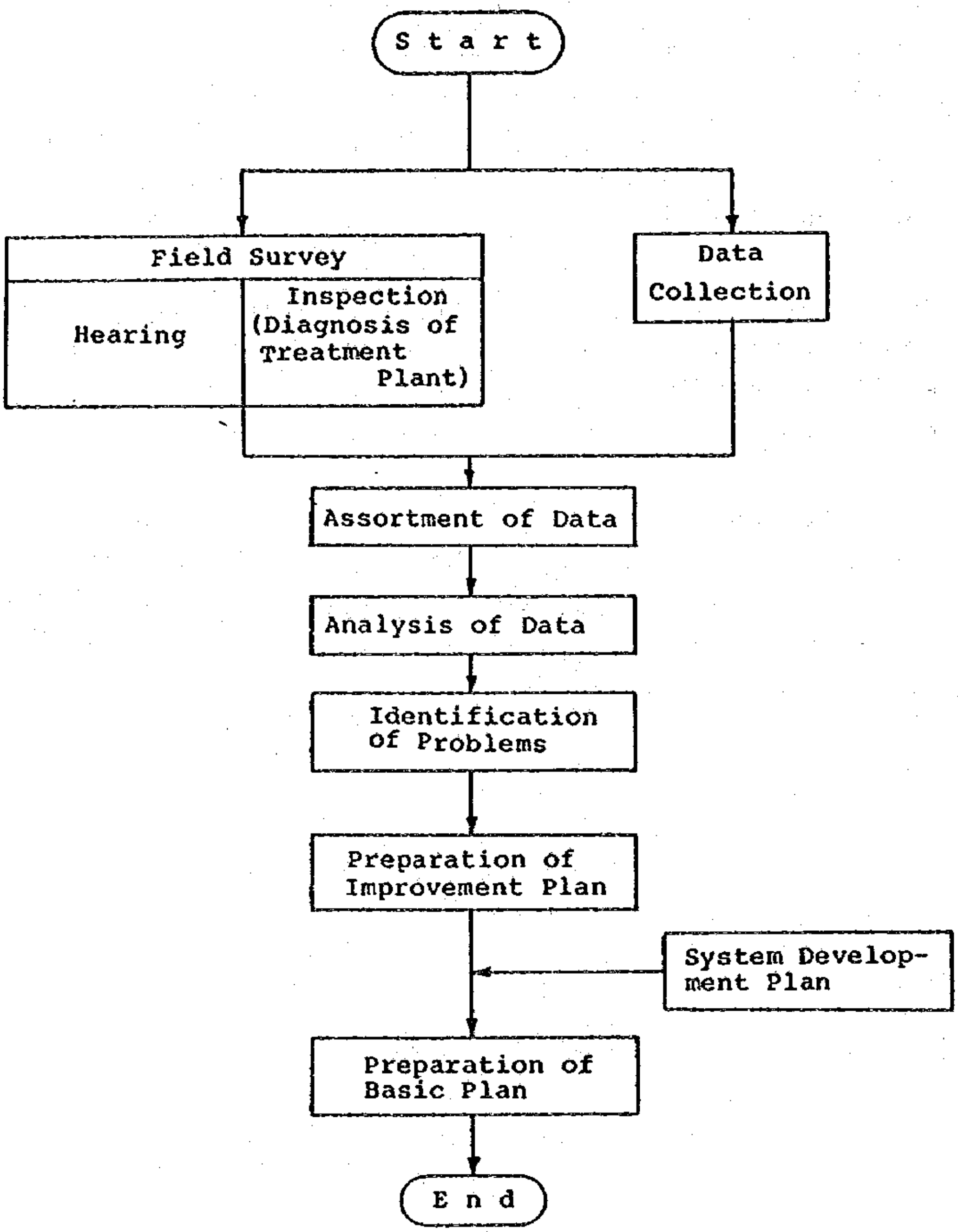


FIGURE 1.1	FLOW CHART OF PREPARATION OF BASIC PLAN FOR OPERATION AND MAINTENANCE

2. DIAGNOSIS ON THE EXISTING TREATMENT SYSTEM

The current condition of operation and maintenance of the two existing treatment plants is reviewed by means of field survey, hearing and collection of relevant records. Based on the findings thus obtained, improvement works, where considered necessary, is proposed for inclusion thereof in the project.

2.1 Ratulangi Treatment System

The Ratulangi plant was constructed and started its operation in 1920's, and the existing facilities of the plant have been deteriorated very much through 60 years operations. Table 2.1 summarizes inappropriate conditions of facilities prevailing in the Ratulangi treatment plant, and proposes necessary countermeasures to be undertaken.

As for quality fo treated water, it is in conformity with the Indonesian Drinking Water Standard, as shown in the Table 2.2 The Result of Treated Water Quality. However, to ensure production of safe water, measures as suggested above are to be taken.

Regarding chemical feeding for an example, from the actual consumption, dosage of aluminum sulfate is 35 mg/l and bleaching powder is 2.3 mg/l in average. Judging from the result of water quality analysis, most optimum dosage rate is 25 mg/l of aluminum sulfate and 3.3 mg/l of bleaching powder.

2.2 Panaikang Treatment System

The problems on water pollution of the Maros transmission canal, and carry-over in sedimentation basin(pulsator) at Panaikang plant were studied in detail in the master plan study. Further detailed study was conducted during this feasiblity study period.

The present problems identified by the study are summarized in Table 2.3. Necessary countermeasures are also shown in the same table.

In spite of problems mentioned in Table 2.3, operation of the plant has been well made to produce treated water of quality in conformity of the drinking water standards of the country.

The average chemical dosage rates, estimated from chemicals consumed in 1984, are shown in Table 2.5. As for chlorine, it is necessary to strengthen pre-chlorination to prevent algae growth in the treatment facilities. As a result of experiments on `chlorine consumption of the raw water, 1.0 mg/l pre-chlorine is advisable. Further, to prevent corrosion of distribution pipes, control of pH by dosing alkali is also recommendable.

2.3 Water Quality at Service Tap

Water quality survey at service connections and taps in service area was conducted in order to check quality of supplied water covering the parameters of residual chlorine, coliform and bacteria. The result of survey is summarized as follows:

1) Analysis shows no coliform group detected in water sampled from connections and service taps. This means the treated water is not polluted and hygienicaly safe.

2) The contour map of residual chlorine shows that the residual chlorine in the service area supplied by old distribution pipelines is less than o.1 mg/l, and from this, chlorine is considered being consumed due to incrustation in the pipelines. (Figure 2.2)

3) Suction of water from water mains has been practiced by many consumers in the service area where water pressure is low, and this will cause the negative pressure in the pipeline sucking sewage into the pipe when ground water level is high during the wet season. This practice is to be discouraged from the hygienical point of view.

4) In some other areas where water flow is very scarce during day time, consumers are used to store water in a small water tank during night time, for use in the following day. Water storage is not advisable, because safety of water is not insured, since no residual chlorine was detected in the store water.

TABLE 2.1 Existing Problems and Countermeasures in Ratulangi Treatment Plant

	Problems	Countermeasures
1.	The existing intake facilities are located at the right bank of the Jeneberang river. In the dry season when river flow is scarce, water flows in the middle or left portion of the river, and due to this, water must be led to the intake well point by ditch. This causes increase of labor cost.	It will cost more if water flow is changed artificially. No other solution than the current ditch.
2.	No measuring devices are equipped at intake, transmission pipe and treatment plant, and measurement of water quantity has not been practiced.	Provision of weir is required for measuring quantity of water.
3.	Filtration bed of the filter is not well maintained well, and in some portion cracks exist. Filtration is not properly done.	Replacement of filter sand in all filters is required.
4.	Chemical dosing facilities including flocculation, sedimentation and chlorination are not maintained properly. Especially, chlorination has not been practiced properly. Due to this, residual chlorine in the treated water fluctuates, sometimes indicates 0 mg/l which is critical from hygienical point of view.	Replacement of the existing chlorination facilities is required.
5.	Water quality test and examination has not been practiced in the plant.	It is recommended to strengthen the system of water quality test and analysis, conducting at least once a month covering the parameters such as residual chlorine and biological test.

TABLE 2.2 WATER QUALITY ANALYSIS OF RATURANGI TREATMENT PLANT

QUALITY PARAMETER	20.08.1984		05.09.1984		11.10.1984		12.12.1984		04.06.1985	
	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.
WATER TEMPERATURE	27.7	29.3	27.5	29.0	30.0	29.5	27.0	26.8	27.5	28.3
pH	7.6	7.2	7.2	7.2	7.4	7.4	6.8	7.0	7.2	7.0
TURBIDITY	32	4	155	4	25	3	73	3	23	4
ALKALINITY	84	64	46	44	64	68	120	70	58	48
TOTAL HARDNESS	3.9	-	2.7	3.0	3.6	4.0	7.7	5.6	3.5	3.5
CHLORIDE ION	-	-	-	-	7	7	-	-	7	8
AMMONIA	0.06	0	0.10	0	0.06	0	0.06	0	0.04	0
DISSOLVED IRON	0.15	0.05	0.30	0.05	0.25	0.10	0.25	0.10	0.05	0
MANGANESE	0.03	0	0.03	0	0.03	0	0.03	0	0	0
COLIFORM GROUP	200	0	500	0	1,200	0	1,100	0	600	0
TOTAL COLONIES	2,060	1	850	0	696	7	2,016	6	574	0
C O D	-	-	-	-	-	-	2.0	1.0	2.1	0.9
RESIDUAL CHLORINE	-	0.4	-	0.3	-	0.2	-	0.2	-	0.1

Note: R.W. : Raw Water
T.W. : Treated Water

TABLE 2.3 Existing Problems and Countermeasures in Panaikang Treatment Plant

	Problems	Countermeasures
1.	Leakage and pollution (by domestic sewer) of Maros transmission canal, especially downstream 4 km span.	To replace the open canal with new pipeline.
2.	Carry-over is taking place in sedimentation basin (pulsator); especially it occures along the rise of temperature.	No other way than increase of operation of back wash.
3.	Filtration capacity drops due to algae growth on the surface of filter media.	Surface layer of filter media must be replaced after cleaning. Pre-chlorination should be increased (dosing from 0.2 mg/l to 1.0 mg/l).
4.	Sludge is accumulated in clear water reservoir.	Clear water reservoir should be kept clean.
5.	Post alkalinity treatment has not been done because the facility is out of order. It is anticipated that pipe corrosion would occur due to this.	The existing facility must be repaired or new facility must be provided with extension program, and dosage of alkalinity of twice as much as the current dosage rate.
6.	Dust and odor are serious in solution tank of bleaching powder; it may affect labourers' health.	Good ventilation must be maintained.
7.	Water loss is much in treatment plant because of chracteristics of pulsator and frequent backwash.	To provide a wastewater basin to separate supernatant and sludge. Supernatant is to be discharged into the transmission canal for reuse.
8.	Workers enter daily into the sedimentation basins for cleaning thereof.	To stop cleaning in the sedimentation basin.
9.	When electric power supply stopped, the plant does not function at all.	To construct a distribution tank to supply water.

TABLE 2.4 WATER QUALITY ANALYSIS OF PANAIKANG TREATMENT PLANT

QUALITY PARAMETER	21.08.1984		03.09.1984		01.10.1984		27.11.1984		24.12.1984		19.01.1985		01.06.1985		19.06.1985	
	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W.	R.W.	T.W	R.W.	T.W.	R.W.	T.W.
WATER TEMPERATURE	25.5	26.5	26.5	27.0	28.0	28.5	28.7	28.9	25.5	27.5	25.5	26.0	26.0	27.9	26.8	28.1
pH	7.8	7.2	7.8	7.2	7.8	7.2	7.6	7.0	7.4	6.8	7.4	7.0	7.8	7.2	7.8	7.2
TURBIDITY	22	2	27	3	55	3	55	3	29	3	20	3	26	2	44	2
ALKALINITY	64	50	60	52	72	60	64	56	60	48	56	48	64	56	62	52
TOTAL HARDNESS	3.2	-	3.0	3.1	3.8	3.8	3.9	4.0	3.4	3.4	3.2	3.1	3.6	3.6	3.6	3.6
CHLORIDE ION	-	-	2	-	2	2	2	-	3	-	-	-	2	3	2	-
AMMONIA	0.08	0	0.08	0	0.06	0	0.06	0	0.08	0	0.06	0	0.06	0	0.06	0
DISSOLVED IRON	0.30	0.05	0.20	0.05	0.25	0.10	0.10	0.10	0.15	0.10	0.20	0.10	0.05	0	0.25	0.10
MANGANESE	0.05	0.03	0.03	0.03	0.03	0	0.03	0	0	0	0	0	0	0	0	0
COLIFORM GROUP	1,800	0	200	0	6,200	0	1,400	0	1,500	0	1,600	0	1,100	0	1,400	0
TOTAL COLONIES	525	0	350	0	840	2	1,176	0	1,260	0	798	0	714	0	644	2
C O D	-	-	11	-	-	-	1.3	0.2	1.1	0.7	0.8	0.4	1.3	0.6	2.4	0.5
RESIDUAL CHLORINE	-	0.8	-	0.4	-	0.05	-	0.6	-	0.3	-	0.6	-	0.6	-	0.5

Note: R.W. : Raw Water
T.W. : Treated Water

TABLE 2.5 DOSAGE RATE OF PANAIKANG TREATMENT PLANT

Unit ; mg/l

Description	Pre-Chlorination	Alum	Post-Arkali	Post-Chlorination
1984	0.12 (0.2)	25	0	0.9 (1.5)
After Rehabilitation	1 (1.7)	25	5	1 (1.7)

Note : Figures in ( ) shows the dosage rate of bleaching powder.

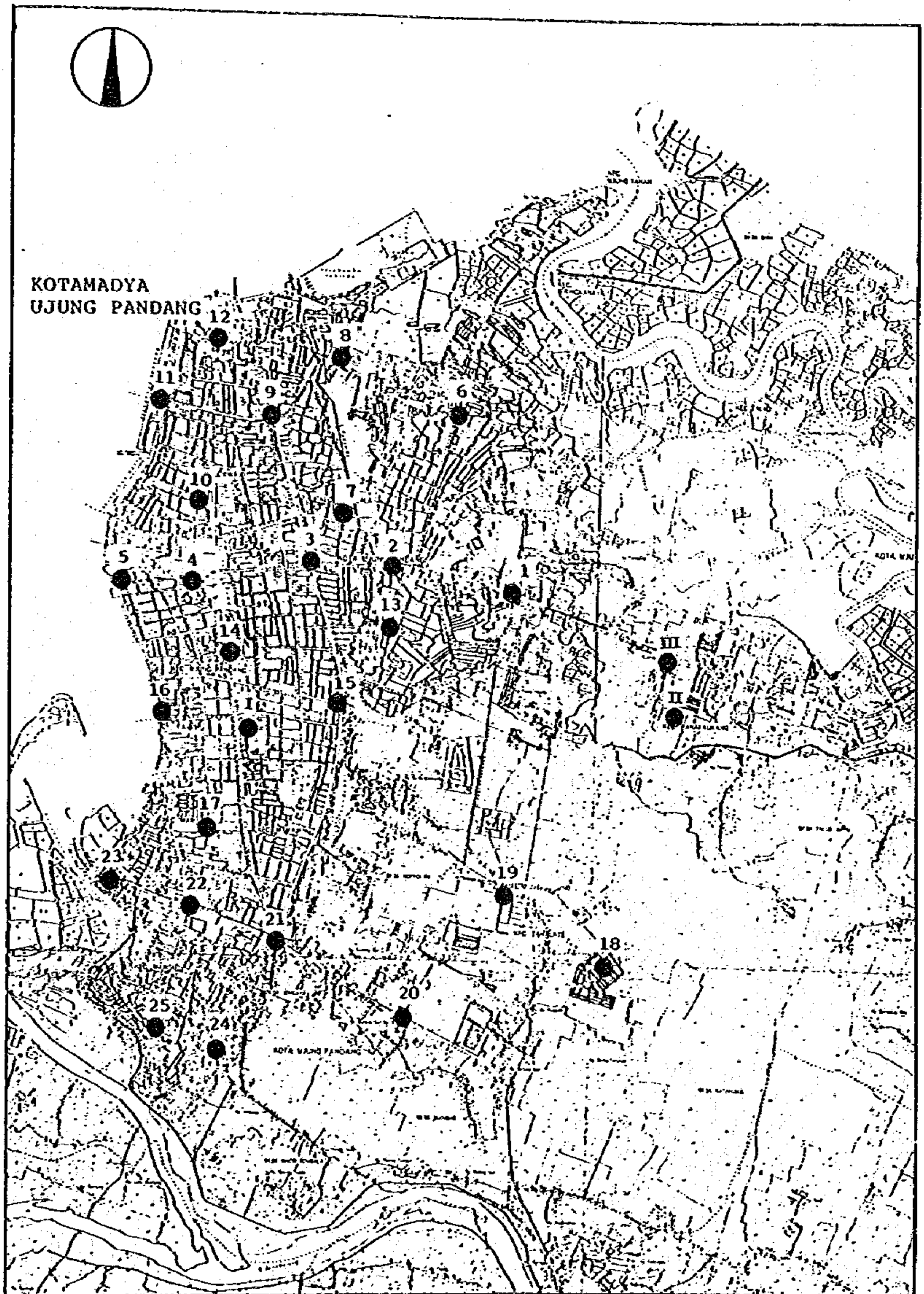


FIGURE	POINTS OF SURVEY
2.1	

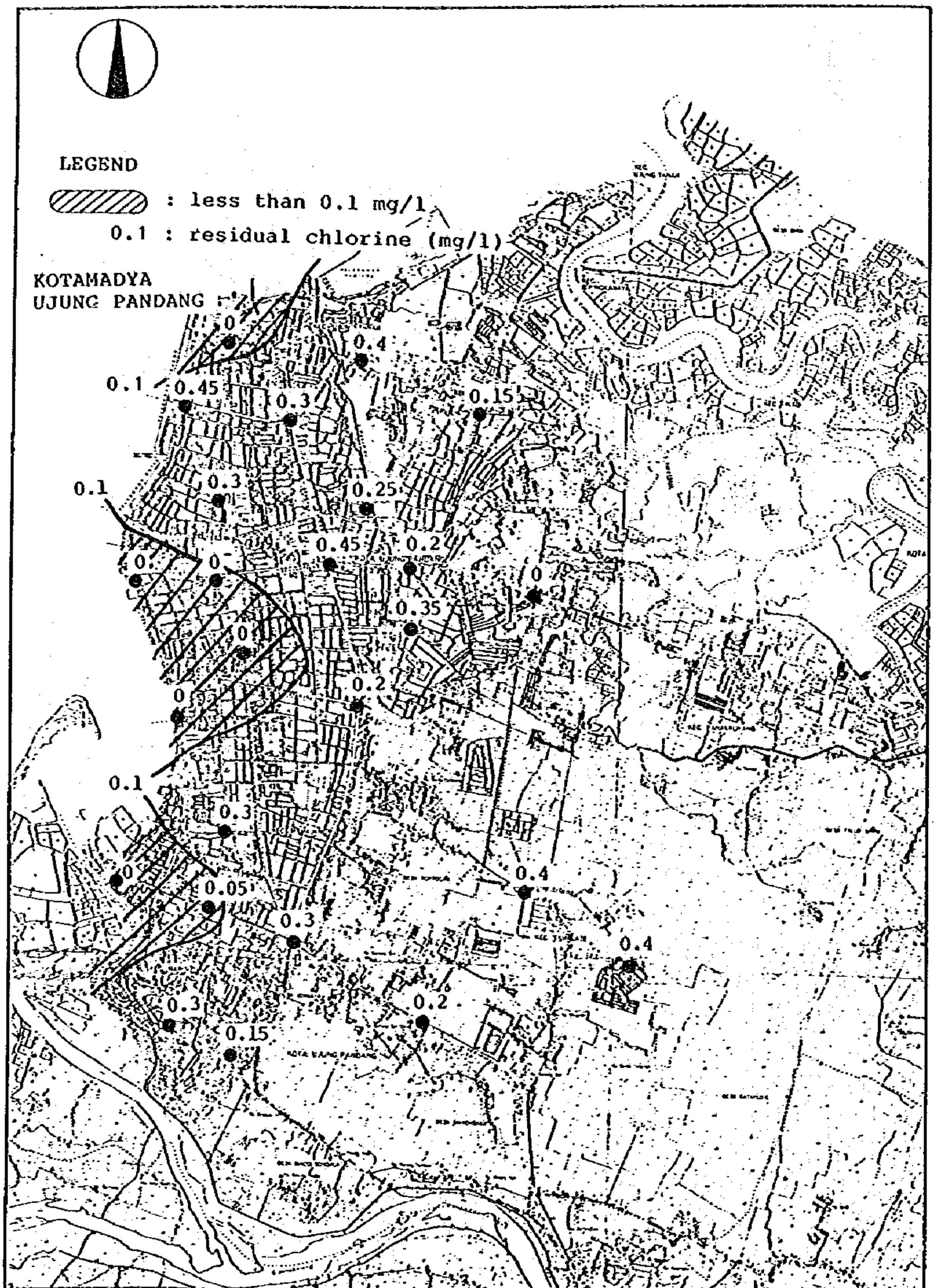


FIGURE	DISTRIBUTION OF RESIDUAL CHLORINE
2.2	

TABLE 2.6 WATER QUALITY AT SERVICE TAP

No.	Old or New System	Total Residual Chlorine	Free Residual Chlorine	Coliform Group	Total Colonies
1	N	<0.05 mg/l	<0.05 mg/l	0 N/100ml	1 N/ml
2	N	0.20	0.15	0	0
3	N	0.45	0.40	0	0
4	O	<0.05	<0.05	0	0
5	O	<0.05	<0.05	0	0
6	N	0.15	<0.05	0	1
7	N	0.25	0.20	0	0
8	N	0.40	0.30	0	0
9	N	0.30	0.20	0	0
10	N and O	0.30	0.15	0	11
11	N	0.45	0.40	0	0
12	O	<0.05	<0.05	0	1
13	N	0.35	0.30	0	0
14	O	<0.05	<0.05	0	0
15	N and O	0.20	0.15	0	0
16	O	<0.05	<0.05	0	2
17	N and O	0.30	0.25	0	2
18	N	0.40	0.40	0	1
19	N	0.40	0.30	0	0
20	N	0.20	0.10	0	0
21	N	0.30	0.20	0	0
22	O	0.05	<0.05	0	17
23	O	<0.05	<0.05	0	8
24	N and O	0.15	0.10	0	0
25	N and O	0.30	0.25	0	0
I	O	0.05	0.10	0	4
II	N	0.50	0.45	0	0
III	N	0.45	0.40	0	0

N : NEW System (mainly from Panaikang)

O : OLD System (mainly from Ratulangi)

3. OPERATION OF THE WATER SUPPLY SYSTEM

The treatment plant at Mangngasa enables to supply safe and sufficient water to the consumers so far as the plant is operated and maintained appropriately. To ensure proper operation of the whole system, the following works are to be carried out at the treatment plants and the central office at Ratulangi.

<u>Treatment Plant</u>

(1) To gauge and control water flow in the system to produce required quantity of water,
(2) To control water quality by conducting water testing,
(3) To determine dosage rate and to control stock of chemicals,
(4) To maintain the treatment facilities in a good working condition.

<u>Central Office</u>

(5) To control and manage the whole systems on the basis of data and information given by the operation center at each treatment plant.

The control and maintenance for operation of the water supply system listed above are to be supported by several works further as tabulated in Table 3.1.

To carry out such works effectively, the preliminary design delineated in the Feasibility Study proposes the construction of laboratory for water testing, installation of valves and gates to control water flow at the several strategic points of the system, purchase of tools and equipment for pipe installation and leakage reduction, construction of work shop and storage building and provision of spare parts for several kinds of pumps and electrical equipment. Project cost includes all the cost described above.

On the other hand, PDAM is recommended to have following vehicles and equipment to conduct repair work, public campaign and patrol as routine work in addition to the current inventory of the PDAM.

1) Motor Vehicles ---- Pick-ups, truck cranes, mini-buses, tank rollies, and motor bikes,
2) Equipment & tools - Road cutters, tampers, hand rollers, pipe threader, torque wrenches, etc.
3) Gauges ------------ Water pressures gauges and flow gauges.

TABLE 3.1 OPERATION OF THE SYSTEM

Facilities	Item No.*	Work Description
1) Intake Facilities	(1)	- Gauging of flow rates at the outlet of grit chamber,
	(1)	- Gauging of water level and discharge at Bili-Bili intake,
	(2)	- Sample testing of the Jeneberang river water and raw water effluent at the grit chamber,
	(4)	- Maintenance of the raw water canal, gabion and the flush gate particularly in rainy season,
	(5)	- Communication with the authorities concerned (Irrigation, industries, etc.)
2) Raw Water Transmission Pipeline	(1)	- Gauging of water pressure and flow rate,
	(2)	- Sample testing of the raw water at the end of pipeline,
	(4) & (5)	- Patrol and maintenance of pipeline, drain pipes and air valves to cope with an unforseen accident,
	(5)	- Collection of information on construction of roads, poles for telecommunication, etc.
3) Mangngsa Treatment Plant	(1)	- Gauging of water level and flow velocity at each facility,
	(1)	- Gauging of water production at the outlet pipes of distribution pumps,
	(2)	- Sample testing of raw water at the mixing well, settled water at sedimentation basin and clear water at the outlet of filters,
	(2)	- Sample testing of drain and waste water,
	(3)	- Complilation of data on the stocks and consumptions of chemicals and their control and handling,
	(4)	- Maintenance of treatment facilities including mechanical and electrical equipment and repair work where required,
	(5)	- Maintenance and control of the stocked materials and survey on availability of materials & equipment,
	(5)	- Communication with electric company regarding schedule of activities, etc.
4) Distribution Facilities	(1)	- Water pressure gauging at the house taps,
	(2)	- Water sampling at the house taps and testing,
	(2)	- Water sampling at the existing wells and testing,
	(4)	- Periodical patrol at the distribution pipelines especially at house connections, public standpipes and pipe bridges,
	(4) & (5)	- Leakage reduction and repair work and data compilation,
	(5)	- Establishment of organization to cope with in case of emergency
	(5)	- Preparation of as-built drawing (service pipes), collection of information on construction of roads, drainage, etc., and communication with authorities concerned.

* --- It shows number of items listed in the context.

ANNEX MEASURES TO BE TAKEN DURING DRY SEASON

1. General

PDAM of Ujung Pandang will have two large water sources, after completion of Stage I Project. They are the Maros River for the existing water system and Jeneberang River for the new water supply system.

As the proposed new intake site named "Bili Bili Intake" is to be located 29 km up-stream from the river mouth, and the most up-stream of other intake sites for other purposes, the intake of PDAM will be secured highly.

On the other hand, the Maros River, the water source for the existing Panaikang treatment plant, will decrease its discharge to a critical level for several weeks in a severe drought which may happen every 5 years or so.

This paper deals with measures to be taken during such dry season periods.

2. River Flow and Water Demand in Severe Dry Seasons

The Panaikang treatment plant will be obliged to reduce to some extent due to shortage of river flow during the severe dry season. It should be noted, however, that such occasion take place once or twice within 10 years.

After the completion of the new treatment plant, daily average demand will be able to be covered every year even if severe drought takes place. This forecast is based on the assumption that the intake quantity of the Panaikang plant is 0.765 m3/sec, after taking into consideration the water loss of Maros raw water canal and possible reuse of waste water in the Plant.

3. Environmental Conditions and Measures to be Taken

There exist the following environmental conditions around the Maros River:

1) The reforestation at the upstream of the Maros River, which has been developed by Authorities concerned in 1969, is contributing to the river water. It shows a trend of increase in the minimum river flow.

2) The Tallo River recorded a considerable amount of flow in the dry season in 1984, and its water quality has no particular problems for treatment, as shown in Table 1.

3) It was observed that water impounded once by the weir at Leko Pancing came out at several hundred meters downstream of the weir, after submerged in the dry season.

4) A water well constructed near Panaikang has a yield of about 10 l/sec.

The above findings should be reconfirmed with further detailed investigation prior to the project implementation in the future.

Measures to be taken are considered as follows:

a) To reuse water discharged from filters and sedimentation basins in Panaikang treatment plant. The waste water will be discharged to the existing intake by gravity and then lifted to the receiving well by the existing intake pumps.

b) In the case after a new plant is constructed, the capacity which exceeds the normal capacity in Panaikang Plant will be covered by overloading of the new plant to possible extent.

The above two plans are considered economical and effective methods. Other measures, which are reliable but need considerable cost, will be the followings:

c) To construct additional treatment facilities in the new plant which shall cover the capacity of Panaikang Plant during dry seasons.

d) To install a bypass raw water transmission pipeline to Panaikang Plant from a new raw water transmission pipeline for the new plant.

However, the above plans of c) and d) are not recommendable, since their costs are ten-times more expensive than those of a) or b), and their operation and maintenance is comparatively complicated.

4. Conclusion and Recommendations

Among above-mentioned measures, plans a) and b) are recommended for implementation. Especially plan a) can be immediately executed with local budget. In this case, water to be recovered by the reuse is about 40 - 50 l/sec. This may be a most effective method to be taken until the new plant is constructed, taking into consideration the conditions of 1), 2), 3) and 4) mentioned in Section 3. In addition, this measure will contribute to the reduction of pollution of the river to which the waste water has been discharged.

Plan b) will be very effective during severe dry seasons after completion of the new treatment plant. This method requires almost no additional works of rehabilitation or replacement because of the following reasons:

- Transmission pipeline, 1,100 mm in diameter has a sufficient capacity to carry an additional volume of raw water.

- Adoption of direct filtration will enable a gravity flow in the treatment process even in case of the overload operation.

- For distribution, standby pumps and generators will be operated.

TABLE 1 WATER QUALITY ANALYSIS OF TALLO RIVER

Items		Near the existing transmission Canal (St 7)		
Sampling Date		19 - 9 - 1984	4 - 10 - 1984	17 -10-1984
Water Temperature	°C	30.5	28.9	29.3
PH		6.8	6.8	7.0
Turbidity	Degree	56	64	40
Electric Conductivity	/cm	177	-	184
Alkalinity	mg/l	42	32	60
Total Hardness	°D	1.8	1.7	2.2
Chloride Ion	mg/l	16.3	15.6	25.6
Ammonia	mg/l	-	-	0.08
Dissolved Iron	mg/l	-	-	-
Manganese	mg/l	-	-	-
Coliform Group	N/100 ml	-	-	800
Total Colonies	N/ml	-	-	2772
Dissolved Oxygen	mg/l	-	-	-
Potassium Permanganate consumed	mg/l	-	-	-
C O D	mg/l	-	-	-

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