7.4.1 Intake Station Site

(1) Criteria for Selection

The intake point must meet the following conditions:

1) to secure the design amount for intake and to remain enough flow in the river for river maintenance even after water intake
2) to have a fast velocity and less sediments of soil due to a close location of a flowing core to the intake side
3) to keep the water depth enough for intake, even though considering some sediments of soil

(2) Selection of Intake Point

Through the reconnaissance survey along the Kalu Ganga, five intake points were selected for a comparative study as follows and as shown in Figure 7.5.

Site I1	1.6 km downstream of the confluence (Hiltotapiya, Udugammana)
Site I2	immediately upstream of the Narthupana Bridge
Site I3	2.8 km upstream of the confluence (Walpita)
Site I4	4.6 km upstream of the confluence
Site I5	7.3 km upstream of the confluence

The intake point was selected with the following two steps:

Step 1 to select each one site from the downstream locations (Sites I1 and I2) and upstream locations (Sites I3, I4 and I4), respectively, taking into account the physical conditions

Step 2 to select one site between the downstream and upstream locations taking into account the quantitative and qualitative problems

Step 1

1) Downstream Locations

Site I1 was selected by the following reasons:

1. At Site I1, due to the rocks projected into the river, the cross sectional area is smaller than up and down stream so that the flow velocity becomes higher. Settling of silt and sand in the river bed is therefore less.
2. Site I2 is located 2 km further from the supposed route of the raw water transmission main in comparison with Site I1. It requires the higher cost both in construction of the raw water transmission main and in operation and maintenance of the pumping facilities.

3. There is a large dye factory immediately upstream of the Narthupana Bridge which has possibility to discharge wastewater into the river and cause pollution.

4. The gap in elevation between the nearby road and water level of the Kalu Ganga is larger at Site I2 than at Site I1. The side intake method in which the raw water taken from the river is conveyed to the grit chamber by gravity, is suitable to Site I1 but not applicable to Site I2 due to its deep excavation works. The intake tower method is instead proposed for Site I2, but it will need a large-scale of temporary works to protect the construction site in the river. Consequently, the construction cost of the intake tower will be higher than that of the side intake (see Figures 7.6 and 7.7).

2) Upstream Location

Site I3 was selected with the following reasons:

1. Site I3 is located immediately upstream of the ferry station at Walpita. The existing river bank is formed with vertical rocks and alignment appears to be suitable for intake.

2. Site I4 is located immediately upstream of the Walpita Bridge where sand bars appear in front of the supposed intake point in the dry season. Sand bars will obstruct water intake in the dry season. If the intake mouth is to be extended beyond the sand bars, it will obstruct the river flow in the flood time.

3. Site I5 is situated at the sharp curve immediately upstream of the ferry station at Nelligashima. As its soil bank is directly hit by the flood flow, there will be a danger of serious erosion in the bank after construction of the intake facilities.

Step 2

The design amount for intake is given as the summation of the design amount for water supply and that to be used for miscellaneous purposes in the water treatment plant. Assuming that the amount of water to be used in the plant is approximately 5 percent of the amount to be produced, the design amounts for the intake are then calculated below for the water demand in the years of 2010 and 2020, respectively.

For 2010 $182,000 \times 1.05 = 191,000$ m^3/d (2.2 m^3/s)

For 2020 $364,000 \times 1.05 = 382,000$ m^3/d (4.4 m^3/s)

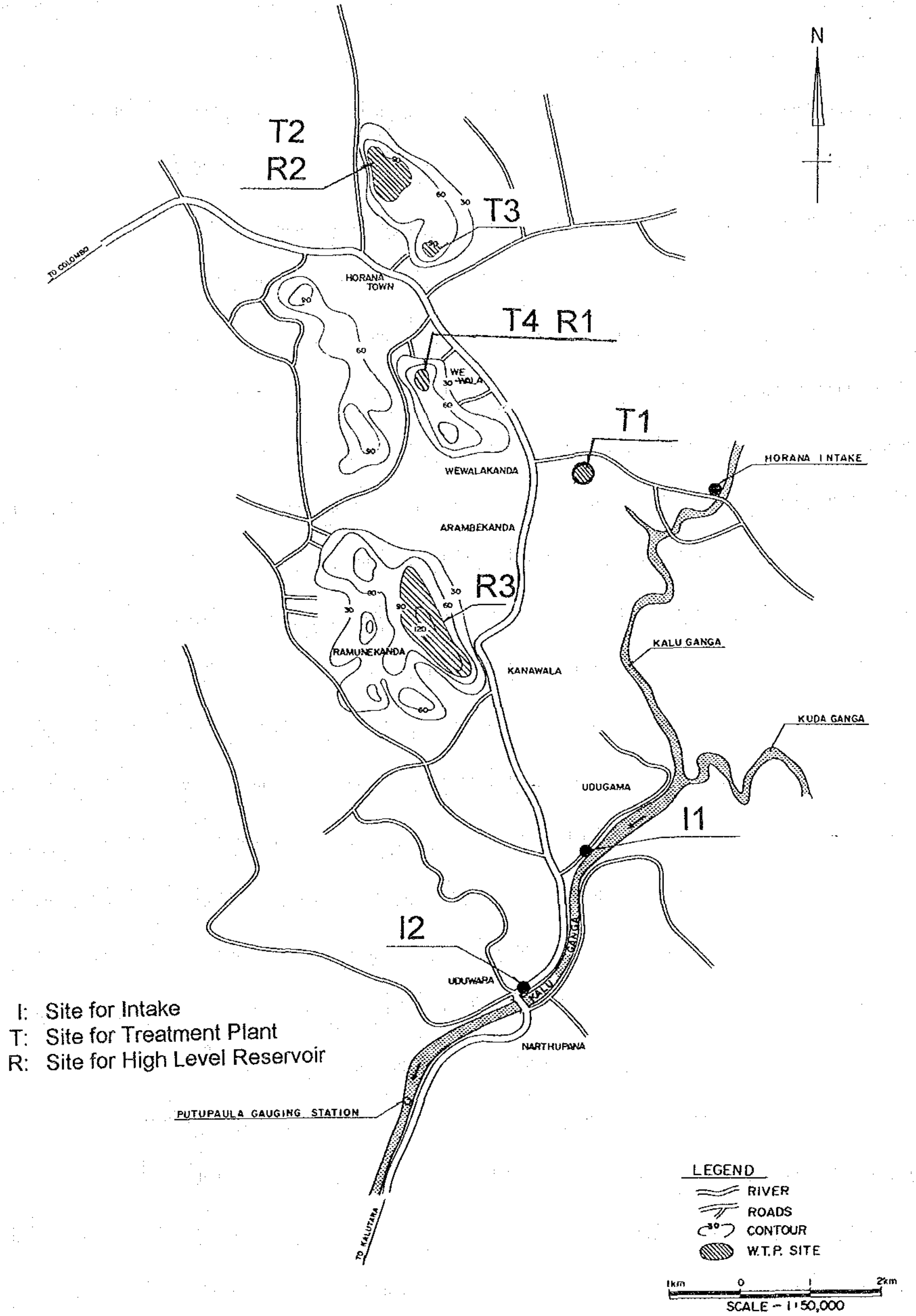

Figure 7.5 Proposed Sites for Intake and Treatment Facilities

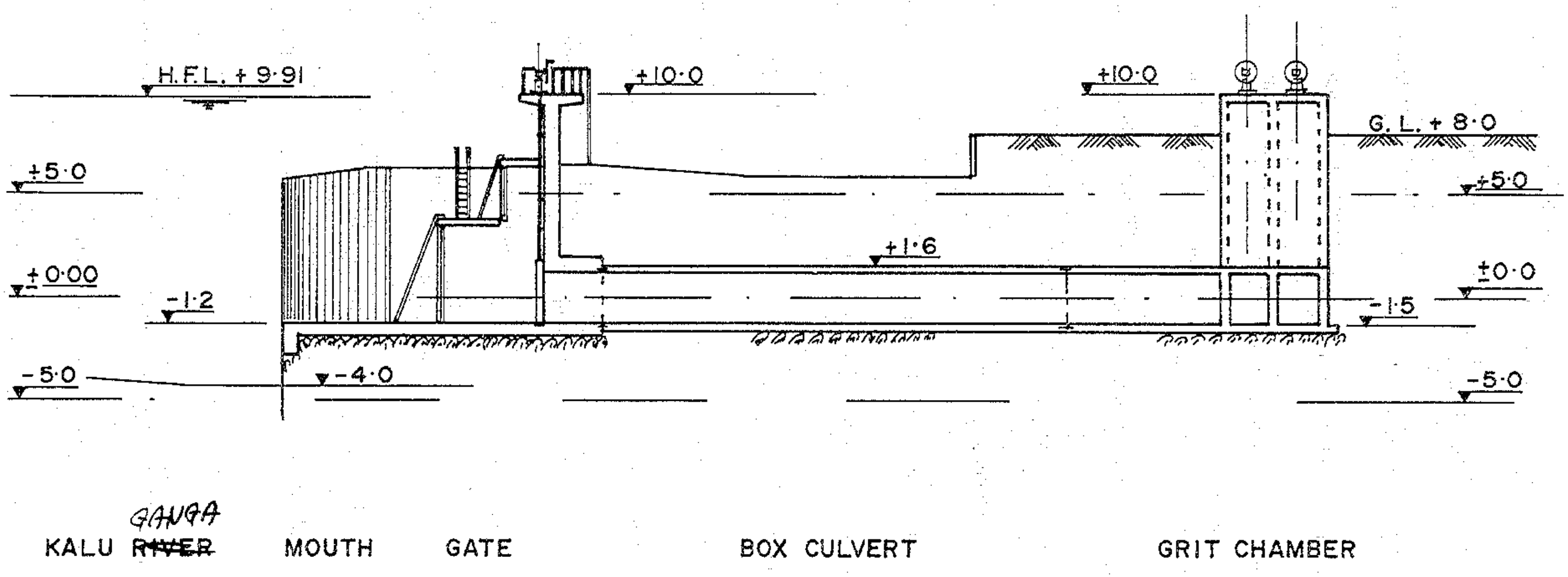

Figure 7.6 Intake Gate to fit for Site 1

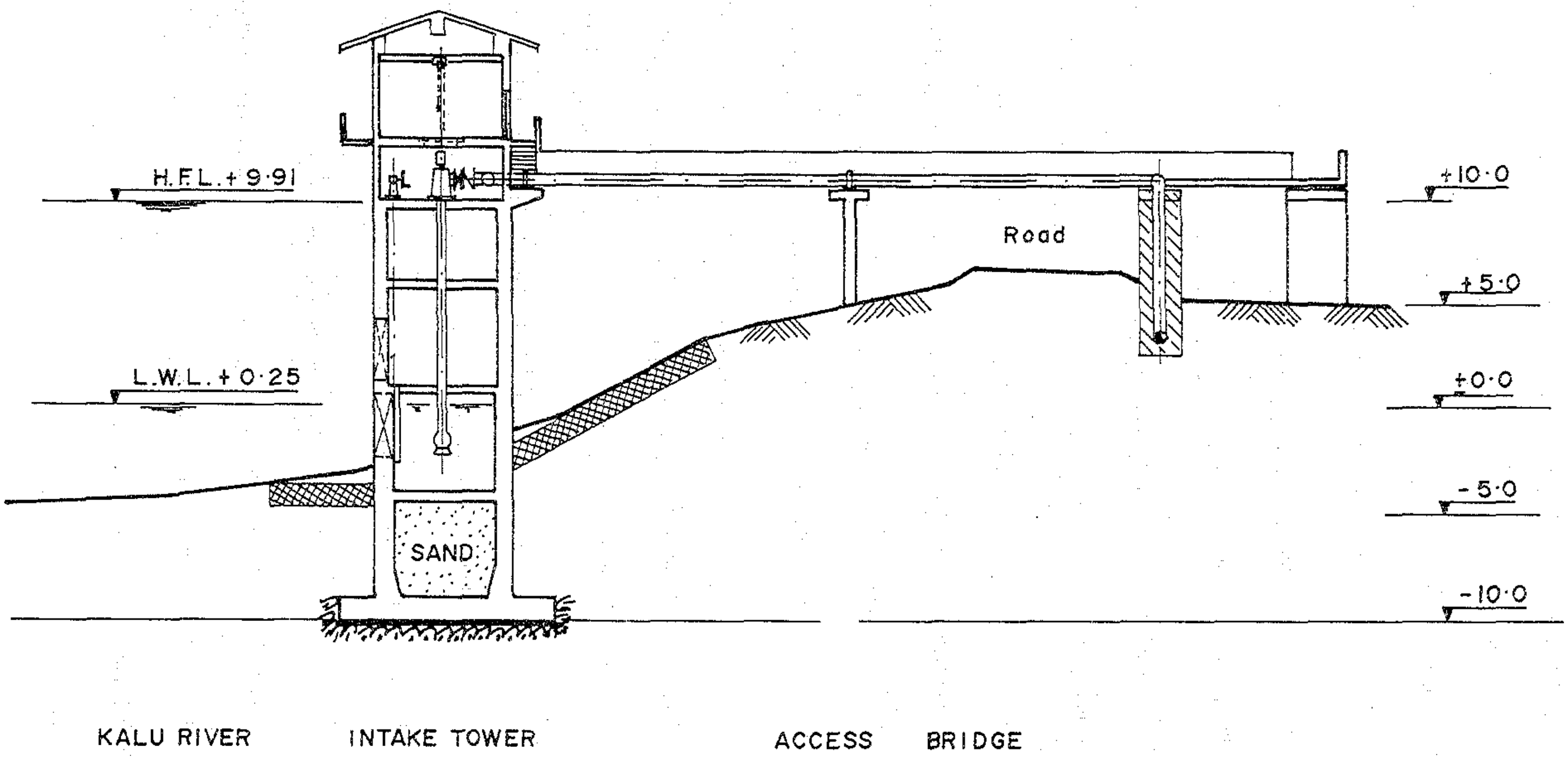

Figure 7.7 Intake Tower to fit for Site 2

1) For Annual Minimum River Flow and River Maintenance Flow

There is no obvious provision on the rive maintenance flow in Sri Lanka, therefore, the extreme annual minimum river flow of 11.0 m^3/d, which shows the severest natural condition in the past, is adopted as the tentative aim of the river maintenance flow in this study.

For the amount of river flow at the confluence of the Kalu Ganga and Kuda Ganga, the ratio of the flow amount coming in from each river is approximately 2:1. In other word, the main stretch of the Kalu Ganga contributes to approximately two third of the total flow after the confluence. As the minimum flow of the Kalu Ganga at Putpaula is estimated at 14.4 m^3/s for a return period of 10 years, that of the Kalu Ganga upstream of the confluence with the Kuda Ganga is calculated as given below.

$$14.4 \text{ m}^3/\text{s} \times 2/3 = 9.6 \text{ m}^3/\text{s}$$

The rates of the intake amount for 2020 demand to the planned minimum river flow are then calculated at 30.6 percent for the downstream location and at 45.8 percent for the upstream location respectively, which suggests that the upstream location has less allowance in the river flow.

For the downstream	4.4 / 14.4 = 30.6%
For the upstream	4.4 / 9.6 = 45.8%

As the tentative aim of the river maintenance flow of the Kalu Ganga is assumed at 11.0 m^3/s, that of the Kalu Ganga upstream of the confluence is likewise calculated below.

$$11.0 \text{ m}^3/\text{s} \times 2/3 = 7.3 \text{ m}^3/\text{s}$$

The river flow and its rate to the tentative aim of the river maintenance flow after the intake are as shown below:

	For the downstream	For the upstream
For 2010	14.4 - 2.2 = 12.2 m^3/s (110% of 11.0 m^3/s)	9.6 - 2.2 = 7.4 m^3/s (101% of 7.3 m^3/s)
For 2020	14.4 - 4.4 = 10.0 m^3/s (91% of 11.0 m^3/s)	9.6 - 4.4 = 5.2 m^3/s (71% of 7.3 m^3/s)

Both locations have no problem for the 2010 design amount for intake but are below the tentative aim of the river maintenance flow for the 2020 design amount for intake with some difference in extent. The river will be placed in a severer condition between the intake point and the confluence in case of the upstream location but in the same condition as the downstream location.

(2) For Salinity Intrusion

When the water contains chloride concentration of more than 250 mg/l, the water likely tastes salty. During the severe drought in 1991, it is said that about 200 mg/l chloride was detected at the Kalutara Intake Station. This was still below the salty level.

The salinity intrusion analysis indicates that, for the downstream location, there will be no effect of salinity intrusion for the 2010 water demand, but there may be an effect for the 2020 water demand. On the other hand, for the upstream location, there is no effect for both target years due to the existence of the drop which can cut the salinity wedge upstream.

3) Construction, Operation and Maintenance Costs

The construction cost is lower in the upstream location due to the shorter length of the raw water transmission main as well as the operation and maintenance cost because of smaller actual head of intake pumps and less friction loss on the raw water transmission main.

4) Operation and Maintenance

The conditions of operation and maintenance for the grit chamber and intake pumping station are identical to both the upstream and downstream locations. In case of the downstream location, the salinity intrusion monitoring and intake level adjustment of intake gates are required for the 2020 water demand.

(3) Conclusion

The downstream location is recommended as the intake point with the following reasons:

There is no problem for both upstream and downstream locations for the 2010 water demand. For the 2020 water demand, the upstream location has a constraints in quantity while the downstream location in quality.

As per the river flow the following are pointed out:

- The minimum river flow of the Kalu Ganga apparently has a declining trend from a long-term view point (see Figure 6.11).
- As the Kalu Ganga is only one water source remaining in the Greater Colombo Area, more water-consuming factories may be shifted to or be located in the service area of the Kalu Ganga Water Supply System than expected which will lead to more increase in water demand as projected.
- At present, there is neither comprehensive basin development program for the Kalu Ganga nor any regulation for water use of the Kalu Ganga. It is therefore in a state of

no control of water intake. Development projects with an intake from the Kalu Ganga might be implemented upstream of the intake point in future.

- There may be a imbalance in rainfall in the Kalu Ganga basin: for example, more rainfall in the Kalu Ganga basin and less rainfall in the Kuda Ganga basin.

In the above situations, the upstream location is disadvantageous as an intake point, since it rely water source only on the Kalu Ganga while water of the Kuda Ganga will not be used. In case there will be a shortage in river flow, there is no way to help.

The occurrence of the qualitative problem at the downstream location due to the salinity intrusion, of which the duration period is expectedly one to two weeks in ten years can be prevented by adjusting the intake level using a stop log or intake gate provided with intake mouth, or constructing a salinity barrier in the Kalu Ganga.

Further, in case that the water source is a river like the Kalu Ganga which has no flow regulation, there is no adjusting function like the intake from the reservoir and the water supply should rely only on the natural river flow. Therefore, in selecting the intake point, the first priority should be given to securing a stable amount of water than the operation and maintenance cost and casiness in operation and maintenance. Thus the downstream location is selected for the intake point.

Although the Kalu Ganga is flooded every year, it has a small river flow and less allowance during a drought. Regardless of the location of the intake point, implementation of any future development requiring water intake from the Kalu Ganga should be restricted upstream of the intake point.

7.4.2 Water Treatment

(1) Consideration in Site Selection

1) Conditions for Site Selection

1. The following three types of water supply systems are considered in selecting the proposed sites for the water treatment plant:

   (a) One-stage pumping system

Intake Station w/ Pumps	⇒	Treatment Plant w/ Reservoir	⇒	Existing Reservoirs

(b) Two-stage pumping system

Intake Station w/ Pumps	⇒	Treatment Plant w/ Pumps	⇒	High-Level Reservoir	⇒	Existing Reservoirs

(c) Direct pumping system

Intake Station w/ Pumps	⇒	Treatment Plant w/ pumps	⇒	Existing Reservoirs

As the treatment facilities perform their functions in the integrated system with other facilities from the intake to distribution, it is important to take into consideration the hydraulic relationship among facilities in addition to the location arrangement.

There are several reservoirs and water towers in the existing service area. If the clear water will be conveyed to those elevated tank by gravity, it will lead to the simplification of operation and maintenance as well as the reduction in the operation and maintenance costs. The existing service reservoirs have water levels of 15 m to 54 m. Considering a friction loss on the transmission main from the new reservoir to the existing reservoirs, it is necessary that the high level reservoir in the one- or two-stage pumping system has a water level of approximately 90 m to transmit the clear water by gravity to the existing service reservoirs.

2. The site must have an elevation of more than +11 m MSL.

There are some possible sites near the right bank of the Kalu Ganga with elevations of 5.5 to 6.0 m. The hydrological analysis of the Kalu Ganga however reveals that the flood levels are +10.18 m and +10.58 m for return periods of 50 and 100 years, respectively, which require the embankment with a height of approximately 6.0 m prior to the construction of the treatment facilities at these sites and their inevitably large-scale foundation works resulting in the high construction cost. Hence, such sites are eliminated from the Study.

3. The site must be located along or near the supposed route of a transmission main to the service area.

Taking into account the proposed intake sites and the existing and new service areas to be covered by the Kalu Ganga Water Supply System, it is economically and technically reasonable that the transmission main will be laid in Roads B157 and A8 due to the shortest distance.

Therefore, the treatment facilities are desired to be located at such a place with an elevation of more than 11 m in the two stage pumping or direct pumping system, and of more than 80 m in the one stage pumping system along or near Roads B157 and A8.

Taking into consideration the above, four sites were selected for the treatment plant and three sites for the high level reservoir in a two-stage pumping system on the basis of the identification on the topographical map with a scale of 1:50,000 and the subsequent reconnaissance as shown in Figure 7.5.

Site T1 on a mount at south of the road branched from Road B157 and going to the Horana Intake Station is owned by the governmental corporation with an area of 14.6 ha. This area is enough for the proposed water treatment facilities. The land with some undulations is mostly covered with shrubs and is sunny. Some housing units are currently located in the site.

Site T2 is located on a small hill north of Horana Town and has an existing road with a gentle slope which makes the construction of the treatment facilities easy with less amounts of soil to be excavated and disposed. However, this site does not have an enough area to accommodate the full treatment facilities for the design year of 2020.

Likewise, Site T3 on a small hill east of Road B157 has an enough elevation to transmit the clear water to the existing service reservoirs in the service area by gravity, but seems difficult to construct the access road and treatment facilities due to its steep slope. In addition, a large amount of soil to be excavated and disposed is expected in the course of site grading.

Site T4 is situated in the rubber plantation on the west of Road B157 and is owned by privates. The configuration of the land is narrow from the north to the south and has a gentle descent southward. Although in the course of site grading, a number of rubber trees will have to be felled, the site will not be sunny due to surrounding trees and hills.

As a conclusion, possible selections are Site T1 for the full treatment facilities with a design year of 2020 and Site T2 only for the treatment facilities with a design year of 2010. System alternatives in relation to Sites T1 and T2 are discussed in Subsection 7.5.2.

As for the site for the high-level reservoir to be located on a hill, Site R1 is suitable because both Site R2 north of Horana Town and Site R3 along Road B157 have steep slopes and no direct access road.

7.5 Proposed Alternative Plan

7.5.1 General Approach to Alternative Selection

To identify the appropriate water supply system, possible alternatives were developed in relation to the location, pipeline route and the operation system, although the alternatives developed are a few due to the limited sites and access for the facilities. All alternatives have been evaluated through a comparative study from the technical, economical and institutional viewpoint.

It should be noted that the costs presented below are used only for a comparative study to select the appropriate alternative and does not show the final values.

7.5.2 Water Intake and Raw Water Transmission

From the results of the site selection stated in Section 7.3, the following three alternatives are developed as to the combination of the intake, transmission and treatment facilities as shown in Figure 7.8.

Alternative A Combination system of one- and two-stage pumping (Figure 7.9)

> The first phase treatment plant with a design capacity for the 2010 demand will be constructed at Site T2 on a mountain together with a distribution reservoir to be used for the one-stage pumping system. The second phase treatment plant to cope with the balance demand for 2020 will be constructed on Site T1 to be used for the two-stage pumping system for which an additional distribution reservoir will be located in the first phase treatment plant.

Alternative B Two-stage pumping system (Figure 7.10)

> The water treatment plant with a design capacity for the 2020 demand will be constructed in two phases at Site T1 from which the clear water will be pumped up to a reservoir to be constructed on Site R1 and then will be gravitated to the existing reservoirs.

Alternative C Direct pumping system (Figure 7.11)

> Alternative C is same as Alternative B other than that the clear water will be directly pumped to the reservoirs or towers in the service areas so that the high level reservoir will not be required.

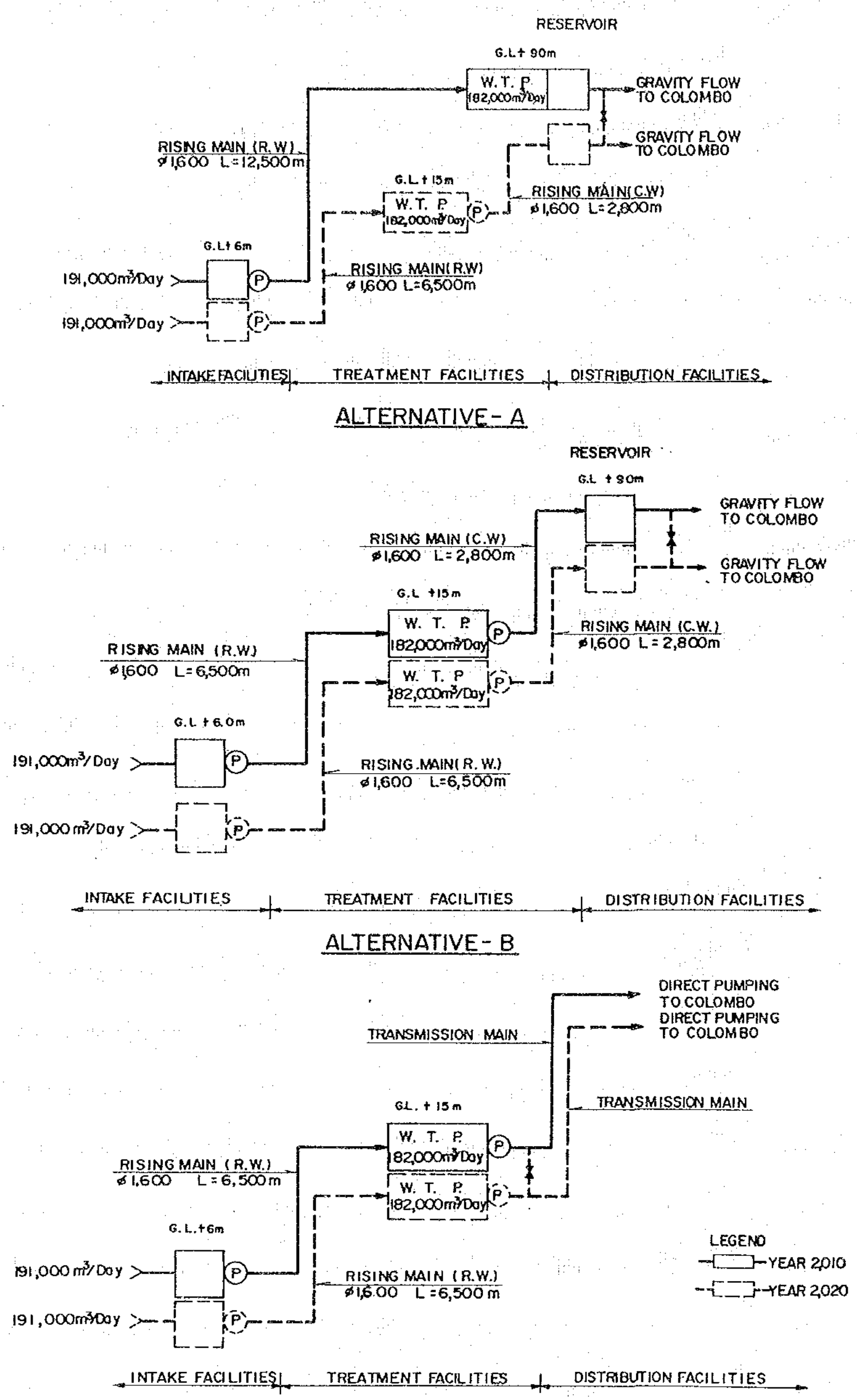

Figure 7.8 System Diagram of Intake-Treatment Alternatives

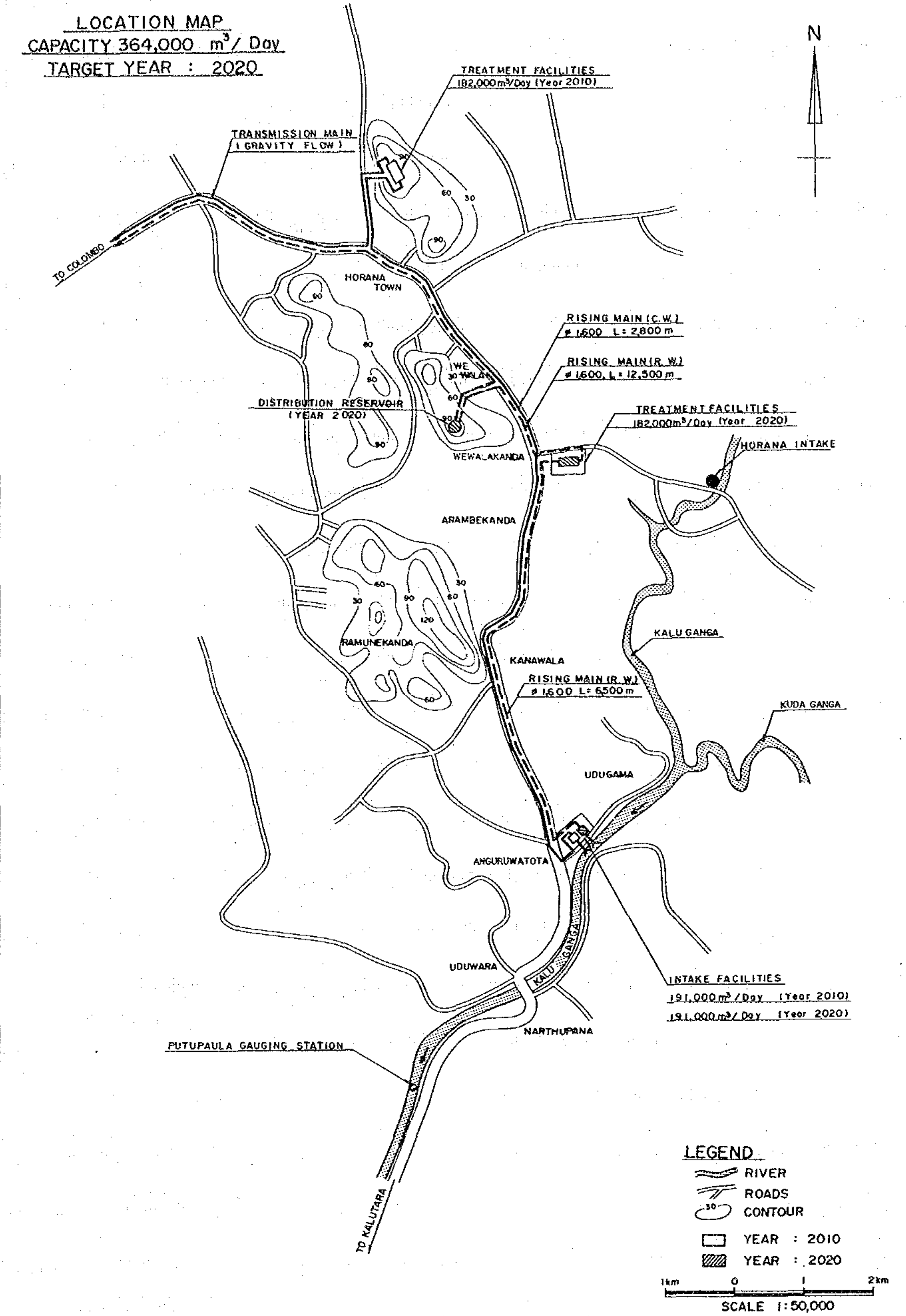

Figure 7.9 Intake-Treatment Alternative A

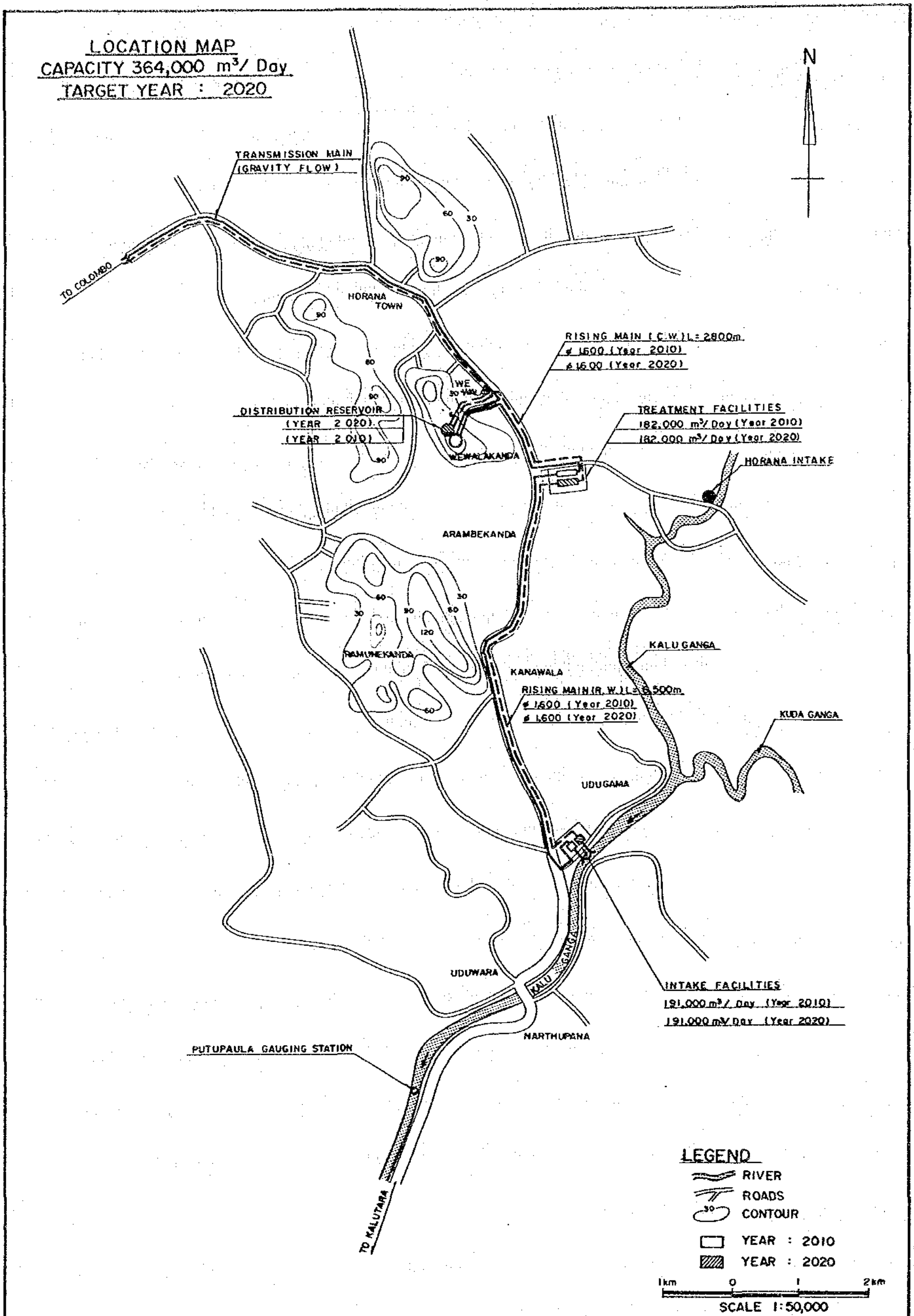

Figure 7.10 Intake-Treatment Alternative B

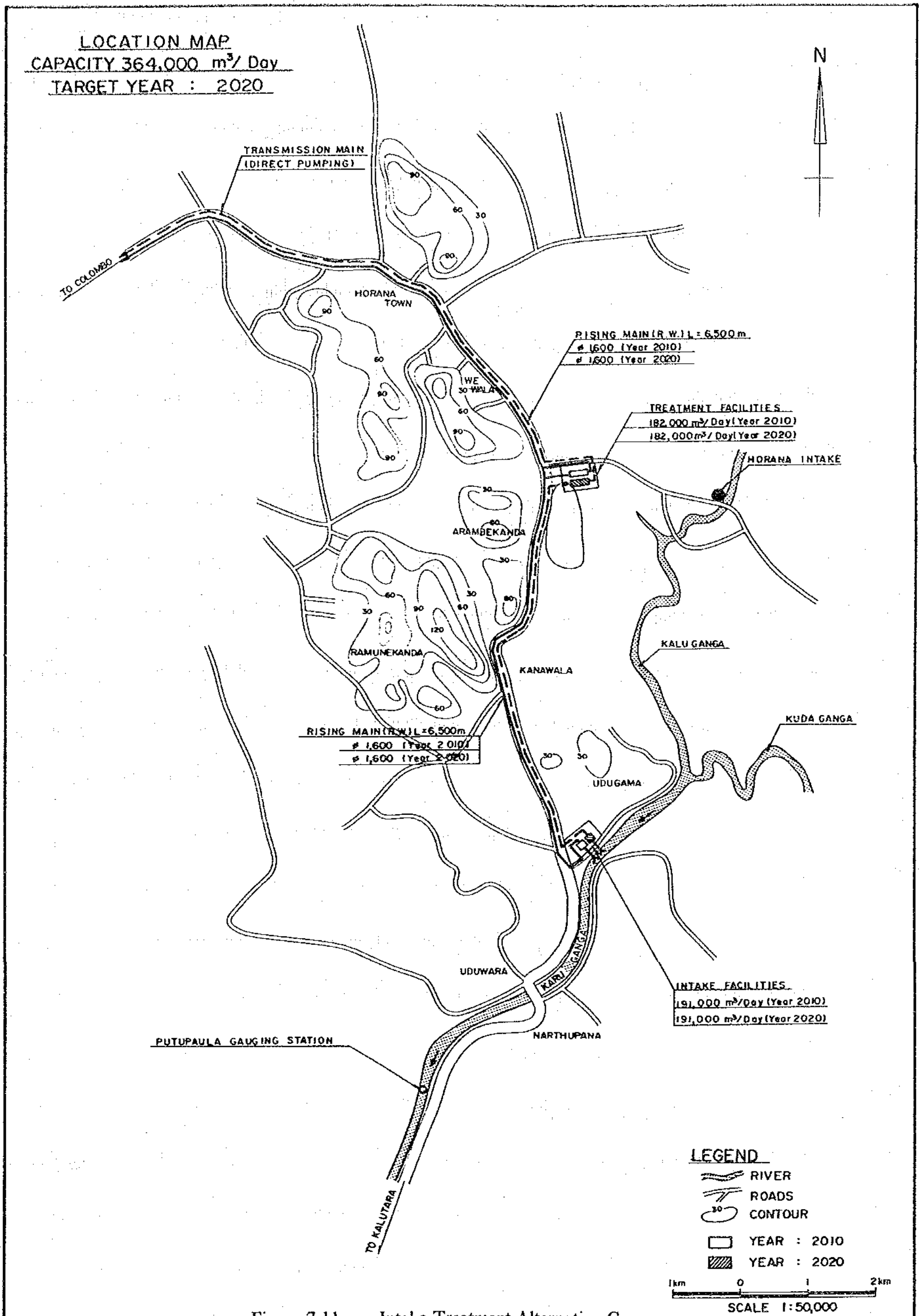

Figure 7.11 Intake-Treatment Alternative C

The features of each alternative are summarized in Table 7.9.

Table 7.9 Features of Proposed Alternatives

Alternative A	Alternative B	Alternative C
System Configuration	System Configuration	System Configuration
Combination of one and two stage pumping system By 2010 One-stage pumping After 2010 Addition of two stage pumping at a separate treatment plant.	Two-stage pumping system	Two-stage Pumping (Direct pumping system)
Advantage	Advantage	Advantage
1. Easy operation of gravity transmission from the first phase plant. 2. More tolerable for power failure due to a storage at the distribution reservoir. Operation of stand-by generators will be less sensitive.	1. Easy operation of gravity transmission system from the high distribution reservoir. 2. Easy operation and long life of clear water high lift pumps due to the stable pumping head. Pumps will always be operated at the best point on H-Q curve. They will be free from cavitation or overload problem. 3. More tolerable for power failure due to a storage at the distribution reservoir. Operation of stand-by generators will be less sensitive.	1. Slightly low construction cost. 2. With speed control system, pumps will be operated safely in lower head and discharge when the demand is low. It will help reduce energy consumption as long as the speed control system will be properly maintained.

(continue to next page)

Table 7.9 Features of Proposed Alternatives (cont'd)

Disadvantage	Disadvantage	Disadvantage
1. High land cost. 2. High manning cost in O&M. 3. More troublesome in operation of two treatment plants at separate sites. 4. Need more maintenance for three separate pump systems. 5. Higher manpower requirement due to two separate treatment plants. 6. In the earlier stage when the water demand is less, pump head will be more than required. Energy consumption will be higher than the direct pumping equipped with speed control.	1. In the earlier stage when the water demand is less, pump head will be more than required. Energy consumption will be higher than the direct pumping equipped with speed control. 2. High equipment cost due to two-stage pumping.	1. Need careful control in discharge and pressure due to direct pumping to reservoirs with different elevations and capacities. 2. Need special protection measures for the pumps against cavitation and overload. Speed control or automatic delivery valve control will have to be installed. 3. Need very careful and costly maintenance if speed control is adopted. If a proper maintenance service will not be available, advantage of speed control will be nil. 4. Water hammer will occur every time the pumps stop and start. For prevention of negative pressure, air chamber will not likely be sufficient for this long transmission. Two or more one-way surge tanks will have to be provided on the transmission pipeline. Maintenance thereof will require an additional care by the operators. 5. Need immediate operation of stand-by power generators in case of power interruption. With power failure, clear water transmission will immediately cease in direct pumping. 6. High equipment cost due to two-stage pumping.

Cost comparison based on the tentative estimates for intake, treatment plant, clear water and distribution reservoirs, and transmission pipelines is presented in Table 7.10.

Table 7.10 Construction and Operation and Maintenance Cost (Tentative Estimates)

Facility	Alternative A		Alternative B		Alternative C	
	2010 Phase 1	2020 Phase 2	2010 Phase 1	2020 Phase 2	2010 Phase 1	2020 Phase 2
Construction Cost (Rs. million)						
Intake	709	498	545	527	545	527
Raw Water Transmission	1,625	845	845	845	845	845
Treatment	1,176	1,511	1,591	1,508	1,891	1,808
Clear Water Reservoir	-	211	119	119	-	-
Distribution Reservoir	432	432	400	401	432	432
Clear Water Transmission	1,575	1852	1,853	1,853	1,703	1,703
Total	5,516	5,350	5,352	5,253	5,416	5,315
Index	(103)	(102)	(100)	(100)	(101)	(99)
Operation Cost (Rs. Million/year)						
Power	100	91	91	91	91	91
Total	100	91	91	91	91	91

Note
1. The construction costs show direct costs only.
2. Excluding the construction cost for distribution facilities which are common to all alternatives.
3. Excluding the chemical costs which are common to all alternatives.
4. Clear water transmission is for the part from the treatment plant to Piliyandala.
5. Figures in parentheses show the rate when the construction cost of Alternative B is set at 100 for each phase.

Alternative A has a large advantage of easy operation and maintenance of a system due to a one-stage pumping for the 2010 demand. After that, the second phase treatment plant will however be constructed at far place due to a lack of space at the first phase plant. The second phase plant will require an additional pumping system to pump up the clear water for gravity transmission to the distribution reservoir on the mountain located far from the second treatment plant. This system will be resultantly composed of the two different systems, namely the one-stage and two-stage pumping systems. The separation of treatment facilities at two different places will lead to double investment for common facilities in the treatment plant such as administrative building, warehouses, workshops, etc., and will therefore require more land area in total and more manpower for operation. The number of operators will have to be double. In addition, the combination of different pumping systems will make the operation and maintenance of a system more complicated. The cost will be higher than those of other alternatives with regard to both construction, and operation and maintenance as shown in Table 7.8 although such differences in cost are marginal.

Alternative B adopts a single operation system, namely, two-stage pumping system and a single treatment plant. This system has largest advantages as shown in Table 7.9 and will make ease of operation and maintenance due to its simplicity in system components. Although the equipment cost will be higher than Alternative A, the total construction cost will be lower due to the lower pipeline costs as shown in Table 7.10.

In Alternative B, pump will be operated only by monitoring the water level in the high level reservoir either by manual or automatic on-off settings. Compared with Alternative C, pump operation with a stable head will lead to more efficient and safe operation. Life of the pumps will therefore be longer. This is one of the most important advantages of Alternative B.

The direct pumping system in Alternative C is essentially a two-stage pumping system. The advantages and disadvantages involved in a two-stage pumping system are accordingly applied to Alternative C.

In Alternative C, operation and control of direct pumping will be extremely difficult for this long distance transmission with the service reservoirs and water towers in different water levels and in the amount of water received. Three important factors in direct pumping are: 1) control of the required discharge amount, 2) protection of pumps from cavitation and overloading which will be commonly resulted from lower head and higher discharge when the water demand is less than the design value, and 3) protection of the transmission pipeline against the water hammer which will occur every time the pumps start and stop. These issues are explained in details as follows:

1) Control of Discharge Amount

When the pumps will be used for clear water transmission, amount of discharge will have to meet the required demand at the service area. As the transmitted water will be received at the service reservoirs and water towers, water level in each reservoir and tower will need to be monitored at the pump station for timely on-off of the pumps. When the reservoir is full and inlet valve is closed, the pump head will sharply go up. When the reservoir inlet is opened, it will go down; or no water will flow if the pumps are off. Close and stable communication in long distance will be inevitable for stable water transmission but will not be easily available.

2) Protection of Pumps from Operation Problems

In the hydraulics of the clear water transmission system for the Kalu Ganga Project, a friction loss (approximately 50 m) will be a dominant factor affecting the pump head compared to the actual head (only from the +15m MSL treatment plant to the +24 m MSL Dehiwala Reservoir). This is typical for a long transmission pipeline. It will mean that the pump head will be much affected by the friction loss which will be affected by the amount of water to be transmitted. In the period when the water demand is less, the transmission amount is less. The required pump head will be less as well.

Under such circumstances for direct pumping and without no proper control, the pumps will be operated out of suitable range on H-Q curve and will consequently face the trouble after a quite short period. As seen at the Ambatale plant, typical troubles of pumps are cavitation, overloading, and vibration. To prevent these problems, pump speed control or automatic control of delivery valve opening are normally adopted to make pumps operate on the best point on the H-Q curve for any discharge amount. Speed control will help energy consumption while the delivery valve control will contribute nothing to energy saving.

3) Protection of the Transmission Pipeline against the Water Hammer

Water hammer will not be avoided to occur in any pumping system. It will be serious in particular at the stoppage of pumps rather than at the start. The longer the pipeline length will be, the higher impact of the water hammer will occur. This is because the intensity of the water hammer will be related to the change in the pressure and flow in minute time after pumps stop which will further be related linearly to the distance of the pipeline.

In the phenomena occurred with the water hammer, pressure surge will be able to tolerated by the strength of pipe material itself although the shock wave will cause vibration in the pipeline. More serious matter will however be an occurrence of negative pressure which can easily crush the pipe as it is pressed by external force. Provision of the air chambers is commonly seen in the existing pumping stations in Sri Lanka for prevention of this negative pressure. One-way surge tank will likely have to be provided for the long transmission line. The number of the tanks will depend on the topography. For the safety of the system, counter measure of the water hammer will be inevitably important. Careful maintenance of such provision will also be a must.

Comparison of the characteristics of pump operation between pumping to the high level reservoir and the direct pumping is shown in Figure 7.12. In this figure, it is clear that the direct pumping will have out of range operation for lower flow.

In spite of its advantage in energy saving, provision of the speed control is not recommended due to the extreme difficulty in maintenance, repair, availability of service and spare parts etc. as local conditions in Sri Lanka.

In addition, Alternative C will require a careful maintenance and operation of the stand-by generator system against the power interruption to avoid the sudden stoppage of water supply. Such problem is not so serious in Alternatives A and B where the storage at the distribution reservoir will continue water transmission in some longer time (2 to 4 hours depending on storage) even after the power interruption. Although the stand-by generator for transmission pump operation will definitely be required for Alternatives A (second phase) and B, timing of operation of the generator will be less sensitive in Alternatives A and B compared to that in Alternative C.

Those additional requirements will bring the difficulty in maintenance of the Alternative C system. Further, as there is no significant difference in cost between Alternatives B and C, it is suggested that Alternative C has no overall advantage over Alternative B.

Using a high level distribution reservoir for gravity transmission in Alternatives A and B will be a great advantage suitable for the condition in Sri Lanka because of its ease of operation. Even with the high lift pumping as in Alternative B, the pump operation will be much stable and safe for pump itself due to the stable pump head. The pumps will likely have much longer life than the existing pumps seen at the Ambatale plant.

As a conclusion, Alternative B is recommended for a combination of intake, raw water transmission, treatment, and clear water transmission facilities.

7.5.3 Water Treatment

(1) Design Principle for Water Treatment

It is envisaged that conventional technologies such as found in the industrialized countries may some times be inappropriate, since the ability of the consumers to pay for water is small. So that plants constructed with expensive and imported technologies are not economically feasible. Even where capital costs are subsidized, operation and maintenance costs, which are born by the country concerned, increase proportionately with the complexity and sophistication of the treatment plant, resulting higher water charges for the consumer.

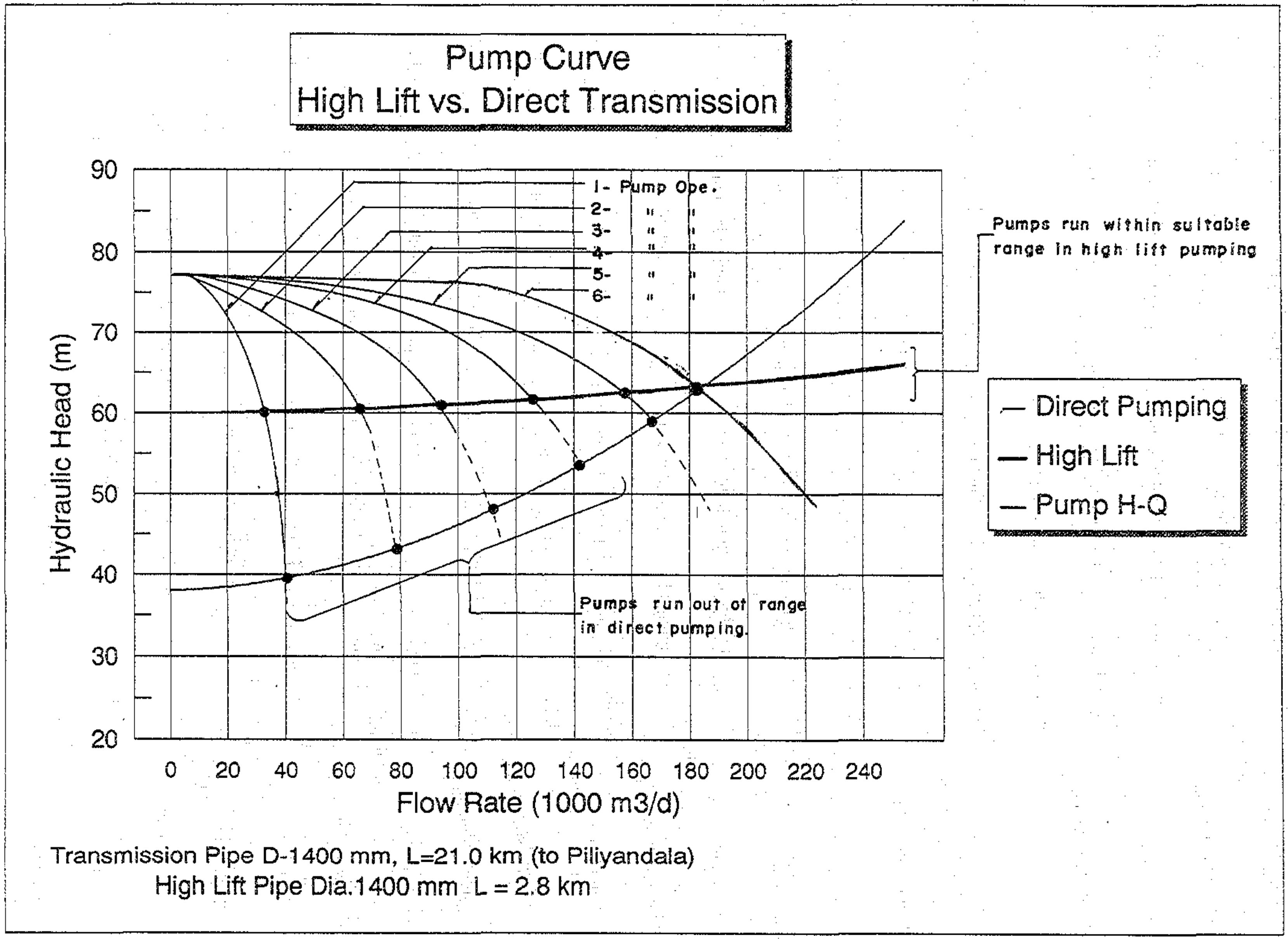

Figure 7. 12 Characteristics of Pump Operation

In general, there is a shortage of skilled personnel to operate and maintain treatment plants in communities in small countries. On the other hand, there is an abundance of unskilled labor, which makes labor-intensive technologies more attractive.

Accordingly, the following technical principles are recommended:

1) To the extent of possible, the utilization of mechanical equipment should be limited to that produced locally.

2) Hydraulically based devices that use gravity for such work as mixing, flocculation, and filter rate control are preferred over mechanized equipment.

3) Mechanization and automation are appropriate only where operations are not readily done manually, or where they greatly improve reliability.

4) Indigenous materials and manufactures should be used to reduce costs and to bolster the local economy and expand industrial development.

(2) Conditions for Treatment Process

The broad choices available in water treatment make it possible to produce virtually any desired quality of finished water from any sources. Therefore, economic and operational considerations become the limiting constraints in selection of treatment units.

Water quality varies from place to place, and in any one place, from season to season, and the resources for construction and operation vary from place to place, so that treatment plant selected must be based on the particular situation.

The primary factors influencing the selection of treatment process are:

1) Standard for potable water.
2) Raw water quality and its variations.
3) Local constraints.
4) Relative costs of different treatment process.

The treated water quality shall conform to the Standard of Potable Water as given in Table 7.7 in the previous section.

Raw water analysis are helpful but unless taken at all seasons, may be misleading because seasonal variations in raw water quality are often extreme in countries with well defined rainy seasons. A limited amount of data regarding raw water quality on Kalu Ganga measured since 1967 to 1994 were obtained during the fist stage Study period. Parameters of pH and turbidity only were measured constantly through the years.

Variation of turbidity in the month through 1986 to 1993 is shown in Data Report (Volume IV) including the highest turbidity unit which rose up to 170 NTU and the second, 120 NTU. However, this level of significantly high turbidity so far as the Kalu Ganga water is concerned rarely occurs and the period is only a single or a couple of days. Turbidity distribution as given in Figure 7.13 measured during 1992 to 1993 reveals that approximately 53 percent of samples is less than 10 NTU and 86 percent of them is less than 20 NTU in 1992, and 40 percent; less than 10 NTU and 65 percent; less than 20 NTU in 1993, in like manner. The water quality in turbidity can, therefore, be categorized as comparatively good for source water. While, daily fluctuation of turbidity as given in Figures 7.14 and 7.15 is considerably intensive. For designing purpose, 75 percent volume with the turbidity between 15 and 25 NTU may be applicable for this Feasibility Study as above indicated in Figure 7.13.

Accordingly, the characteristics of turbidity implies that the rapid filtration be preferentially chosen to the slow filtration as the treatment process, however during the successively long periods of low turbidity, the raw water may be directly forwarded to the filter.

The pH values indicate slightly acidic or neutral as stated in the different subsection in this report. The detail records with respect to pH are summarized in Data Report (Volume IV) measured through 1967 to 1968 and 1992 to 1993.

Other water quality parameters are such that iron values can be in excess of the drinking standard; phosphate and nitrate levels are sufficient to support algae growth; heavy metals are absent; and oxygen demand is low.

Local constraints are quite different from those in the industrialized countries. Considerations should be given to the followings:

1) Limitations of capital
2) Availability of skilled and unskilled labor
3) Availability of major equipment items, construction materials, and water treatment chemicals
4) Availability of local codes and specifications for materials
5) Influence of local traditions, customs, and cultural standards.

(3) Proposed Alternatives for Treatment Process

1) Selection of Principal Treatment Process

The selection of treatment process for the source water from rivers, in general, conforms to either of 1) chlorination only, 2) slow sand filtration, or 3) rapid sand filtration based on the raw water quality and/or characteristics. However, clear categorization in selection amongst these processes by numerical criteria is difficult, since quality of the raw water largely varies in seasons. The practical design guidelines for the selection of treatment process are recommended in many water works organizations in the world. The followings are the excerpt from the Japanese governmental guidelines.

1. In case the water quality in question conforms with the standard like E. coli. not more than 50 (in 100 ml, MPN), total colonies not more than 500 (in 1 ml) and other items of quality tests conform to the standards, chlorination only may be applied.

2. In case the annual turbidity indicates below 10 degree (kaolin 1 mg/ l), below 2 mg/l BOD, below 1,000 E. coli. (100 ml, MPN) for raw water quality in average, slow sand filtration can be employed. In general, when the average annual turbidity of raw water is below 10 degree, chemical coagulation is not necessarily effective. Further, the purifying force is considerably good due to screening by slow filtering, absorption and biochemical action, and a certain amount of ammonia, manganese and odor-emitting matter can be removed, resulting in superior water quality to that of rapid sand filtration.

3. In case the quality of raw water does not apply to 1) or 2), rapid filtration will be better adopted, in which necessary equipment for pre-treatment must be provided for such as chemical coagulation.

The design manual prepared for the NWSDB recommends appropriate treatment processes as given in Table 7.11 for the relevant water quality in Sri Lanka.

In conjunction with the analysis of raw water quality made in this subsection together with comprehensive analysis stated in the previous subsection, the principal treatment process shall be a rapid sand filter system.

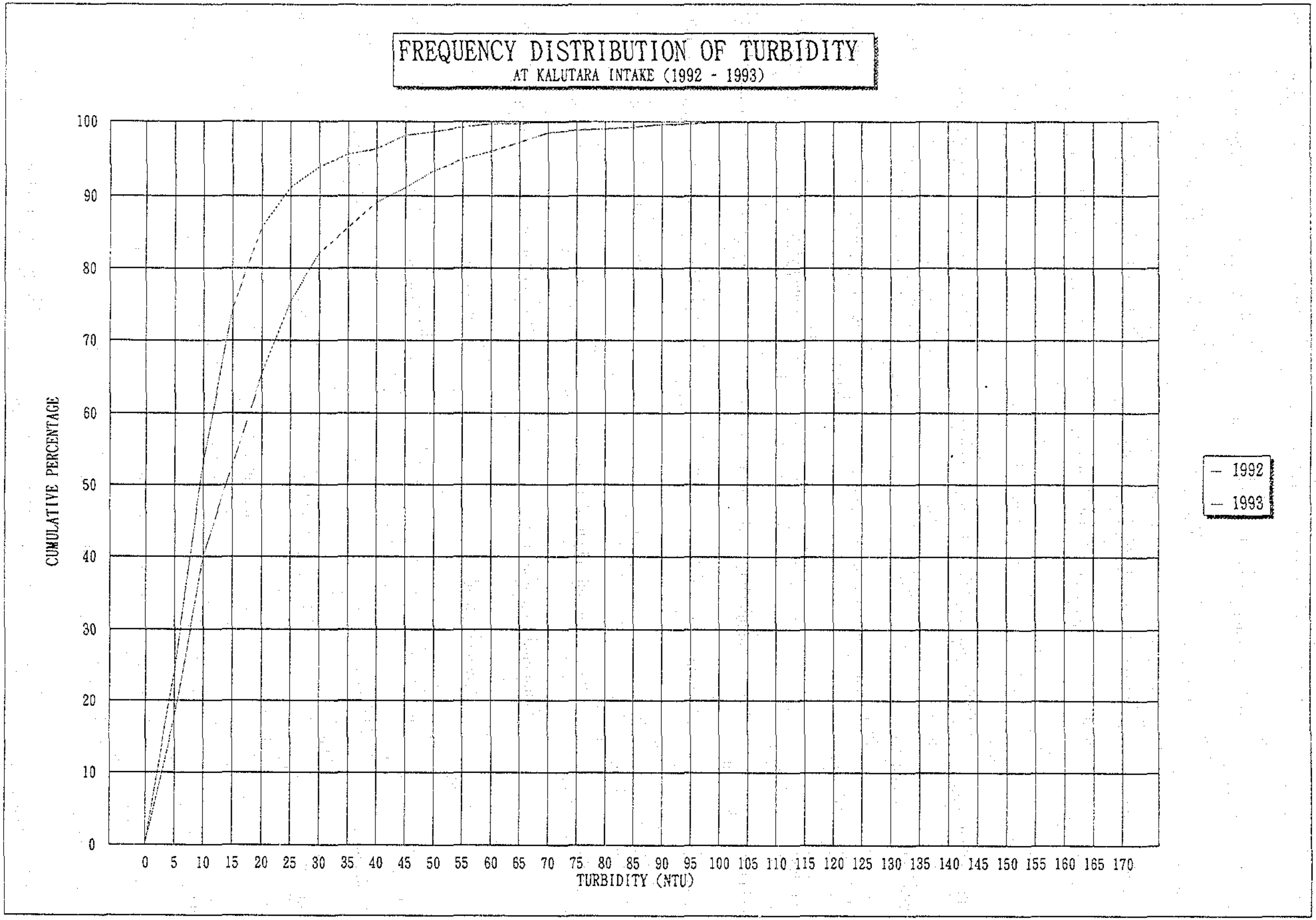

Figure 7.13 Turbidity Distribution

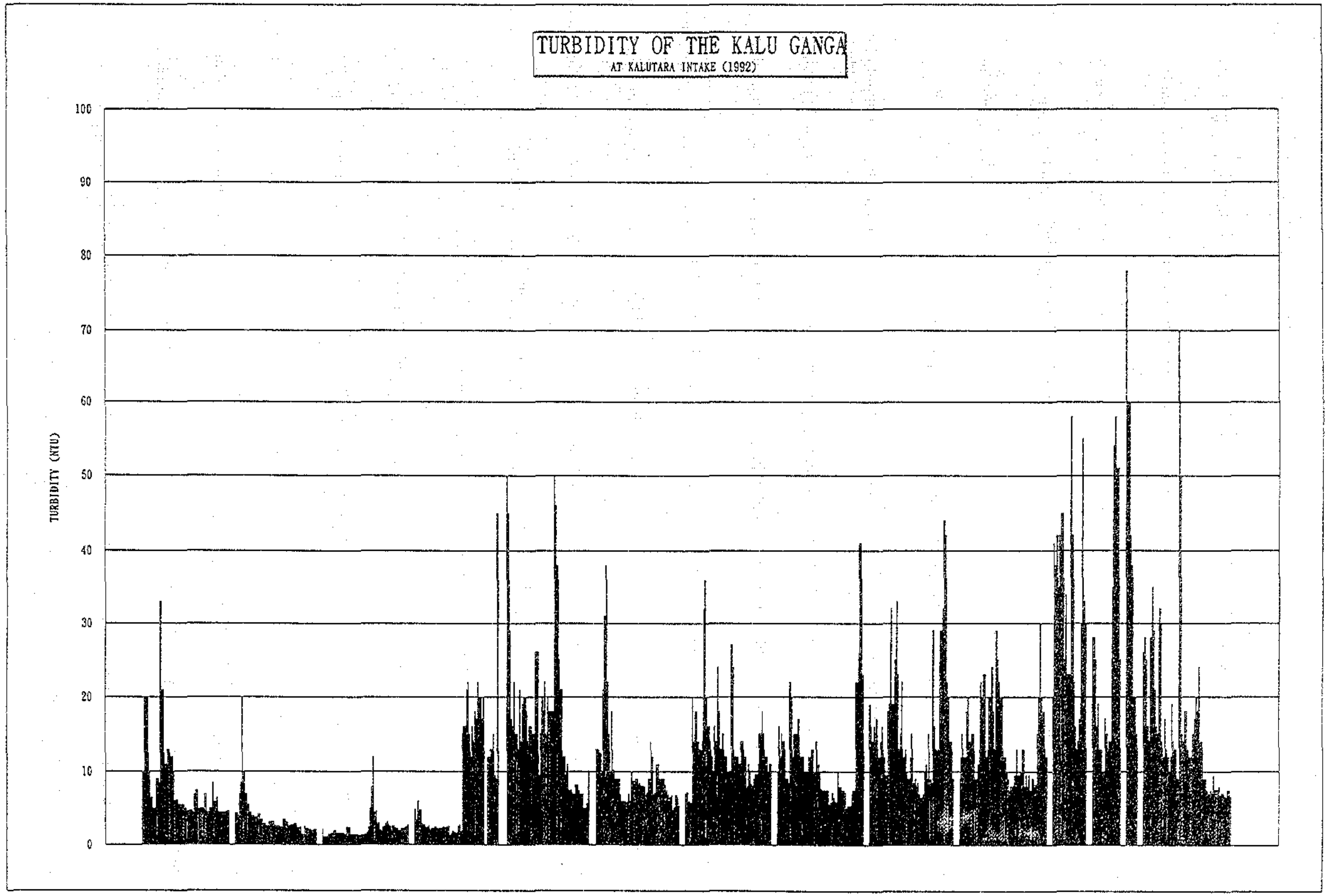

Figure 7.14 Turbidity Fluctuation (1992)

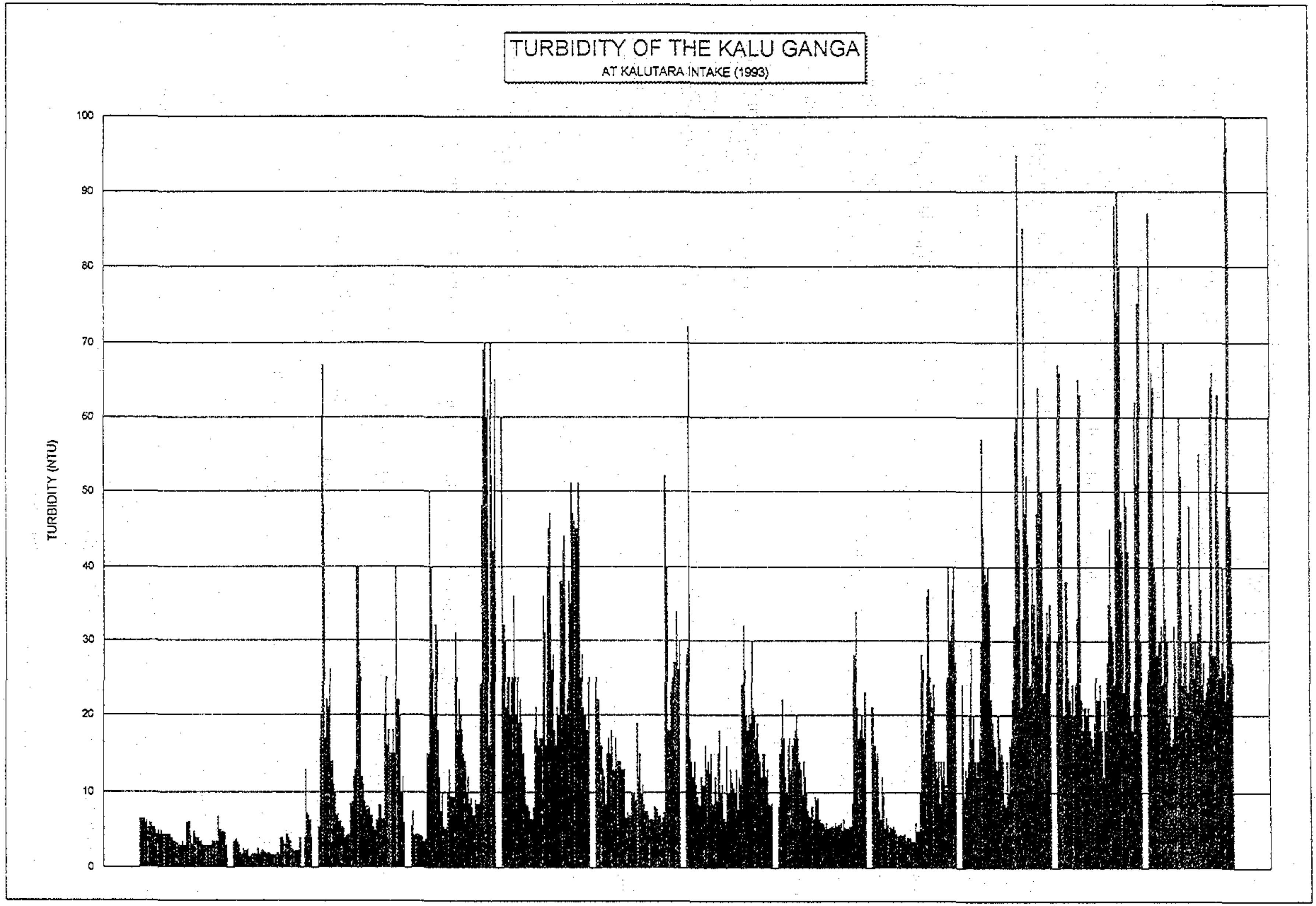

Figure 7.15 Turbidity Fluctuation (1993)

Table 7.11 Treatment Process Recommended in the NWSDB Design Manual

Water Quality Characteristic to be Reduced	Raw Water Storage	Screen-ing	Micro-strainers	Aeration	Pre-Chlori-nation	Prelimi-nary settle-ment	Floccu-lation	Coagu-lands and setting	Rapid filtra-tion	Slow sand filters	Post-chlori-nation	Superchlori-nation and dechlori-nation	Lime (-soda) sofening	Acti-vated carbon	Desalt-ing
Floating debris	-	E	R	-	-	-	-	-	-	-	-	-	-	-	-
Algae	-	-	R	-	R	-	-	E	E	-	-	-	-	-	-
Turbidity:															
0 - 5 TU	-	R	-	-	-	-	-	R	R	E	E	-	-	-	
5 - 30 TU	-	R	-	-	-	-	R	E	E	R	E	-	-	-	-
30 - 100 TU	-	R	-	-	-	-	R	E	E	R	E	-	-	-	-
100 - 750 TU	-	R	-	-	-	R	R	E	E	R	E	-	-	-	-
750 - 1000 TU	R	R	-	-	-	R	R	E	E	-	E	-	-	-	-
> 100 TU	R	R	-	-	-	E	R	E	E	-	E	-	-	-	-
Colour:															
< 30 Hazen	-	-	-	-	-	-				-	-	-	-	-	-
> 30 Hazen	-	-	-	-	-	-	E	E	E	-	-	-	-	-	-
Tastes & odours	-	-	-	R	R	-	-	-	-	R	-	R	-	R	-
Calcium carbonate															
> 200 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	E	-	-
Iron and manganese:															
< 0.3 mg/L	-	-	-	R	R	-	-	-		-	-	-	-	-	-
0.3 - 1 mg/L	-	-	-	R	-	-	-	E	E	R	-	-	-	-	-
> 1 mg/L	-	-	-	E	E	-	-	E	E	R	-	-	-	-	-
Chlorides:															
0 - 500 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
> 500 mg/L	-	-	-	-	-	-	-	-	-	-	-	-	-	-	E
Coliform bacteria, MNP per 100 mL:															
0 - 20	-	-	-	-	-	-	-	-	-	-	E	-	-	-	-
20 - 100	-	-	-	-	-	-	-	E	E	R	E	-	-	-	-
100 - 5000	-	-	-	-	R	-	-	E	E	R	E	-	-	-	-
> 50000	R	-	-	-	E	R	-	E	E	-	E	-	-	-	-
Toxic Chemicals	-	-	-	-	-	-	-	E	E	-	-	-	-	E	-

Source: NWSDB, Legend: E = Essential R = Recommended

2) Basic Approach for Selection of Unit Process

Rapid Mixing

The function of a rapid mix system is to disperse coagulant uniformly throughout the entire mass of water with maximum possible rapidity in order to ensure that the coagulation process is as effective as possible. Rapid mix units are located at the head end of the plant and are designed to generate intense turbulence in the raw water by either mechanical or hydraulic means.

The primary difference between mechanical and hydraulic rapid mixing devices is the manner by which they import turbulence in the incoming raw water. For mechanical rapid mixers the degree of turbulence is a function of the equipment's horsepower and is largely independent of flow, whereas the degree of turbulence for hydraulic mixers is measured by the loss in head and is dependent on flow. Mechanical mixers are generally proprietary devices whose major technical advantages are that mixing is not a function of flow and they are flexible in adjusting the degree of turbulence to suit particular treatment needs. However, this advantage is of little consequence in places where the equipment cannot be kept in repair and where skilled operators are unavailable to make necessary adjustments.

Hydraulic rapid mixers are designed for either of two types of flow conditions; open channel flow or pressure flow in pipes. When feasible, open channel flow in gravity channels is preferred, as such designs eliminate costly pipes and fittings, and can reduce the total capital cost of the plant. The general types of open channel hydraulic mixers are 1) hydraulic jump mixers, 2) flumes and 3) weirs.

Flocculation

Flocculation is the process of gentle and continuous agitation, during which suspended particles in the water coalesce into larger masses so that they may be removed from the water in subsequent treatment processes, particularly by sedimentation. Flocculation follows directly after the rapid mix process and, like rapid mixing, the agitation may be induced either by mechanical or hydraulic means.

Mechanical flocculators are preferred in the industrialized countries because of their greater versatility; that is , the speed of the mechanically operated paddles can be adjusted to suit variations in flow, temperature, or raw water quality. The principal elements of mechanical flocculator systems are agitator impellers, drive motors, speed controllers and reducers, transmission systems, shafts and bearings.

A more practical approach is to use hydraulic flocculators that do not require mechanical equipment, nor continuous power supply if gravity flow is available, and which can be built primarily from concrete, brick, wood, or masonry by local labor at relatively low cost.

In baffled channel flocculation, mixing is accomplished by reversing the flow of water through channels formed by around-the-end or over-and-under baffles. Baffled channel flocculators are limited to relatively large treatment plants, say greater than 10,000 m^3/day capacity, where the flow rates can maintain sufficient head losses in the channels for slow mixing without requiring that baffles be spaced too close together, which would make cleaning difficult.

Horizontal-flow flocculators with around-the-end baffles are sometimes preferred over vertical-flow flocculators with over-and-under baffles because they are easier to drain and clean; also, the head loss, which governs the degree of mixing, can be changed more easily by installing additional baffles or removing portions of existing ones.

Sedimentation

The sedimentation process in water treatment provides for the settling and removal of heavier and larger suspended particles from water. Most commonly, it is used for removal of flocculated particles prior to filtration. The removal efficiency in the sedimentation basin determines the subsequent loading on the filters and, accordingly, has a marked influence on their capacity, the length of filter runs, and the quality of the filtered water.

The two major classifications for the design of sedimentation basins are:

1. Horizontal-flow units
2. Inclined- plate unit
3. Up-flow units

The advantage of up-flow units are, by combining the pre-treatment process that precede filtration, that substantial savings can be realized in construction costs and manpower requirements. Up-flow clarifiers perform quite well under suitable conditions and skilled supervision, provided their hydraulic capacity is not exceeded. When up-flow clarifiers are overloaded, sludge escapes from the blanket in large volumes and clogs the filters, interfering with the entire treatment process.

For communities in small countries, horizontal-flow tanks without mechanical sludge removal are much to be preferred, because they require no importation of equipment, and labor for cleaning the tanks is readily available. Equally important, when horizontal-flow tanks are overloaded, most of the settleable solids will still be removed, so that the filters can continue to operate normally. Overloading of plants is a chronic condition in small countries because the financing of plant expansion rarely keeps up with demand.

1. Horizontal-flow Sedimentation

Horizontal-flow sedimentation is a gravity separation process in which a settling basin provides a quiescent environment that enables particles of specific gravity greater than water to settle to the bottom of the tank. A well designed horizontal-flow sedimentation basin can remove up to 95 percent of raw water turbidity following effective coagulation and flocculation; the remaining turbidity is removed in the filters.

Rectangular horizontal-flow clarifiers without mechanical sludge removal are advantageous for small- or medium-scale plants because of their simplicity and ability to adapt to various raw water conditions, such as sudden changes in turbidity, flow increases, or too-high flow rates.

Circular-shaped basins are not appropriate, and their main advantage over rectangular basins is where circular mechanical sludge removal equipment is to be used.

In the communities in small countries, manual sludge removal is preferred over the importation of mechanical sludge removal equipment, because the latter is difficult to maintain under the technical and climatic conditions prevent in those countries, and it is more costly than employing labors to clean the tanks manually.

2. Sedimentation with Inclined-plates

When inclined-plate is installed in either up-flow sludge blanket-type clarifiers or horizontal-flow basins, the unit can improve clarifier performance and increase the capacity of conventional clarifiers by 50 to 150 percent. Furthermore, it may also be incorporated into the design of new sedimentation basins, reducing the settling area to one-fourth to one-sixth of that required by conventional basins.

However, for communities in small countries, the use of inclined-plate should be limited, in most cases, to expand settling basin capacity and/or improving plant effluent quality and the use may be limited in hot and sunny climates where algae growth on plates can be a troublesome maintenance problem. The incorporation of settlers in the design of new plants for communities in small countries to reduce basin size and cost is usually not justified, because land is generally not restricted and low-cost labor and materials are available for construction of the settling tanks.

Further, when conventional settling tanks are installed during initial plant construction, the option remains for installing inclined-plate in the future.

3. Up-flow Sedimentation

Combining sludge-blanket type clarifiers with flocculation may be an appropriate technology for larger plants in urban centers. These clarifiers have no moving parts and, except for a few valves, require no mechanical equipment. They may be appropriate under the following conditions:

- o Relatively constant raw water quality with turbidity not less than 10 NTU and not exceeding 900 NTU,
- o Plants that are designed with enough excess capacity so that the unit process will not be overloaded, and
- o Availability of skilled supervision

In general, the surface loading rate and detention time, though they may be variant depending on the types such as slurry circulation type, sludge blanket type including palpitation type and compound type, are likely to be 50 - 85 m/day and 1.5 - 20 hr.

So far as the investigation with respect to the sedimentation units in the existing treatment plants at Labugama, Kalatuwawa and Ambatale, problems such as deficit of chemicals, inadequate control or non control of influent raw water quantity, lack of measuring devices and inaccurate recording of data are outstanding. Especially, non-control of inflow quantity will do harm not only on settlement effectiveness but also clogging subsequent filter media resulting in unpreferable treated water quality, whatever the sedimentation process is adopted. Taking into account this fact, horizontal-flow units principally are preferred.

Advantages of horizontal-flow units over up-flow units.

- o The process more tolerant of hydraulic and quality variations.
- o The process gives predictable performance under most operational and climatic conditions.
- o The process is easy to expand.
- o The process works exceptionally well when silt loads are very high.
- o Construction costs are low, permitting oversizing.
- o Operation and maintenance are simple.

Table 7.12 lists recommended surface loading rates (settling velocities) and detention period for various conditions likely to be encountered in practice.

Table 7.12 Design Guidelines for Horizontal-Flow Settling Basins

Type	Description	Surface Loading Rate (Settling velocity) (m/day)	Detention Time (hr)
A	Small installations with precarious operation	20 - 30	3 - 4
B	Installations planned with new technologies and reasonable operation	30 - 40	2.5 - 3.5
C	Installations planned with new technologies and good operation	35 - 45	2 - 3
D	Large installations with new technologies and excellent operation, with provisions for adding coagulant aids whenever necessary	40 - 60	1.5 - 2.5

Source: Azevedo-Netto, 1977., Properly designed hydraulic rapid-mix and flocculation units.

Filtration

Filtration is a physical, chemical, and in some instances biological process for separating suspended impurities from water by passage through porous media. Two general type of filters are commonly used in water treatment: the slow sand filter and the rapid sand filter. Because of higher filtration rates, the space requirement for a rapid filtration plant is about 20 percent of that required for slow sand filters.

The design variables for rapid filtration include 1) type of filter rate control, 2) type of filter media, 3) filter bottoms and underdrains, 4) backwashing arrangements, and 5) auxiliary score wash systems. The principal consideration for 1), 2), 3) and 4) is given as follows:

1. Type of filter Rate Control

There are two ways of filtration, namely constant-rate filtration and declining-rate filtration. Applying these ways of filtration, only four types of filtration-rate control system have been often introduced and performed properly. These types are:

(a) Constant-rate filter with a flow meter and a flow control valve

(b) Constant-level filter with an influent weir, a level sensor and a control valve

(c) Rising-level filter with an influent weir and without control and backwash valve

(d) Declining-rate filter with a flow restriction and without an influent weir

Among these types, (a) to (c) are categorized into constant-rate filtration, while (d) into declining-rate filtration

The first constant-rate filter is highest in its cost and skill for maintenance due to precise devices among four types and has been losing its popularity. Another disadvantage of this type is that, when the control system is damaged, the filter is changed to the declining-rate filter, particularly at water treatment plants in developing countries.

The second constant-level filter is secondly highest in its cost and skill for maintenance.

The third rising-level filter is capable of backwashing a filter using the rest of filters in the same train and is referred to as a self-backwash filter. This type is always highly evaluated in developing countries because of its limited use of mechanical equipment such as backwash pumps and valves.

The fourth declining-rate filter is very simple to design and build and generally produces the good quality of water, if the initial filtration rate at the beginning of filtration cycle is properly controlled by an orifice etc. The disadvantage of this filter is that there is no clear indication for filter backwashing which is regularly done based on the duration time of filtration, even though the filter is still clean.

Through the above studies, the rising-level, constant filter with a self-backwash system is recommended.

2. Type of Filter Media

Sand has been used traditionally as the filter medium in water treatment plants because of its wide availability, low cost, and the satisfactory results that it has given. Sand filters remain the predominant method of filtration in many countries.

Dual-media filters possess several distinct advantages over conventional sand filters, 1) higher filtration rates (10 to 15 m/hr) than for conventional filters, resulting in a reduction in the total filters area and cost for a given design capacity; 2) longer filter runs at any given loading; and 3) the capacity of existing sand filters can be easily increased at low cost by the conversion to dual-media beds.

The latter advantage may be exceedingly beneficial to those communities in developing countries that are burdened with overloaded and inefficient treatment plants. Dual-media beds can be incorporated into an existing filtration system without changes in plant structure or method operation if the hydraulic capacity of the influent and effluent piping is adequate.

The cost for converting plants from single-media sand filters to high-rate dual-media filters was estimated in India at only US\$ 0.8/m^3/day of plant capacity, or US\$ 0.4/m^3/day of increased capacity (Ranade and Gadgil, 1981). The cost estimates included the cost of modification in the influent and underdrain system, and the cost of placing new media.

Most dual media filters in use today in the United States, the top anthracite layer has an effective size of 0.8 - 1.2 mm, a uniformity coefficient of less than 1.85, thickness of a few inches to two thirds of the total filter thickness of 0.6 - 0.75 m.

While, the design criteria for the NWSDB shows that pilot testing in Sri Lanka using a layer of 50-60 cm coconut charcoal (specific gravity 1.42) of 0.83-1.4 mm size over a 25 cm layer of sand (0.5-1.4 mm) has shown good results. However, adoption of dual media itself is still likely a testing level in Sri Lanka.

As a conclusion, the filter shall be a conventional type at this stage of the Interim Report leaving an availability for conversion with a dual media later by the next stage.

3. Filter Underdrain

The major requirements of the filter underdrain system are the support of the filter media and the uniform distribution of the waste water across the entire filter bed. In many instances, bottoms can be reinforced concrete slabs with plastic strainers, precast concrete percolated block, or glass-tube orifices, or simple perforated-pipe lateral systems.

The precast concrete perforated underdrain systems, as commonly adopted, is constructed with long reinforced concrete V-blocks that are inverted and supporting graded gravel layers. The advantage of this underdrain are simplicity of design and low head loss during back washing.

The perforated pipe lateral system consists of a central manifold pipe to which are attached a series of lateral pipes with orifices to distribute waste water or to collect the filtered water. The losses through the orifices are kept comparatively high (about 1 to 3 m) to maintain uniform distribution of backwash water.

The number and depths of the gravel layers, both to support of the media during filtration and to contribute to uniform distribution of backwash water, depend on the size of openings in the underdrain. Larger openings require a deep gravel layer, and hence a deeper filter box, but have reduced head loss during backwashing. Smaller openings, such as the slots in nozzles, require only the finer gravel layers but have higher head loss.

4. Backwash Arrangements

The purpose of backwashing is to remove the suspended material that has been deposited in the filter bed during the filtration cycle. When a filter is backwashed, an upward flow is introduced at a rate sufficient to fluidize the filter media. Rates of backwash need to be high enough to fluidize all the filter media, but not to be higher. The percent of expansion that accompanies any rate is a function of the size and specific gravity of the media, and temperature of the water.

Most of the head loss occurs in the underdrain system (1 to 3 m), although self-backwash or interfilter-washing units are designed with relatively low head losses in the underdrains (20 to 30 cm).

Three types of backwash arrangements that are suitable in this project will be 1) elevated waste water tanks, 2) taking waste water from the high-pressure distribution system tank, which is wasteful of energy but lower in installation costs, and 3) self-backwash or interfilter-washing units. Further study regarding selection of these will, after arrangement of surveys and subsequent layouts of the facilities, be carried out.

7.5.4 Clear Water Transmission

(1) Planning Concept

Water transmission pipelines will be constructed to convey treated water from the new treatment plant at Horana to the service area. In consideration of the transmission system, alternatives are prepared with the concept as follows:

1) Long term development plan is prepared for the 2020 demand. However, it is not practical to construct the facilities meeting the 2020 demand in one phase. It may be a normal practice to design the facilities for around 10 to 15 year future taking into account the .reliability in projection for long span of time. Projection must be reviewed every 15 to 20 years.

   From this point of view, facility planning is prepared applying two step construction for the 2010 and 2020 demands, respectively.

2) As recommended in the previous sub-section, the clear water transmission will be made by gravity from a ground reservoir (High Level Reservoir) to be constructed on the peak of the hill at Horana.

3) Transmission pipeline will be laid in the Piliyandala Road (B5) connecting Horana and Colombo considering its rather straight alignment and shorter length than the other minor roads.

4) Moratuwa area will be supplied from a branch pipeline from Piliyandala to minimize the transmission distance to Moratuwa.

5) Panadura will be supplied either from Moratuwa or by a pipeline along Ratnapura Road (A8) which will also pass Bandaragama service area.

6) Supply to Dehiwala, Moratuwa, and Panadura will firstly be made to the existing reservoirs or towers in each area.

7) Transmission will be made by gravity as much as possible to the reservoirs or towers for using the energy given to the water at the intake and high lift pumping system and for avoiding two or more step pumping. System with less pumping stations will result in the great advantage in ease of operation and cost saving not only in pumping cost but also in maintenance cost for pumping stations.

(2) Optimum Elevation of High Level Reservoir

Pipe diameters of the transmission pipeline will be dependent on the water level in the High Level Reservoir which will be constructed for gravity transmission from Horana. Consideration are given to the combination of pipeline diameters and water level in the High Level Reservoir. The higher the High Level Reservoir is located, the smaller the pipe diameters will be due to larger water head available for gravity flow. On the other hand, pumping cost from the treatment plant to the High Level Reservoir will become higher for larger head pumping.

A comparison is made for different water levels in the High Level Reservoir. Construction cost and pumping cost calculated in net present value is summarized below:

Table 7.13 Summary of Cost Comparison

Water Level in High Level Reservoir	Construction Cost of Pipeline (Res.'s) (1)	Pumping Cost (NPV for 50 years operation) (Rs.'000) (2)	Total Cost (Rs.'000) (3)=(1)+(2)	Ratio (Minimum as 100)
+55.0 m	3,958,360	307,206	4,265,566	160
+65.0 m	3,067,360	368,647	3,436,007	129
+75.0 m	2,445,560	430,088	2,875,648	108
+85.0 m	2,312,560	491,529	2,804,089	105
+95.0 m	2,198,560	552,971	2,751,531	103
+105.0 m	2,046,560	614,412	2,660,972	100

As shown above, the higher the water level of the high level reservoir, the lower the total cost. This means that to locate the reservoir at higher elevation will be more economical than having smaller pumping head. This is resulted from the fact that the substantial construction cost will be required for the pipe construction compared to the pumping cost.

It is therefore recommended to locate the High Level Reservoir at higher elevation than 100 m MSL.

(3) Alternative Routes and Diameters

On the basis of the fundamentals as described above, two transmission alternatives are proposed for comparison as follows:

Transmission Alternative TA-1

Dual pipeline system for 2010 and 2020 demands
Panadura supplied through Moratuwa

Transmission Alternative TA-2

Dual pipeline system for 2010 and 2020 demands
Panadura supplied through Bandaragama

Layout of each alternative are shown in Figures 7.16 and 7.17.

(4) Cost Comparison of Alternatives

Diameters, lengths of pipelines for each alternative are determined from the hydraulic model analysis assuming the gravity flow to Dehiwala Reservoir G4, Moratuwa Reservoir G5 and Panadura Reservoir G7 and Tower G12, respectively.

The result of comparison are shown in Table 7.14.

Table 7.14 Construction Cost Comparison for Alternatives

	1993 base	Alternative TA-1		Alternative TA-2	
	Unit Cost (Rs./m)	Length (m)	Cost (Rs.mil.)	Length (m)	Cost (Rs.mil.)
1st Phase (for 2010 demand)					
1200	56,000	22,350	1,252	22,350	1,252
1100	48,000	9,650	463		
1000	40,000	4,800	192	9,650	386
800	30,000			4,800	144
500	15,000	10,500	158	15,250	229
250	6,000	5,450	33	1,000	6
Sub-Total		52,750	2,097	53,050	2,016
2nd Phase (for 2020 demand)					
1200	56,000	22,350	1,252	22,350	1,252
1100	48,000	9,650	463	9,650	463
Sub-Total		32,000	1,715	32,000	1,715
Total Cost			3,812		3,731

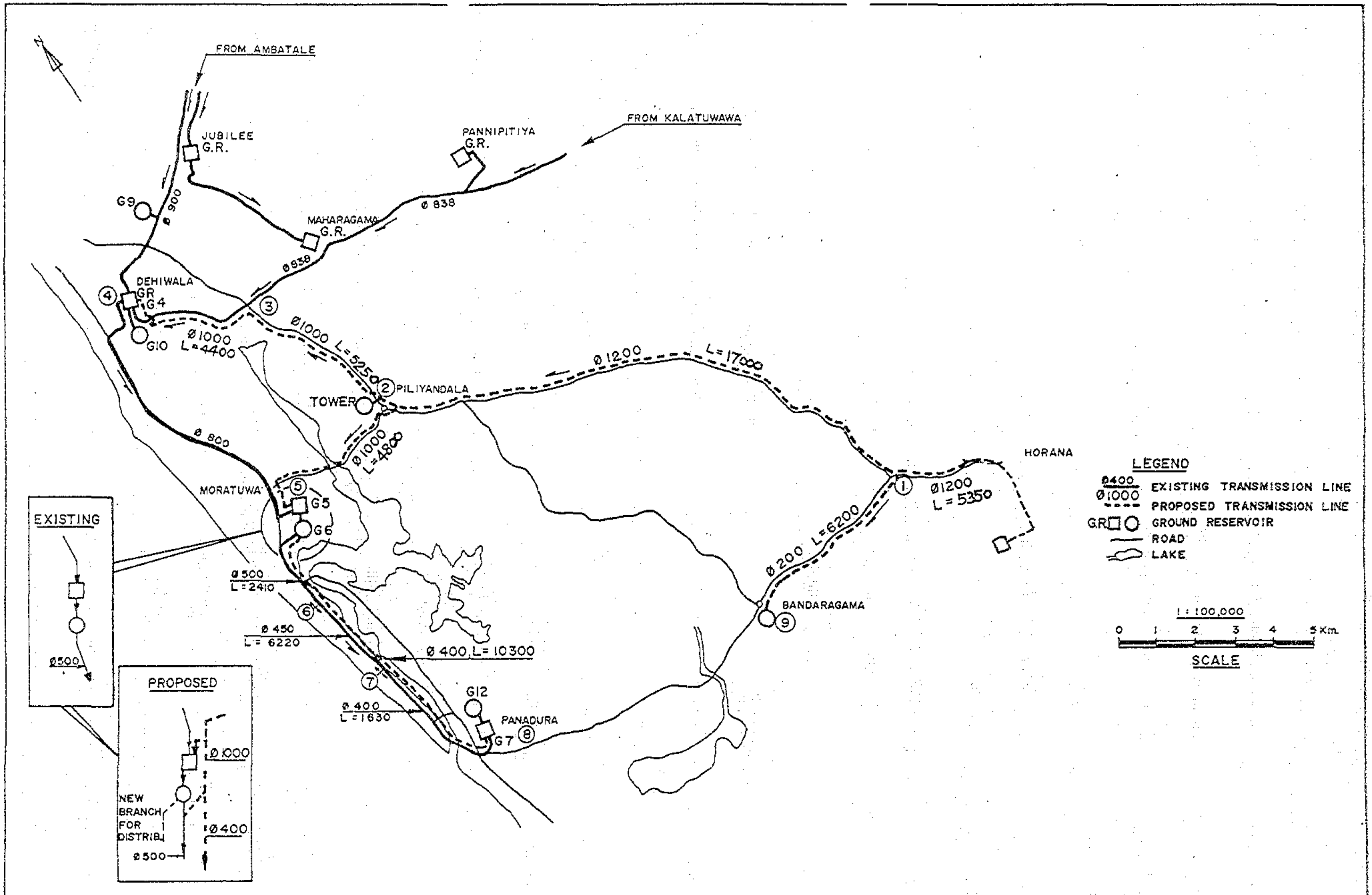

Figure 7.16 Transmission Alternative TA-1

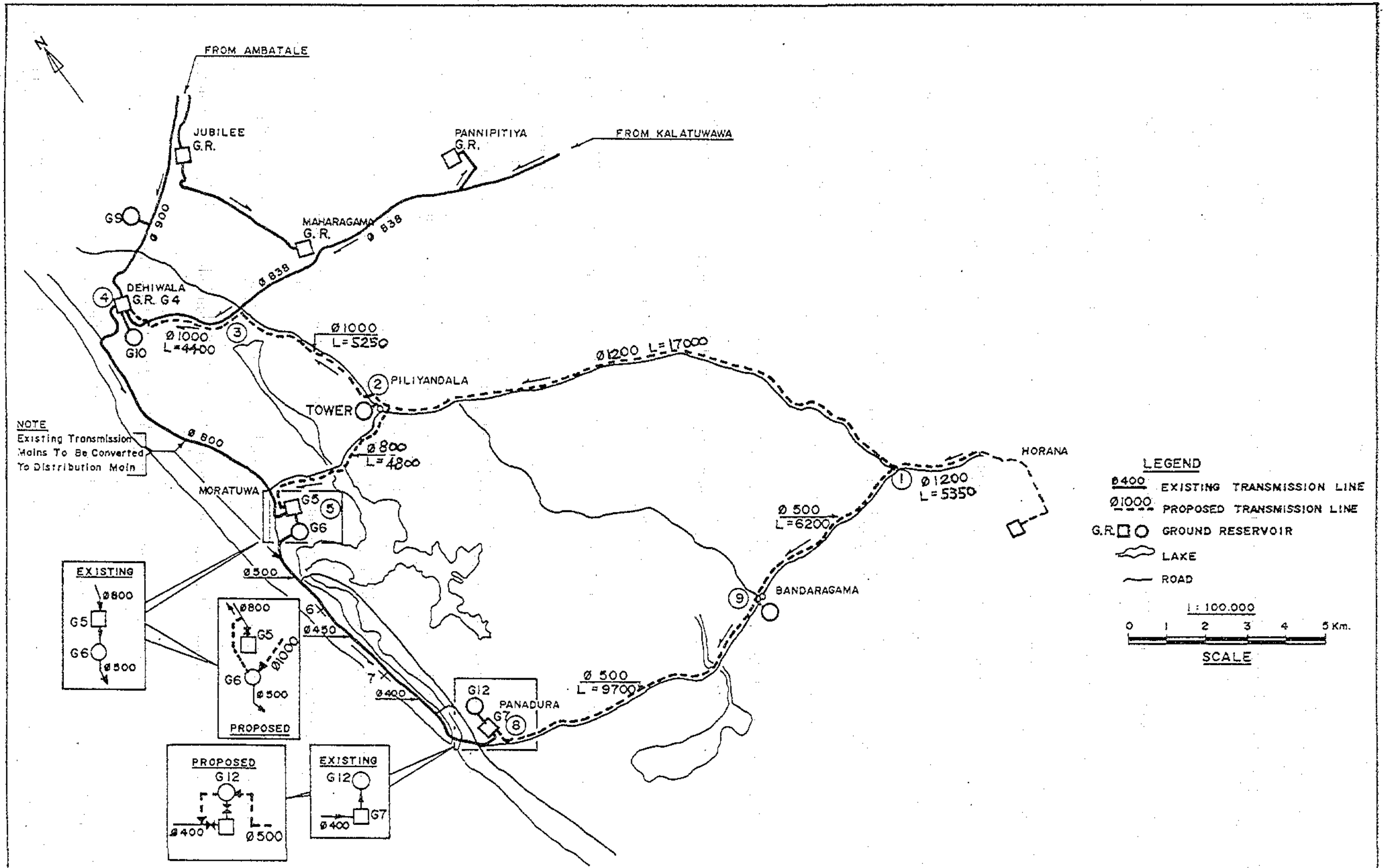

Figure 7.17 Transmission Alternative TA-2

As shown in the table above, Alternative TA-2 shows lower construction costs. TA-2 also has an advantage that it will directly supply to Panadura so that the existing Moratuwa-Panadura transmission pipeline will be used as a distribution main. Operation costs for two alternative have no difference. Alternative routing TA-2 is therefore recommended.

(5) Transmission to Each Service Area

In the recommended transmission alternative TA-2, transmission to each service area is planned as follows:

1) Transmission to Dehiwala

Water will be conveyed to Dehiwala reservoir by gravity. Transmitted water will be supplied to Dehiwala Low Zone as proposed in section 7.6.5 by gravity from the reservoir. Pumping to the existing water tower G10 will be made for distribution to Dehiwala High Zone. Distribution to C.M.C. will also be made by gravity from the C.M.C. reservoir which will be interconnected with the reservoir G4.

2) Transmission to Moratuwa and Keselwatte

Water will be conveyed to Moratuwa by gravity through a 800 mm branch pipe extended from Piliyandala. Water will be gravitated to the existing Moratuwa reservoir G5 and to a proposed water tower which will supply to Moratuwa Low Zone as explained in section 7.6.5.

Water received in the reservoir G5 will be pumped to Tower G6 for distribution for Moratuwa High Zone and to a new tower at Keselwatte for distribution to Keselwatte service area.

3) Transmission to Panadura and Bandaragama

Transmission to Panadura will be made through a 500 mm branch pipe along Ratnapura Road (A8) extended from Pokunuwita Junction. Water will be gravitated to the existing Panadura reservoir G7 and tower G12.

Transmission to Bandaragama will be made by tapping to the 500 mm main.

4) Transmission to Kesbewa Main Area

Water for Kesbewa Main Area will be transmitted directly to a new water tower to be constructed at Piliyandala. The water tower will be connected to the new 1200 mm main constructed along Piliyandala Road (B5).

5) Transmission to Horana

Water transmission to Horana will be made to the existing Horana reservoir by a 200 mm branch pipe from the 1200 mm transmission main. This transmission will be considered as supplemental measure to the existing Horana water supply scheme which is presently taking water from the Kalu Ganga through infiltration gallery and by pumping main. Tapping to the new transmission main in future is therefore recommended as necessary basis when the water demand increases and the intake capacity will not catch up the demand.

7.5.5 Pipe Material

(1) General

In the past projects of the NWSDB, several types of pipe material have been used. Included therein are cast iron, ductile iron, mild steel, asbestos cement, polyvinyl chloride (PVC), galvanized iron etc. At present, ductile iron pipe (DIP) is used for most of the medium and large diameter pipes of 250 mm or larger. DIP is imported from UK, Japan, France and Korea. PVC pipe is used for the small diameter pipes less than 250 mm.

(2) Transmission Pipeline

For the proposed transmission mains, ductile iron and steel pipes are considered as alternative materials. Their advantages and disadvantages are compared as shown in Table 7.15.

Total costs of DIP are lower than SP for the diameters 1200 mm or smaller. Considering the cost difference and necessity of thrust blocks for DIP in large sizes, SP is recommended for the diameters more than 1200 mm.

(3) Distribution Pipeline

DIP is recommended for the distribution pipes with diameters of 250 mm or larger as presently adopted in the NWSDB practice. The most attractive advantage of DIP is an ease of installation (both for laying and jointing) which will ensure tight joint.

PVC pipe is locally manufactured and considered as one of the suitable pipes for small diameter in terms of cost and characteristics. PVC is therefore recommended for smaller diameter distribution pipeline.

Table 7.15 Comparison of Ductile Iron and Steel Pipes

Item	Ductile Iron Pipe (DIP)			Steel Pipe (SP)		
Durability	Wall thickness is larger than SP. Durability is generally better than SP.			Need proper coating and lining to prevent corrosion.		
Joint Work	Push on/flange joints. Easy to ensure tight joint.			Welded/coupling/flange joints. Need skillful welders and careful testing for completeness of welding. Welding work is affected by weather.		
Pipe Bedding	Pipe is more rigid than SP. Normally selected excavated material may be used for bedding and backfill.			To prevent deformation, bedding and backfill should be made carefully. Normally sand should be used for bedding and backfill.		
Thrust Block	For push-on joint, thrust blocks are needed at bend and tee to prevent pull off. For large diameter pipes, large size of thrust block should be provided.			With welded joint, thrust block is normally not needed. Welded joints form a series of pipes as one continuous structure.		
Others	Push-on joint has flexibility for bend. Small curving can be made without fittings to some extent.			Butt welding joint need a very neat treatment (cutting and grinding) on joint face.		
Cost dia.(mm) (Rs. /m)	Material (CIF)	Laying	Total	Material (CIF)	Laying	Total
800	28,750	5,950	34,500	29,375	13,220	42,545
900	35,000	7,000	42,000	32,500	14,630	47,130
1000	43,750	8,750	52,500	41,250	18,560	59,310
1200	60,000	11,750	70,500	56,250	25,310	81,560
1350	75,000	16,250	97,500	65,000	29,250	94,250
1500	87,500	18,750	112,500	75,000	33,750	108,750
1650	117,500	23,500	141,000	91,250	41,660	132,910

7.6 Kalu Ganga Water Supply System Recommended

7.6.1 Outline of the System Recommended

Through a comparative study on the possible alternatives, the following system is finally recommended for the Kalu Ganga Water Supply Project:

The raw water will be taken from the Kalu Ganga at the intake station to be constructed at Udugammana Village, Madurawala, downstream of the confluence of the Kalu Ganga and the Kuda Ganga. The raw water will then be pumped up to the water treatment plant after screening and grit removal.

The water treatment plant will be constructed on the flat plateau with an elevation of +15 m MSL along Road A8 about 3.5 km south of Horana U.C. The raw water will have a treatment of chemical coagulation, sedimentation and rapid sand filtration. The sludge generated in the sedimentation basins will be withdrawn therefrom for treatment in the sludge lagoons. Backwash drain from the filters will be returned to the receiving well to minimize the discharge to the nearby watercourse. The treated clear water will be lifted up to the distribution reservoir to be constructed on the hill with an elevation of about +90 m MSL and approximately 2.8 km northwest of the water treatment plant.

The clear water will be directly transmitted by gravity from the new Horana Reservoir to the existing Dehiwala Reservoir (G4) via Roads A8 and B5 and to the existing Moratuwa Tower (G6) branching from Piliyandala through Piliyandala-Moratuwa road. A small branch to the newly constructed existing Bandaragama Tower via Road A8 from the main transmission main. The existing Panadura Tower (G12) will be supplied the clear water by the transmission main after supplying the Moratuwa Tower (G6).

7.6.2 Intake Facilities

The layout of the intake station are shown in Figure 7.18.

The raw water will be normally taken from the Kalu Ganga through the intake mouth with a bottom elevation of -2.900 m MSL. The stop log will be provided with the intake mouth so as to raise the bottom elevation of the intake mouth to -0.90 m MSL at the time of salinity intrusion of which the water level is estimated at -0.92 m MSL at the location of the intake mouth under the condition of an annual minimum river flow of 14.4 m^3/sec with a return period of 10 years and the intake amount of 4.2 m^3/s (364,000 m^3/d) for 2020 water demand at the high tide.

The raw water will be introduced through the screen and intake gate the grit chamber which will be covered with concrete and soil for safety and prevention of the leaves from entering. The inlet and outlet gates will be provided with each chamber.

The raw water will be then pumped up to the water treatment plant after grit removal. In the intake pumping station the pump motors and electrical equipment will be installed on the first floor with an elevation of +10.00 m MSL against the flood.

7.6.3 Treatment Facilities

(1) Proposed Optimal Unit Process

1) Receiving well

The principal function of receiving well is to stabilize fluctuation of water level and regulate raw water quantity flowing into the plant to enable easy and appropriate dosing, sedimentation and filtration. For stabilization of water level, adequate baffling devices, surface area and detention period are required together with quantity measurement equipment. The rectangular type of receiving well is proposed for convenience of construction at this stage of Interim Report. The retention time and effective depth shall be 1.5 min and 5.0 m, respectively.

2) Mixing Basin

The mixing basin shall be selected amongst hydraulic jump mixers, flumes or weirs for ease of operation and maintenance. Of these weir type shall be selected at this stage. The rapid mixer shall be a channel type decreasing the width forwarded to the rectangular weir. The dimension is tentatively determined as 2.5 m, 1.0 m and 10 m for width, depth and length, respectively. Mixing time of 5 second on the basis of design criteria of the NWSDB and a retention time of 2 min in the basin may be suitable.

3) Flocculation Basin

Horizontal-flow baffled channel (around-the-end) type may be appropriate on operation and maintenance view point and ease of cleaning. The retention time of 25 min on the basis of design criteria of the NWSDB may be suitable. The effective depth shall be 3.5 m.

4) Sedimentation Basin

Horizontal-flow type sedimentation is selected tentatively at this stage due to ease and cost on operation and maintenance and availability of further improvement in the future when needed. The water surface road shall be 25 mm/min and effective depth may be suitable with 4.0 m.

5) Rapid Sand Filter

For filtration, rapid sand filter with a rising-level, constant rate and conventional media shall be selected as stated in the previous sub-section. The filtration rate may be suitable by adopting lower value of design criteria of the NWSDB, subsequently resulting in 120 m/day. The filter shall be self-backwashed plus surface wash at this stage.

(2) Computation of Preliminary Design Capacity and Required Area

Design parameters and capacity of the facilities together with required area computed are summarized in Table 7.16. For determining design capacity, 5 percent of allowance and, in like manner, 20-50 percent for the required area, are taken into consideration. The required area for a water treatment plant resulted in approximately 10 ha in total.

(3) Layout of the Treatment Plant

Layout of the proposed treatment plant is shown in Figure 7.19.

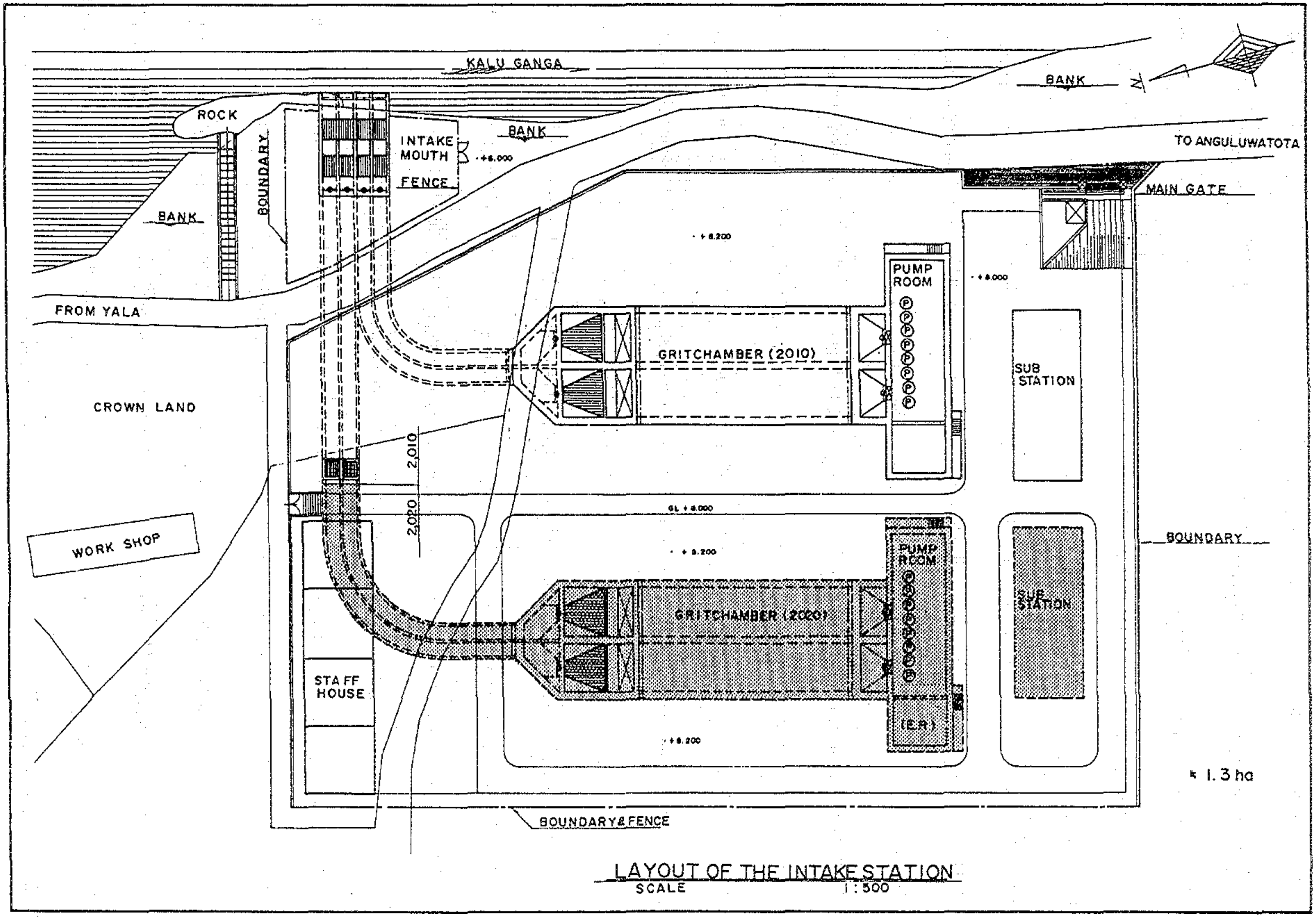

Figure 7.18 Layout of the Proposed Intake Station

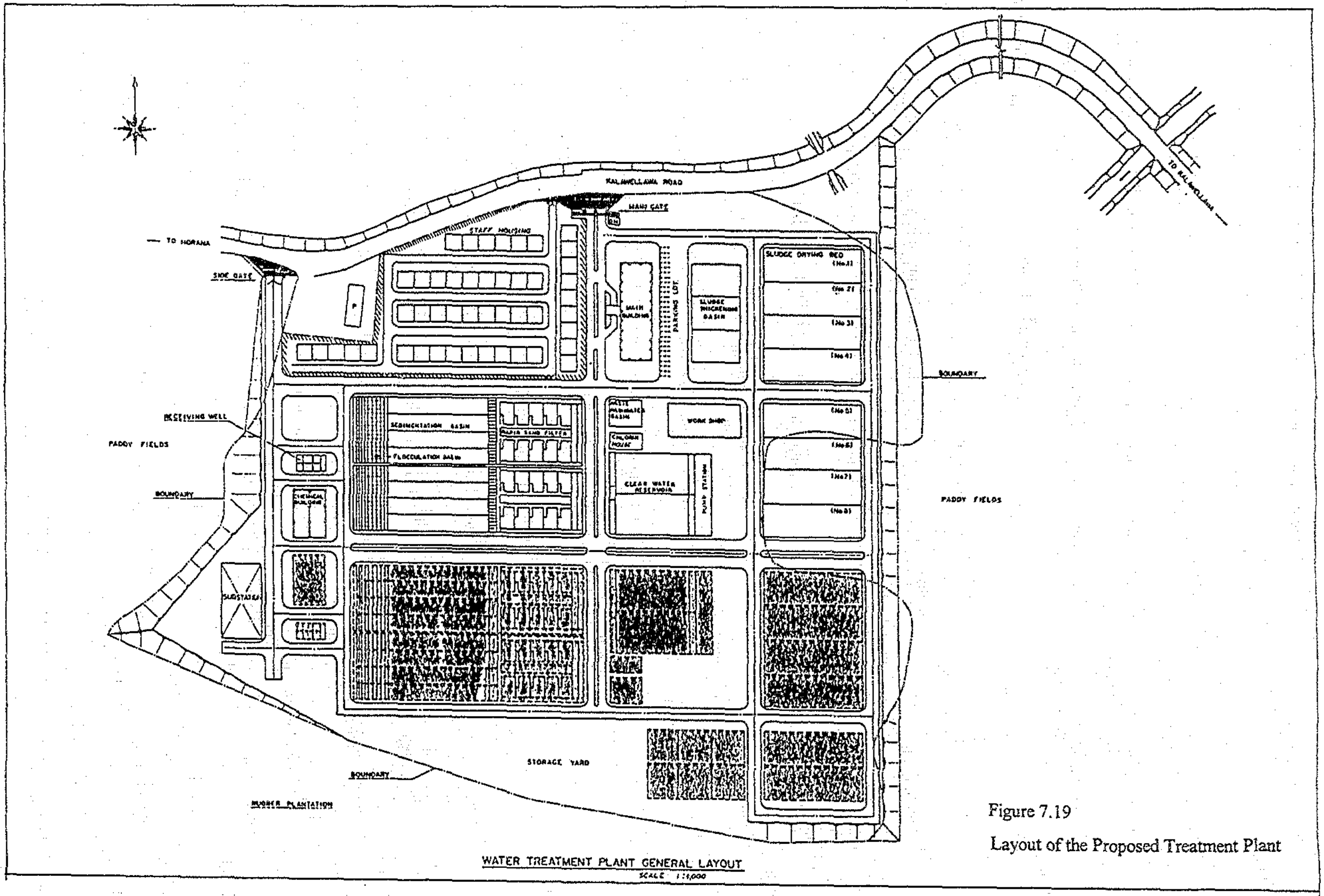

Figure 7.19

Layout of the Proposed Treatment Plant

Table 7.16 Design Parameters of Water Treatment Plant

Facility	Design Parameter	
Receiving Well	Retention Time	2 min
Mixing Chamber	Type Mixing Time Retention time	Hydraulic Mixing 2 sec 2 min
Flocculation Basin	Type Retention Time GT	Baffled Channel 25 min 50,000
Sedimentation Time	Type Retention Time Surface Loading Depth Overflow Rate	Horizontal Flow 2.5 hours 25 mm/min 4 m 500 m3/m/d
Rapid Sand Filter	Type Filter Media Filtration Rate Filter Wash	Rising-level, Constant Rate Sand 120 m/d Self-backwash plus Surface Wash
Chlorination Chamber	Retention Time	2 min
Clear Water Reservoir	Retention Time	1 hour

7.6.4 Transmission Facilities

(1) Transmission Pipeline

System diagram and hydraulic profile of the transmission system required for the development of the Kalu Ganga Water Supply Project are shown in Figures 7.20 and 7.21. Component of pipeline are summarized in Table.7.17.

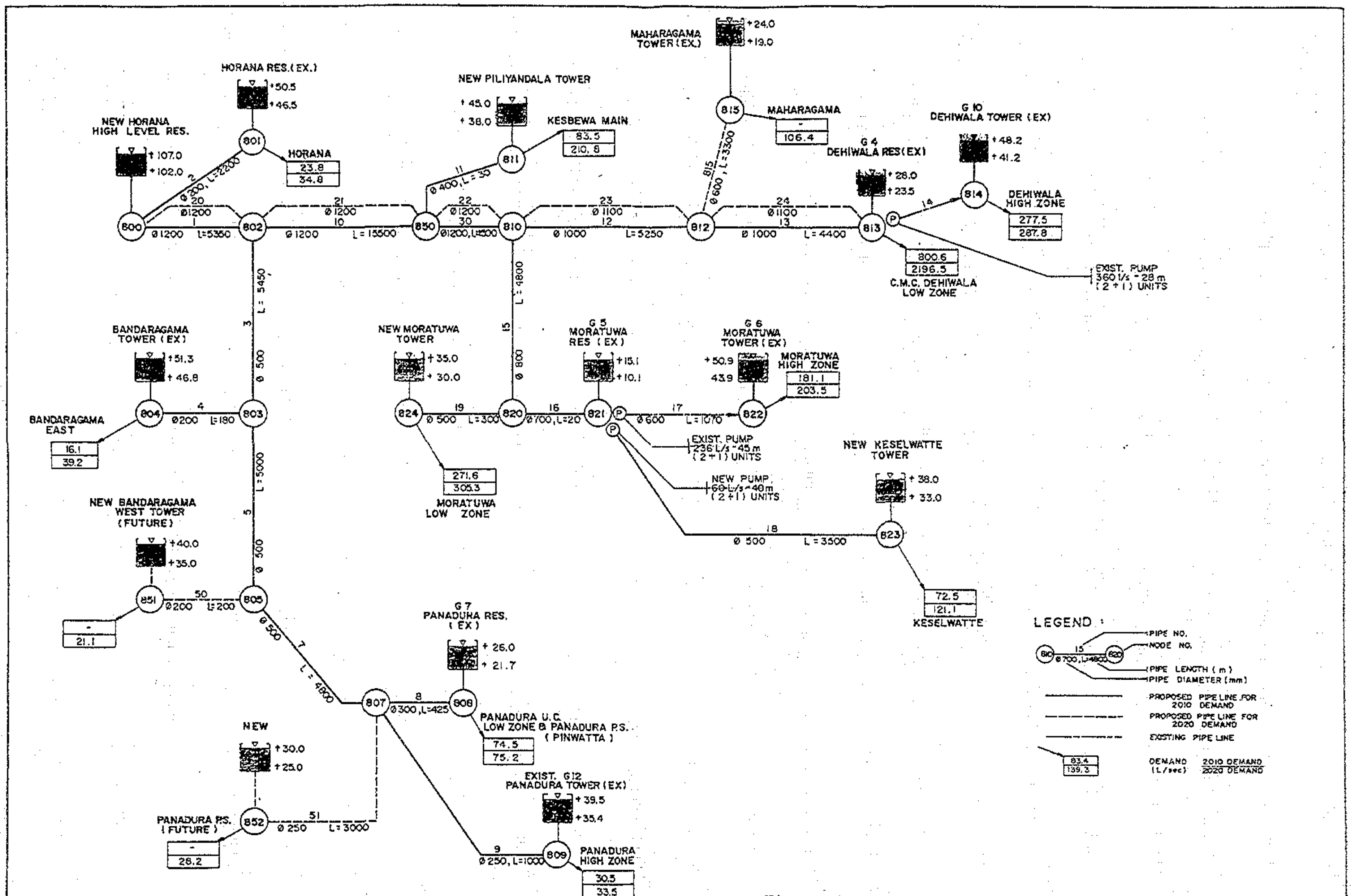

Figure 7.20 System Diagram of Proposed Transmission Stystem

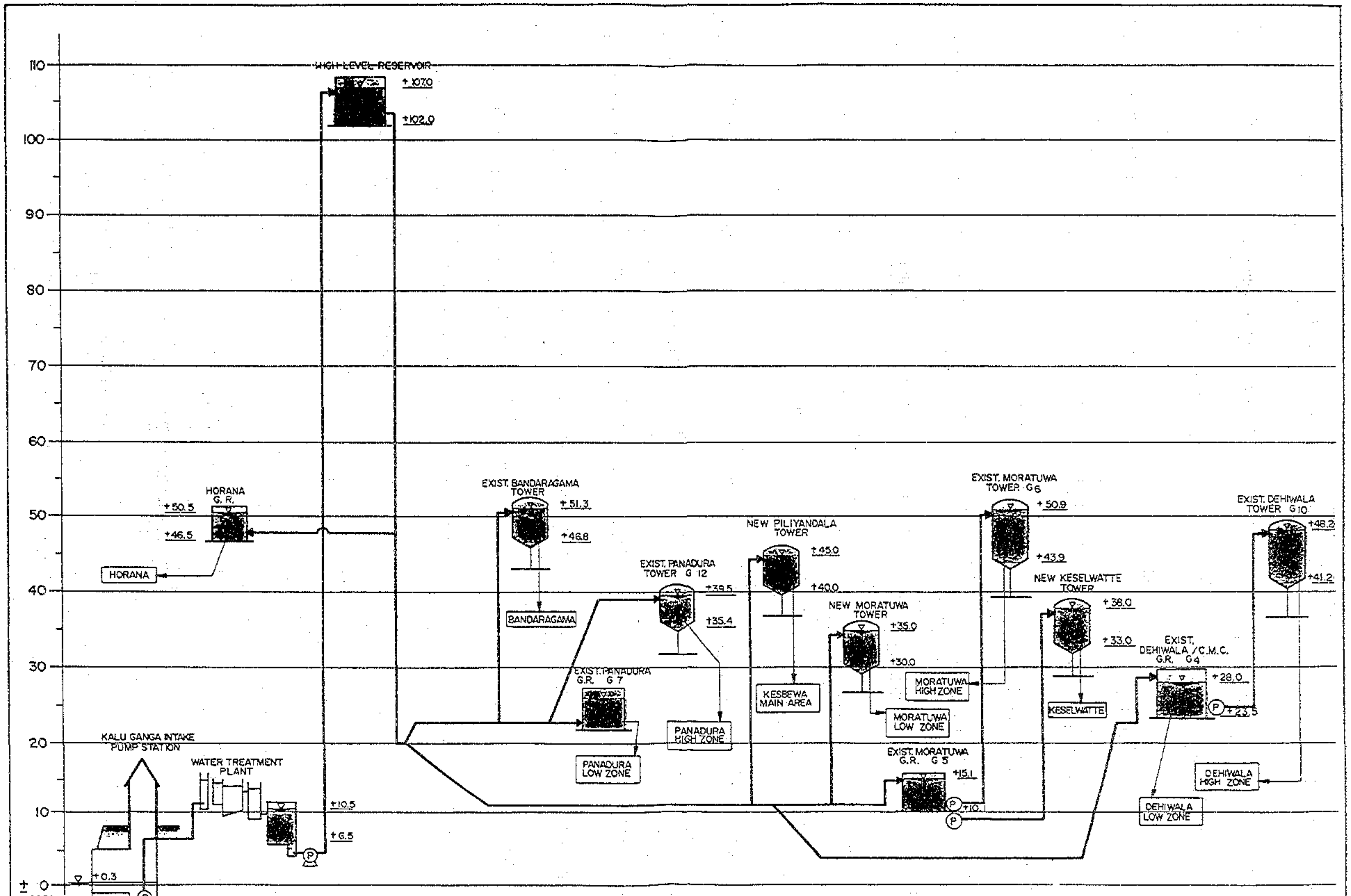

Figure 7.21 Hydraulic Profile of the Proposed Transmission System

Table 7.17 Summary of Transmission Facilities

Pipeline	Diameter (mm)	Material	for 2010 Demand Length (m)	for 2020 Demand Length (m)
Raw Water Transmission				
Intake to WTP	1500	SP	7,670	7,670
Clear Water Transmission				
WTP to HLR	1650	SP	3,000	3,000
HLR to Pokunuwita J.	1200	DIP	6,680	6,680
Pokunuwita J. to Piliyandala	1200	DIP	17,000	17,000
Piliyandala to Dehiwala	1000	DIP	9,580	
Piliyandala to Dehiwala	1100	DIP		9,580
Piliyandala to Moratuwa	800	DIP	4,800	
Pokunuwita J. to Panadura	500	DIP	15,250	
Moratuwa to Keselwatte	500	DIP	3,500	
Moratuwa Res. to New Moratuwa Tower	500	DIP	50	
Connection to New Piliyandala Tower	400	DIP	30	
Kalatuwawa Main to New Homagama G.R.	400	DIP	200	
Kalatuwawa Main to Kesbewa Sub Area	300	DIP	1,000	
Connection to Exis. Horana G.R.	200	DIP	180	

7.6.5 Distribution Facilities

(1) General

Water distribution planning was prepared for the eight divisions as: Dehiwala, Moratuwa, Panadura, Kesbewa, Keselwatte, Homagama, Bandaragama, and Horana. Treated water distribution facilities for the proposed project will consist of reservoirs, water towers, and distribution pipelines.

In the planning of the distribution system, the following points were taken into consideration.

- o Existing facilities will be used as much as possible.
- o To reduce the pumping costs, an entire service area is divided into zones as necessary.
- o To accommodate the future demand, minimum reinforcement of pipeline will be made by addition of new pipes.
- o Sources of water are determined as defined in the transmission plan.
- o Pipes of 100 mm or larger in diameter are defined as distribution main. Only distribution mains are considered in the distribution network analysis.
- o Pipes smaller than 100 mm in diameter are defined as branch pipe. Length of branch pipe is estimated from the area of each service area and considered in cost estimates.

Layout and diagram of the distribution network are presented in Drawings (Volume V).

Concept of the proposed distribution system for each area is described in the following sub-sections.

(2) Dehiwala Service Area

Dehiwala service area covers entire Dehiwala M.C. At present, Dehiwala area is supplied from two water towers (G9 and G10). G9 tower receives water from the Ambatale-Dehiwala transmission main while G10 tower is supplied from ground reservoir G4 by pumping. The ground reservoir has two compartment called as G4 and C.M.C. G4 reservoir is receiving water from Ambatale via Ambatale-Dehiwala transmission main. Ground reservoir C.M.C. is supplied water from Kalatuwawa-Dehiwala main and serving for the C.M.C. service area.

The area supplied from the water tower G9 is located at north. This area is called Dehiwala north zone. Rest of the area is presently supplied from the water tower G10 except for the some areas along the transmission main from Dehiwala to Moratuwa which are taking water for distribution directly from this main..

Most of the land in the Dehiwala service area lies below 15 m MSL. It will be possible to serve most of the area directly from the ground reservoir and provide satisfactory pressures. If this modification is made, it will help reduce leakage by reducing supply pressure as well as reducing pumping cost.

This recommendation follows the study named as "Operation and System Control of the Greater Colombo Water Supply System" prepared by Engineering-Science and Resources Development Consultants in 1988.

With this zoning, Dehiwala area is divided into three zones as follows:

North Zone :	Area north of Dehiwala canal. To be supplied from water tower G9.
Low Zone :	Area to be supplied from Dehiwala Ground Reservoir G4. A new distribution main is proposed to be laid as shown in distribution pipeline layout.
High Zone :	Area to be supplied from water tower G10. This zone stretches south along Galle Road.

The configuration of the Dehiwala distribution system is schematically shown in Figure 7.22.

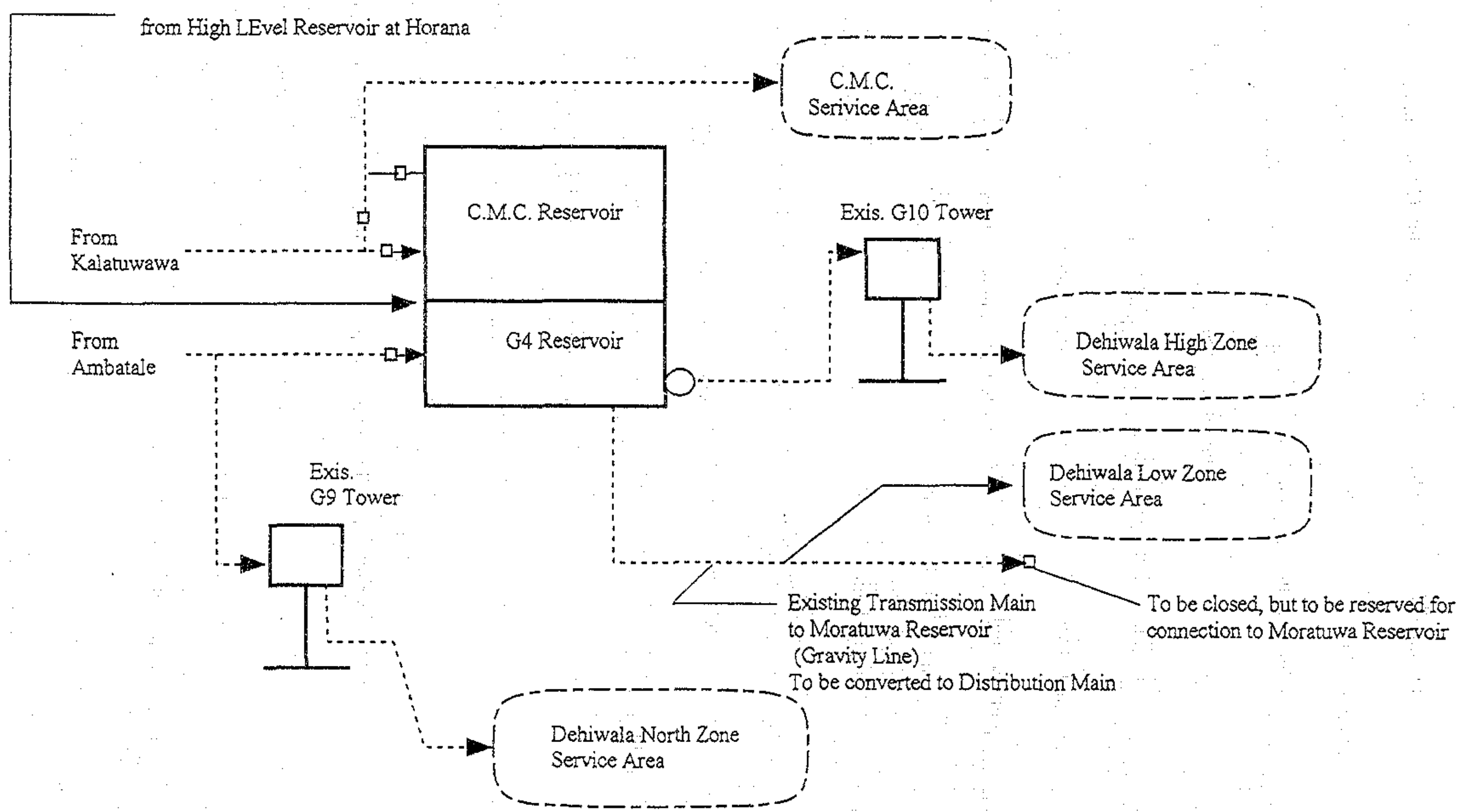

Figure 7.22 Configuration of Dehiwala Distribution System

(3) Moratuwa Service Area

Moratuwa service area covers entire Moratuwa U.C. At present, Moratuwa area is supplied from water tower G6 which is supplied water from Moratuwa Reservoir G5. Moratuwa reservoir G5 is presently receiving water from Dehiwala reservoir G4 through a 800 mm cast iron transmission main along Galle Road.

As well as Dehiwala, most of the land in the Moratuwa service area lies below 15 m MSL. It will then be possible to divide the area into two zones according to elevation. Elevation of the existing ground reservoir, however is not sufficient to supply to the low zone by gravity. Construction of a new water tower is therefore proposed. A new water tower may be located in the promise of the existing ground reservoir and at the elevation which will make TWL +35.00 m. This proposed new tower will be used to serve for the low zone by gravity so that the pumping amount to G6 tower will be substantially reduced and operation cost for this pumping will also be reduced.

This recommendation also follows the study of "Operation and System Control of the Greater Colombo Water Supply System" as well as for Dehiwala.

With this zoning, Moratuwa area is divided into two zones as follows:

Low Zone :	Area to be supplied from the new water tower.
High Zone :	Area to be supplied from water tower G6. This zone also covers the southern area stretching to Panadura.

The configuration of the Moratuwa distribution system is schematically shown in Figure 7.23.

(4) Panadura Service Area

Panadura service area covers entire Panadura U.C. At present, Panadura area is divided into two pressure zones by elevation. A northern part of Panadura is supplied from water tower G12 which is supplied water from Panadura reservoir G7 by pumping. Reservoir G7 also directly supply the rest of Panadura U.C. by gravity.

Panadura reservoir is receiving water from Moratuwa tower G6 through a 450 mm transmission main. This area is also being supplied water from the Kalutara Water Supply Scheme through a 250 mm PVC transmission main. This additional supply was provided in 1991 to help a water shortage in the Panadura area.

The existing zoning is recommended to be used in future distribution. The configuration of the Panadura distribution system is schematically shown in Figure 7.24.

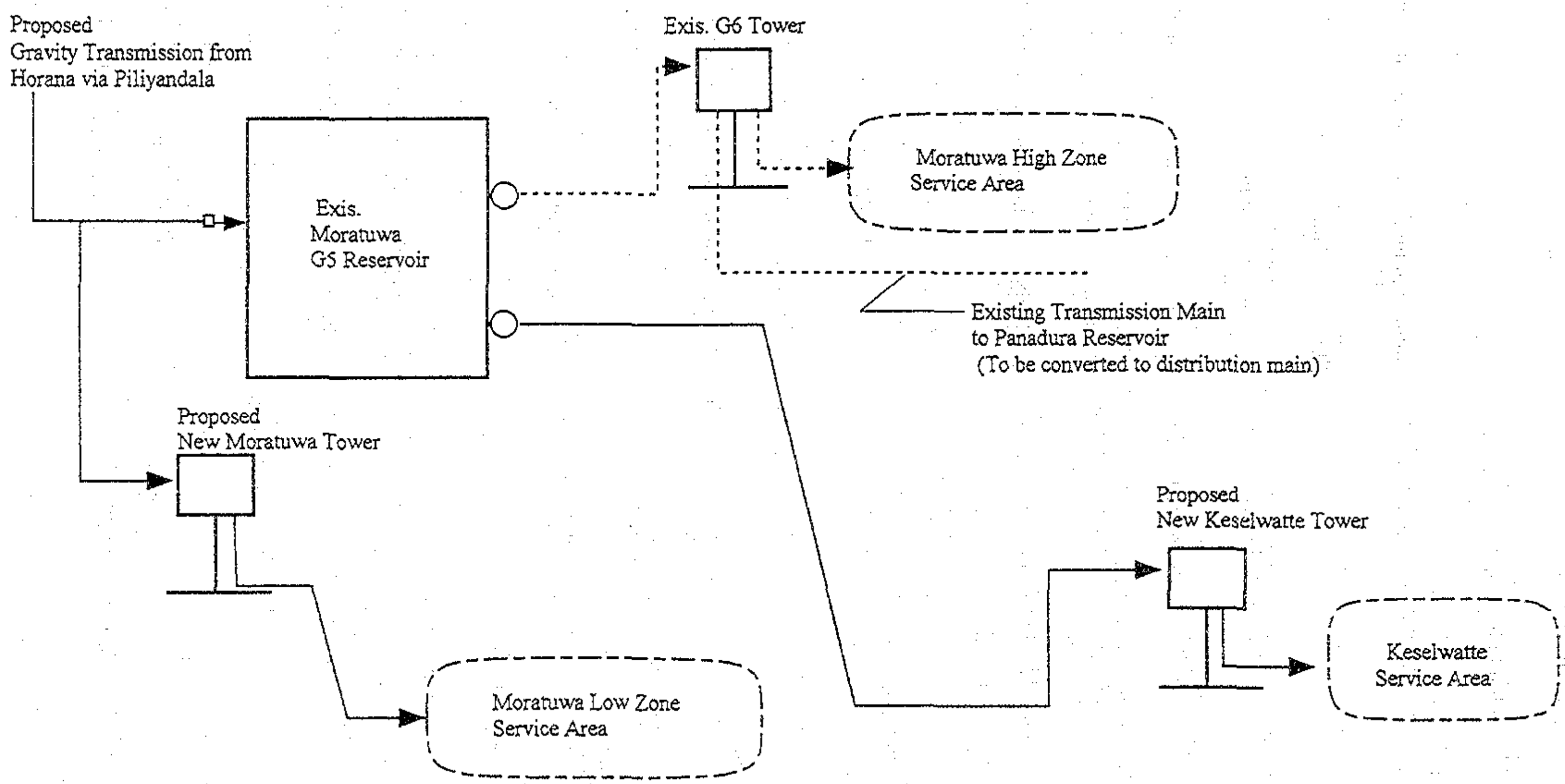

Figure 7.23 Configuration of Moratuwa Distribution System
(including Keselwatte)

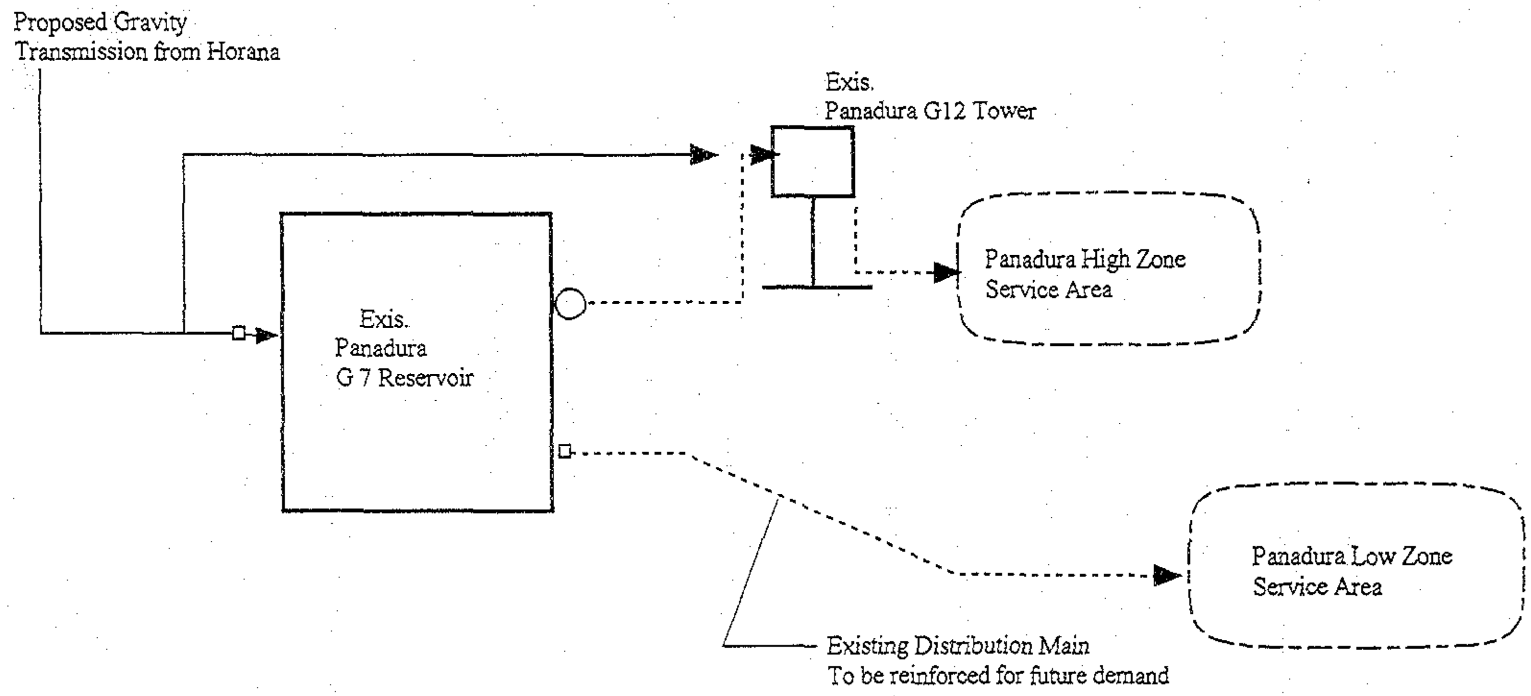

Figure 7.24 Configuration of Panadura Distribution System

(5) Kesbewa Service Area

Kesbewa service area covers almost half of Kesbewa P.S. This service area is the area scheduled to be implemented under the Towns South Project under OECF finance.

The area is divided into two sub-areas considering the topography as listed below:

Main Area : Area along Piliyandala road and its surrounding. To be supplied from the proposed new transmission main from the new treatment plant at Horana. It is proposed to construct a water tower which will directly receive water from the new transmission main.

Sub Area : Area located north-eastern. This area is located close to High Level Road so that it is proposed to be supplied water from the Kalatuwawa-Dehiwala transmission main which is running under High Level Road.

The configuration of the Kesbewa distribution system is schematically shown in Figure 7.25.

(6) Keselwatte Service Area

Keselwatte service area covers Keselwatte area in Panadura P.S. This area will also be implemented under the OECF financed Towns South Project. This area has a narrow shape along old Galle Road.

It is proposed to build a water tower for gravity supply therefrom. Water will be transmitted to the water tower from Moratuwa reservoir by new pumping station.

The configuration of the Keselwatte distribution system is schematically shown in Figure 7.23 together with the Moratuwa system.

(7) Homagama Service Area

This area will also be implemented under the OECF financed Towns South Project as well as Keselwatte. Homagama service area covers a central area of Homagama P.S. Treated water is planned to be supplied to a new ground reservoir from the existing Kalatuwawa - Dehiwala main. A high level reservoir is proposed at the elevation of +40m MSL to receive water by pumping from the ground reservoir and distribute it to the service area.

The configuration of the Homagama distribution system is schematically shown in Figure 7.26.

(8) Bandaragama Service Area

There is an existing distribution system at the town center of Bandaragama consisting of deep well, water tower and distribution pipelines. In the future development plan, the water tower is proposed to be supplied water from the proposed transmission main laid from Horana along the national road A8. Some expansion to meet the 2010 demand is proposed to be implemented.

The configuration of the Bandaragama distribution system is schematically shown in Figure 7.27.

(9) Horana Service Area

The existing Horana water supply system covers the entire Horana U.C. Water taken from Kalu Ganga is conveyed to the high level ground reservoir located at the hill side north of Horana town center. Water is then distributed to the service area by gravity. The existing distribution pipelines will be used for future with necessary reinforcement by constructing additional pipes. There will be no expansion of the service area to outside of Horana U.C. area.

The configuration of the Horana distribution system is schematically shown in Figure 7.28.

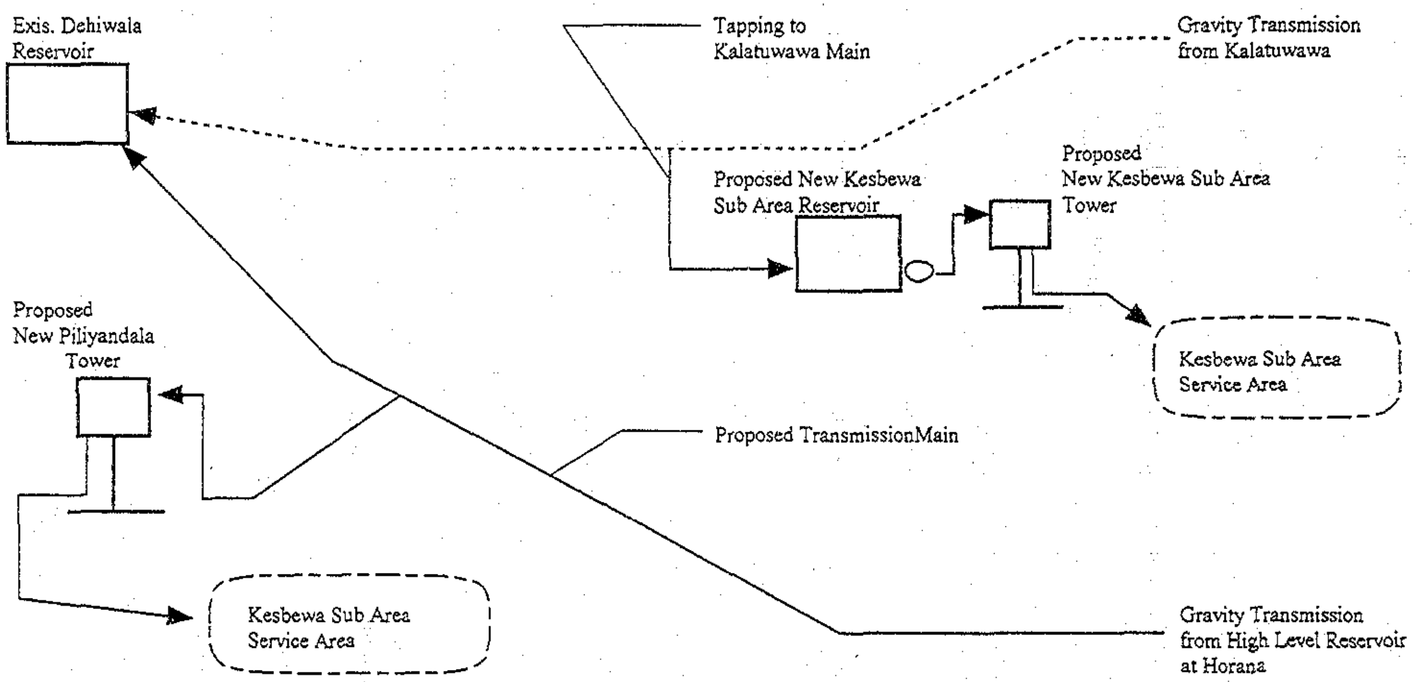

Figure 7.25 Configuration of Kesbewa Distribution System

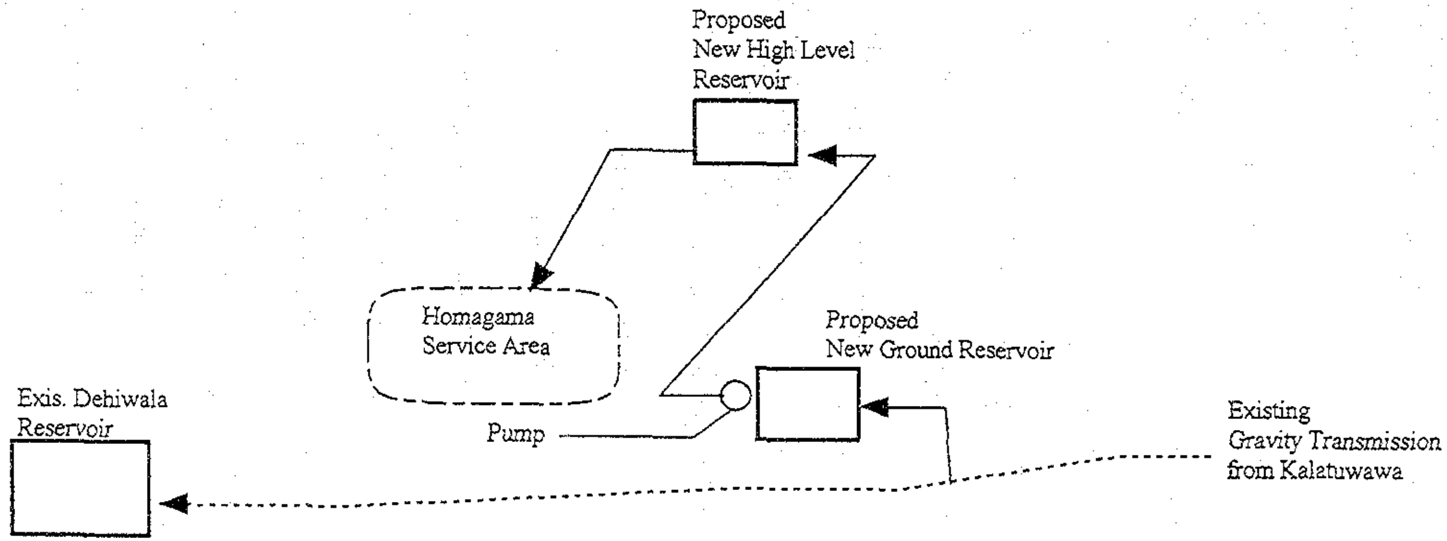

Figure 7.26 Configuration of Homagama Distribution System

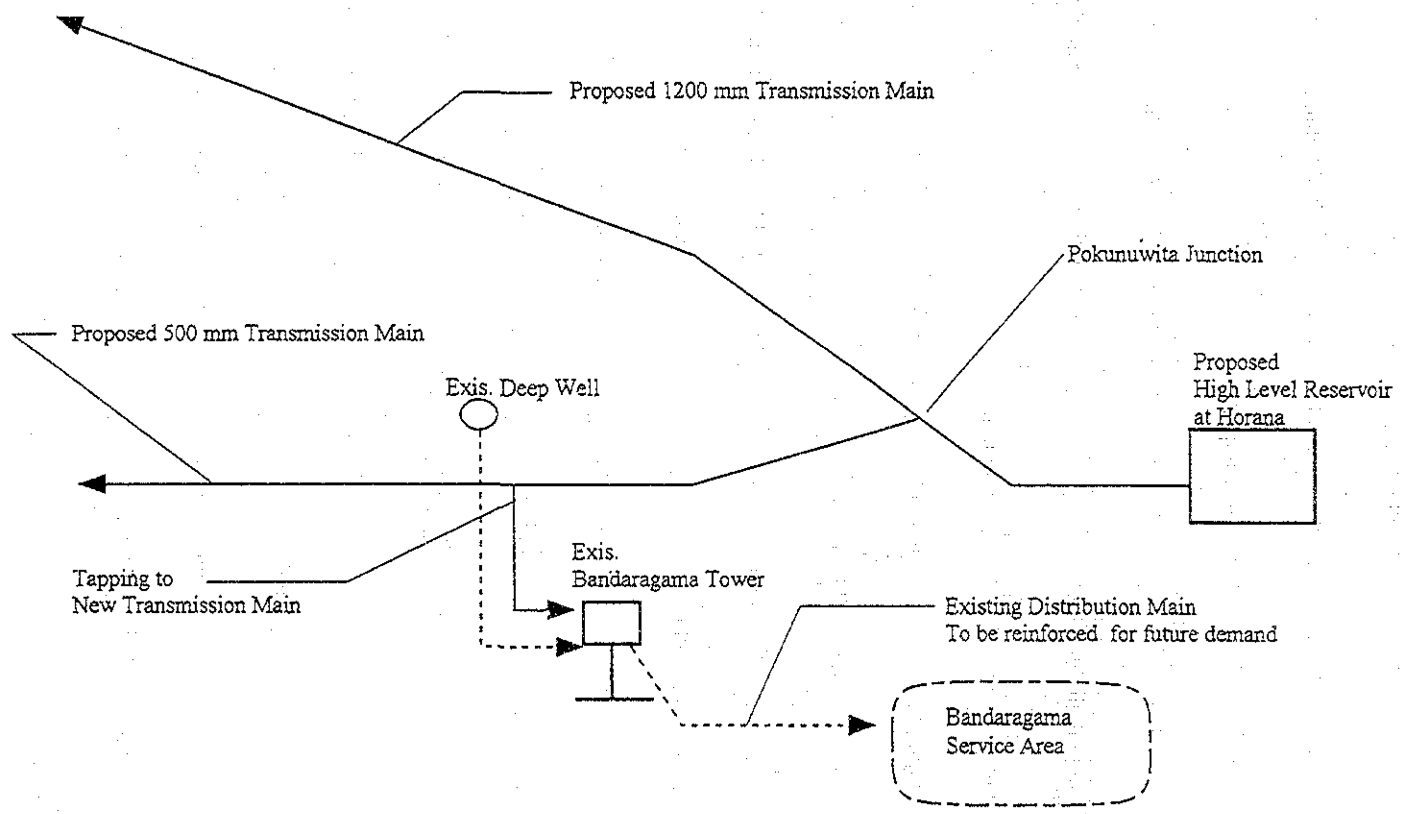

Figure 7.27 Configuration of Bandaragama Distribution System

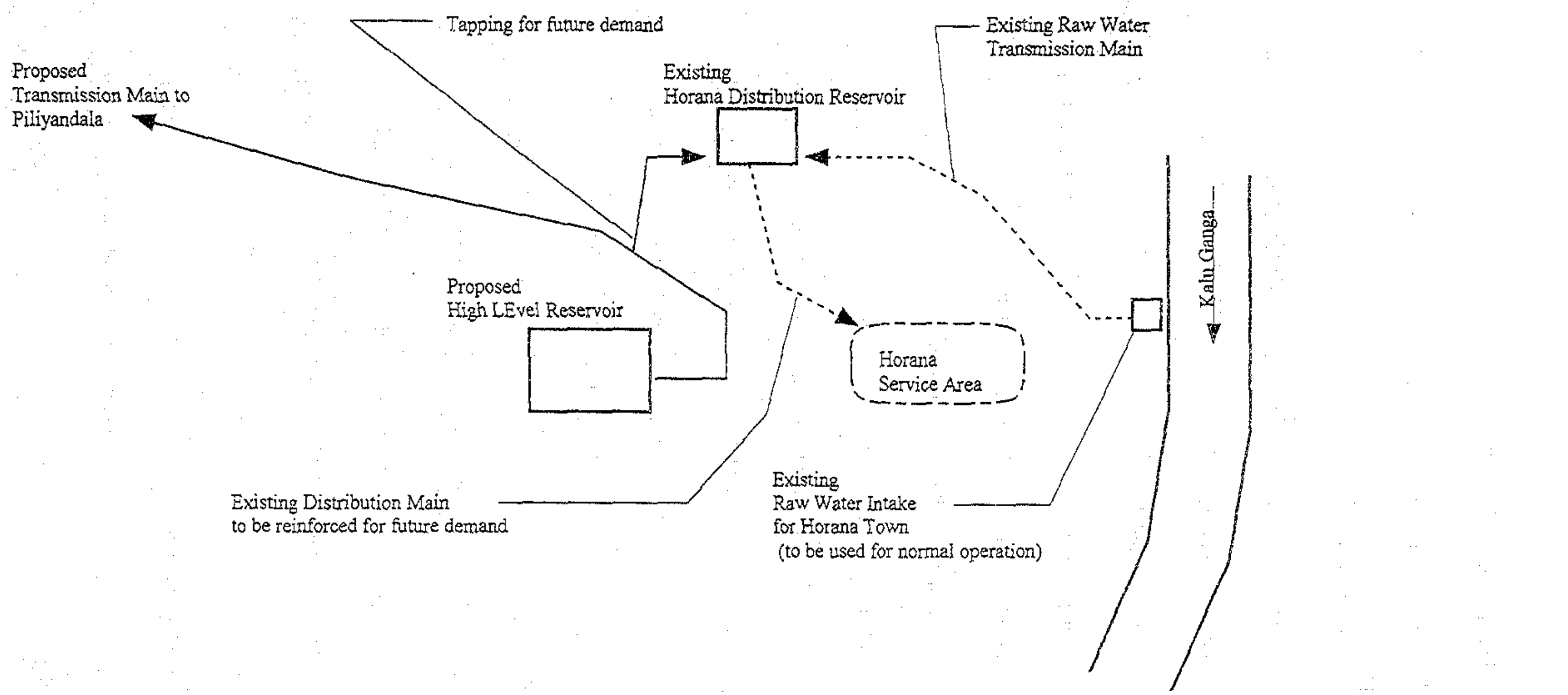

Figure 7.28 Configuration of Horana Distribution System

7.7 Phased Implementation Plan

(1) General

The design target years for the Kalu Ganga Water Supply System are set at 2010 for the Feasibility Study and at 2020 for the long term development plan. Based on these design target years, the implementation plan is divided into two Phases: (I) Phase 1 from present to 2010, and (ii) Phase 2 from 2011 to 2020.

From the view point of efficient and economical use of fund for this large scale project, it is obvious to implement the construction of the proposed facilities in multiple stages with the water demand satisfied in each planning year. Concept of stage construction is prepared for the facilities as described in the following sub-sections.

Facilities constructed in each stage are summarized in Table 7.18 below.

Table 7.18 Stage Construction of Treatment Plant

Facilities	Phase 1 (2010)		Phase 2 (2020)	
	Stage 1	Stage 2	Stage 1	Stage 2
Intake Station				
* intake mouth	1/1	-	-	-
* grit chamber	1/2	-	1/2	-
* intake pump	1/4	1/4	1/4	1/4
* pump station	1/2	-	1/2	-
Raw Water Transmission	1 line	-	1 line	-
Treatment Plant				
* major treatment facilities	1/4	1/4	1/4	1/4
* major pump equipment	1/4	1/4	1/4	1/4
* power receiving system	1/1	-	-	-
* major building	1/1	-	-	-
Clear water transmission	to Panadura	to Dehiwala	duplicate to Dehiwala	-
Distribution facilities	Moratuwa	Other areas	reinforcement in each area	

Note: The figures show the rate of facilities to be constructed in the particular stage to the full facilities in 2020.

(2) Intake Facility

Intake facilities will be constructed at the river bank and will need a temporary work in the river to dry the construction area. Such temporary work will be costly and will likely affect the environment. Construction work in the river will also affect the quality of raw water if an additional part of the intake mouth will be constructed in future. It is therefore recommended to complete in one stage the works which will need the temporary work in the river. Other part of the structure which will not require the work in the river may be constructed in later stage when the demand arises.

Entire part of the intake mouth for the 2020 demand will be constructed in the first stage. Intake channel will be constructed in half in length and temporarily closed. Grit chamber and pumping station will be constructed for the 2010 demand only. Future expansion of grit chamber and pumping station for the 2020 demand will be made in the adjacent site as shown in the design drawing.

Installation of pumps for the 2010 demand will however be divided into two stages. Out of eight intake pumps, four units will be installed in the first stage and other four units will be added in the second stage. Electrical power supply system will be constructed for 2010 demand in the first stage.

(3) Raw Water Transmission Pipeline

Raw water transmission will be carried out by a single pipeline for the 2010 demand and by an additional pipeline for the 2020 demand.

The first pipeline of diameter 1500 mm for the 2010 demand will be constructed in the first stage.

(4) Water Treatment Plant

The treatment plant for the 2010 demand will have a capacity of 40 mgd (182,000 m^3/d). Considering the time span for stage construction, it is proposed to construct 20 mgd of treatment capacity in the first stage and another 20 mgd in the second stage. Buildings which are difficult to divide in two stages will be constructed in the first stage.

Phasing for Phase 2 for the 2020 water demand will likely be implemented in the same manner as the Phase 1 implementation.

(5) Clear Water Transmission Facility

Clear water transmission pipeline is a major part of facility which will need large investment. It is financially effective to divide the pipeline construction into two stages as well as the treatment plant. Three alternatives for construction of the transmission main are compared as follows:

Alternative 1

- o Extend the transmission main to Dehiwala from the High Level Reservoir.
- o Branch line from Piliyandala to Moratuwa is assumed to be constructed in the Towns South Project (see Section 7.9 (2)).
- o Extension of the 500 mm main for Panadura will be made in stage 2.

Alternative 2

- o Extend the transmission main to Piliyandala from the High Level Reservoir.

- o Branch line from Piliyandala to Moratuwa is assumed to be constructed in the Towns South Project (see Section 7.9 (2)).
- o Extension of the 500 mm main for Panadura will be made in stage 2.

Alternative 3

- o Extend the 500 mm main from Pokunuwita Junction to Panadura
- o Branch line from Piliyandala to Moratuwa is assumed to be constructed in the Towns South Project (see Section 7.9 (2)).
- o Extension of the 1200 mm main for Piliyandala and the 1000 mm main for Dehiwala will be made in stage 2.

Comparison of the alternatives are summarized in Table 7.19.

Table 7.19 Comparison of Alternatives for Stage Construction of Transmission Main

Item	Alternative 1	Alternative 2	Alternative 3
Pipeline	dia.1200 - 23,680 m dia.1000 - 9,580 m Total 33,260 m	dia.1200 - 23,680 m	dia.1200 - 6,680 m dia.500 - 15,250 m Total 21,930 m
Water Demand Supplied from Kalu for 2005 (Daily Average)	Dehiwala 2,980 m^3/d Moratuwa 34,583 m^3/d Panadura 8,447 m^3/d Keselwatte 3,959 m^3/d Piliyandala 5,507 m^3/d Horana 2,863 m^3/d Total 58,339 m^3/d	Moratuwa 34,583 m^3/d Panadura 8,447 m^3/d Keselwatte 3,959 m^3/d Piliyandala 5,507 m^3/d Horana 2,863 m^3/d Total 55,359 m^3/d	Panadura 8,447 m^3/d Bandaragama 714 m^3/d Moratuwa 10,400 m^3/d Horana 2,863 m^3/d Total 22,424 m^3/d
(Daily Maximum = Daily Average x 1.15)	67,090 m^3/d (15 mgd)	63,660 m^3/d (14 mgd)	25,800 m^3/d (6 mgd)
Hydraulic Transmission Capacity (not more than production capacity of WTP)	91,000 m^3/d (20 mgd)	91,000 m^3/d (20 mgd)	25,800 m^3/d (6 mgd)
Construction Cost	Rs. 2,734 mil.	Rs. 2,131 mil.	Rs. 850 mil.

Alternatives 1 and 2 have little difference in supply amount for the water demand in 2005 (approximately 5 percent). The difference is derived from the supply amount for Dehiwala. This is because the water demand of Dehiwala in 2005 will be mostly supplied from Ambatale and Kalatuwawa so that the contribution from the Kalu system will be quite small.

Cost difference between Alternatives 1 and 2 is about Rs.600 million (approximately 28 percent). This is considered quite large compared to the difference in the supply amount. Shifting a large investment to stage 2 will be an benefit in financing arrangement .

Cost difference between Alternatives 2 and 3 is Rs.1,281 million which is substantial. This aspect may have to be given consideration from the view point of investment schedule and financial capacity of the NWSDB.

Alternative 3 will show a shortage in transmission capacity to meet the daily maximum demand in the years before the stage 2 implementation has been completed. The demand in daily average basis will however be satisfied. This alternative will therefore not lead to immediate inconvenience in water supply as long as the daily average demand is satisfied although the full demand may not be fulfilled in some time in a year when the demand increases by seasonal fluctuation. This alternative will have an advantage in smaller investment in the first stage which may facilitate the fund arrangement for this large scale of project.

Taking into account the smaller investment in stage 1, Alternative 3 is recommended. Facilities to be constructed in each stage are as shown in Table 7.20.

Transmission diagram in 2005 is presented in Figure 7.29 as a transition period between Stage 1 and Stage 2.

Table 7.20 Stage Construction of Clear Water Transmission Facility

Facility	Stage 1 WTP capacity 20 mgd	Stage 2 WTP capacity 40 mgd
Pipeline		
WTP to H.L.R.	dia.1650 mm, L= 3,000 m	-
H.L.R. to Pokunuwita Junction	dia.1200 mm, L= 5,350 m	-
Pokunuwita Junction to Piliyandala	-	dia.1200 mm, L= 17,000 m
Piliyandala to Moratuwa	dia.800 mm, L= 4,800 m	-
Piliyandala to Dehiwala		dia.1000 mm, L= 9,580 m
Moratuwa to Keselwatte	dia.500 mm, L= 3,500 m	-
Pokunuwita J. to Panadura	dia.500 mm, L= 15,250 m	-
Connection to Horana	-	dia.200 mm, L= 2,200 m
Connection to Homagama	dia.400 mm, L= 200 m	
Connection to Kesbewa Sub Area	dia.300 mm, L= 1,000 m	
Storage Facility		
High Level Reservoir	vol. = 30,000 m^3	

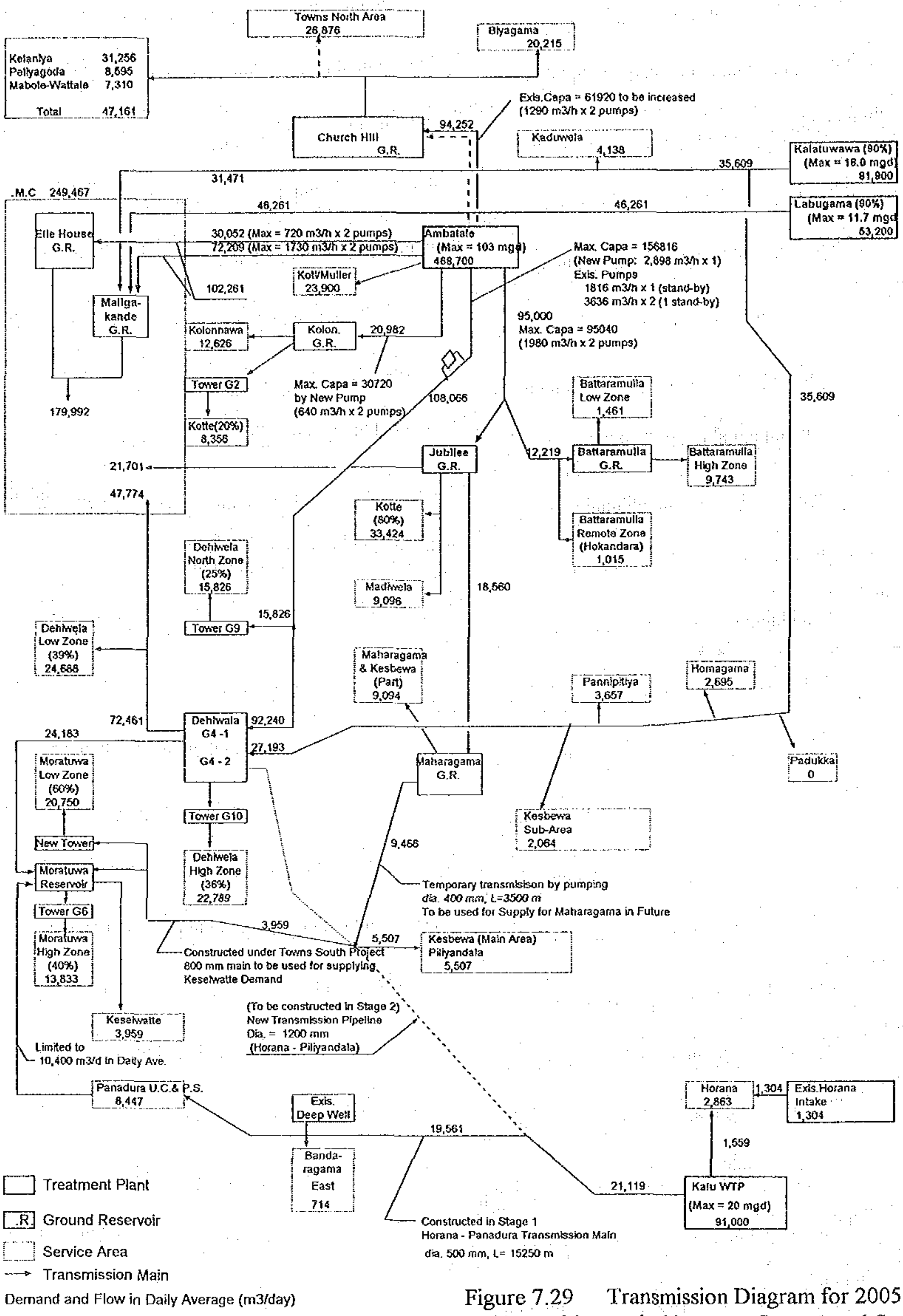

Figure 7.29 Transmission Diagram for 2005 Demand
(Transition period between Stage 1 and Stage 2)

(6) Distribution Facility

Expansion of distribution system will be made in stage including the Towns South Project which is presently ongoing under the OECF finance and will likely be completed prior to the commencement of the Kalu Ganga Project.

These service areas under the OECF project are all new area as follows:

- Kesbewa Area
- Homagama Area
- Keselwatte Area

Areas other than the scope of the OECF project already have the existing water supply facilities as described in the other part of this report. The scope proposed in the Kalu Ganga Water Supply Project is considered as reinforcement or modification of the existing distribution system to meet the future demand. Such area will be:

- Dehiwala Area
- Moratuwa Area
- Panadura Area
- Bandaragama Area
- Horana Area

Of these areas, Moratuwa area is given high priority for reinforcement of the distribution system since there are is fewer pipes in the area compared with the other area and the demand for water supply is high. Moratuwa area is therefore recommended to be included in the first stage implementation while the other areas will be implemented in the second stage.

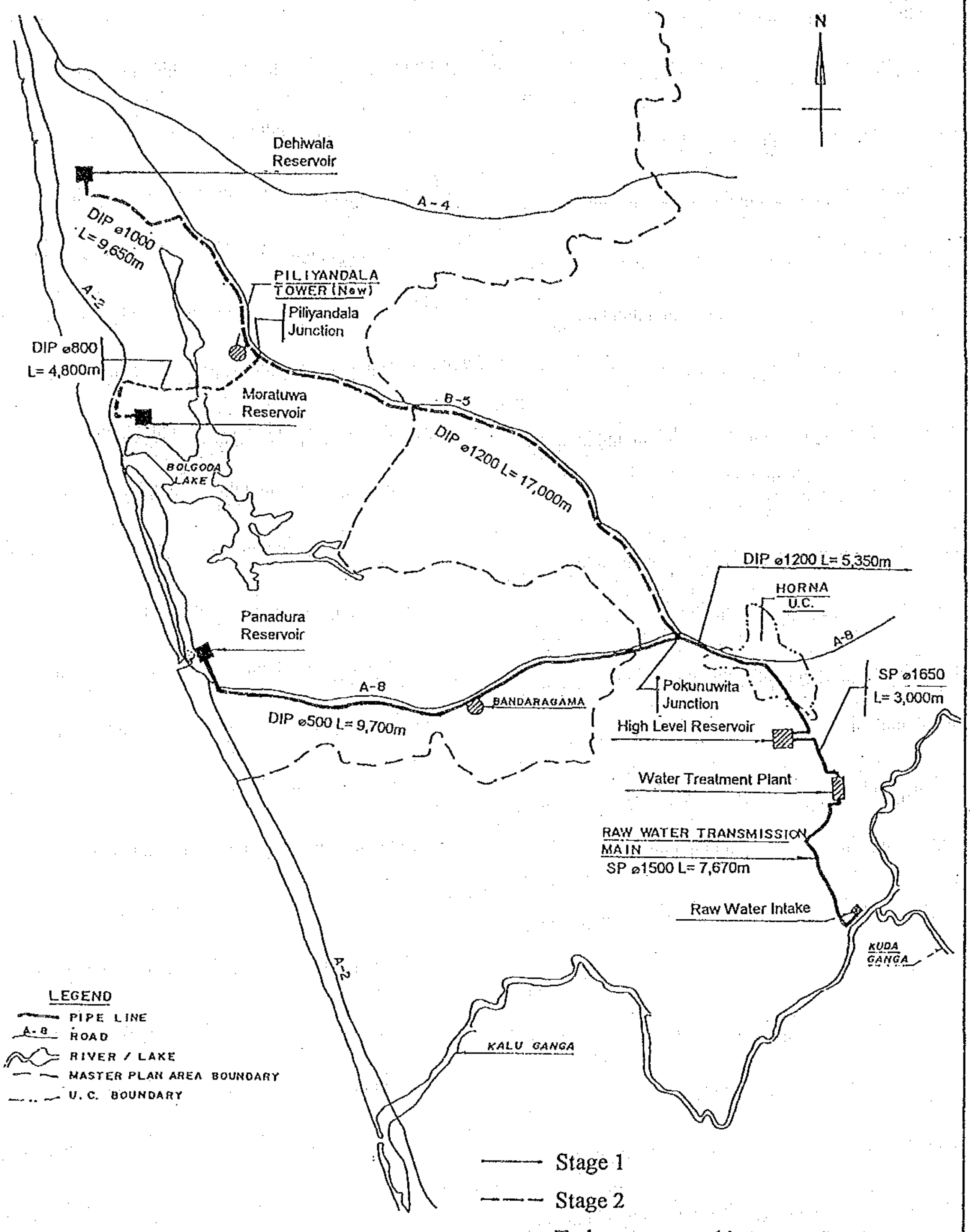

Figure 7.30 Layout of the Kalu Ganga Water Supply System

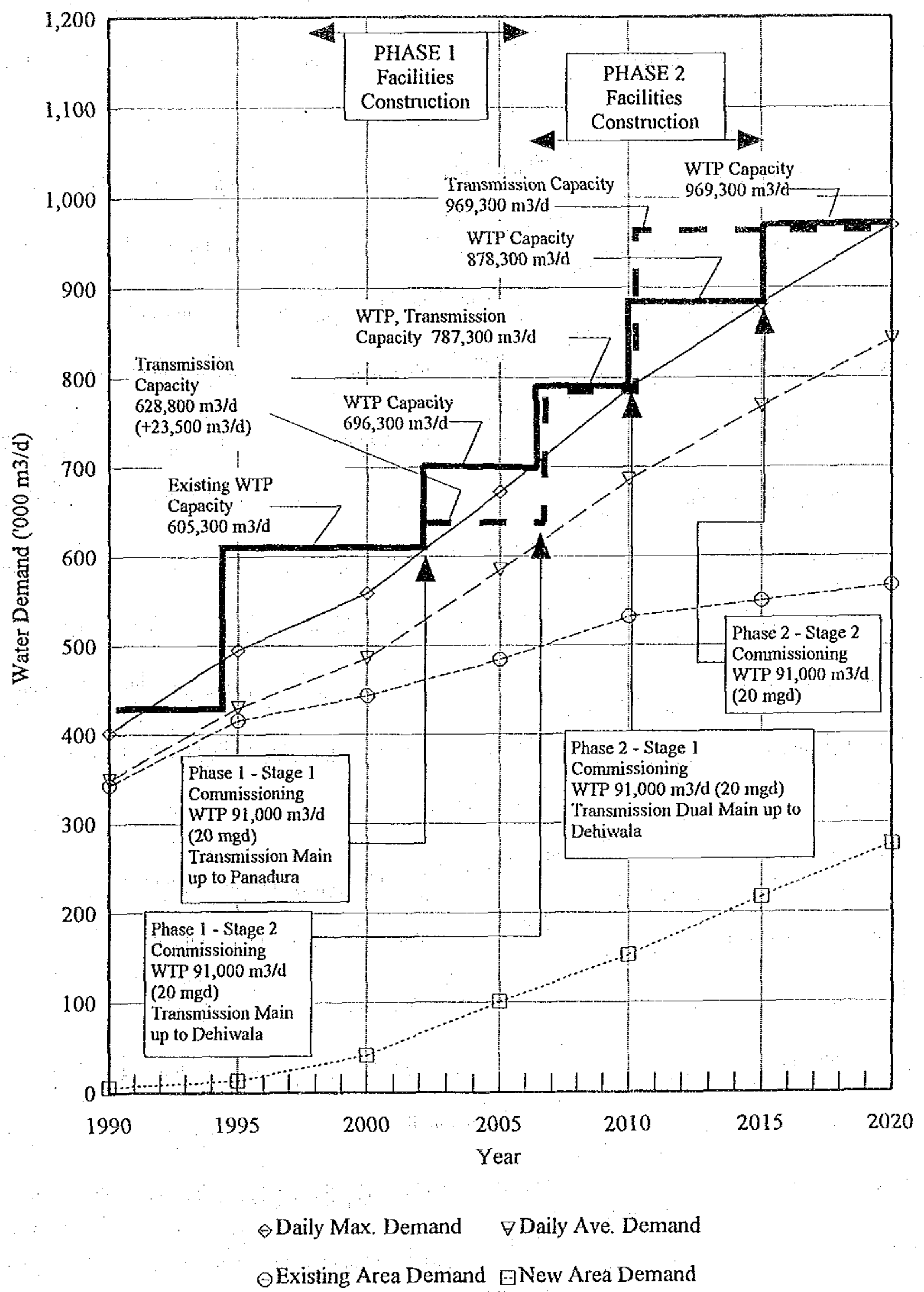

Figure 7.31 Water Demand Projection and Phased Implementation

7.8 Relationship with Other Projects

The Kalu Ganga project is proposed to augment the water production and water supply capacity of the Greater Colombo water supply system. It will provide an additional water source so that the Greater Colombo water supply system will be able to cope with the increasing water demand and proceed with the expansion of the service area.

Along with the need for the Kalu Ganga Water Supply Project, several projects are ongoing in Greater Colombo for the expansion of water supply. Relationship between the Kalu Ganga project and the other projects, particularly the service area expansion projects, is discussed in this section.

(1) Towns East of Colombo Water Supply Project - OECF financed

The service area for this project consists of three areas such as : Battaramulla, Pannipitiya and Kaduwela. Battaramulla service area will be supplied water from Ambatale treatment plant via the new 1100 mm Ambatale-Jubilee transmission main. Pannipitiya and Kaduwela will receive water from Kalatuwawa - Dehiwala and Kalatuwawa - Maligakande transmission mains, respectively. Therefore, this project will have no direct relation with the Kalu Ganga.

(2) Greater Colombo Water Supply System - Coverage of Southern Urban Areas (Towns South Project) - OECF financed

Under this project, expansion of three service areas will be implemented consisting of Kesbewa, Keselwatte and Homagama. The detailed design for the project is scheduled to commence in 1994. Construction of the distribution facilities is scheduled to be commenced in 1996 and be completed in 1997.

Out of the three service areas, Homagama service area is planned to receive water from the Kalatuwawa - Dehiwala transmission main so that there will be no direct relation with the Kalu Ganga project. Keselwatte is planned to be supplied water from Moratuwa either by pumping from the ground reservoir G5 or by gravity from the water tower G6 depending on the detailed analysis in the detailed design.

Kesbewa is divided in two zones as described in Section 7.6.5. Kesbewa sub-area will be supplied water from the Kalatuwawa-Dehiwala transmission main.

Kesbewa main area was planned, in the Master Plan Update, to be supplied from new Jubilee reservoir via a 700 mm new transmission main since there was no other source considered to obtain water at the time of the preparation of the feasibility study for the Tows South Project. This area is however proposed in the Kalu Ganga Project to receive water from the new Kalu Ganga system via the new transmission main connecting Horana and Dehiwala. This scheme is contradicted from the original transmission plan for Kesbewa which proposed the 700 mm transmission main to be constructed. Water transmission for Kesbewa from the Kalu system is however more economical and technically reasonable if the Kalu Ganga Project is to be implemented.

For the Towns South Project which will be implemented prior to the Kalu Ganga Project, water transmission to its service areas will need to be made from the existing water sources using the existing transmission facilities. Consideration on the water transmission is studied as follows:

1) Constraints in the existing system

Transmission main from Dehiwala to Moratuwa

The existing transmission main from Dehiwala to Moratuwa has a diameter of 800 mm with a length of about 9,600 m. This pipeline was laid for clear water transmission from Dehiwala reservoir to Moratuwa reservoir by gravity. It has however been tapped for distribution for the area along the pipeline route. To maintain the pressure at the tapping points, the inlet valve at Moratuwa reservoir is throttled. Nominal transmission capacity of this pipeline is calculated at about 40,000 m^3/d in case no tapping is made. The net transmission capacity to Moratuwa is however very limited by the tapping for distribution so that the pipeline will not be able to increase the transmission amount for future. The existing transmission amount is estimated at about 25,000 m^3/d from the pumping record at Moratuwa reservoir. This figure is considered as a net maximum transmission capacity.

The limitation in the transmission capacity will also affect the transmission for Keselwatte.

Transmission main from Kalatuwawa to Dehiwala

This pipeline has a total length of about 41,200 m with a diameter of 840 mm. Greater Colombo Water Supply Division of the NWSDB regards this main as a major supply source for the C.M.C. area in its southern part. The use of this main for supplying new service area should be limited as much as possible. Only Homagama area may be allowed to tap on this pipeline since Homagama has no alternative source other than this main.

2) Possible Source of Supply to Kesbewa and Keselwatte

Keselwatte is planned to be supplied from Moratuwa reservoir by pumping. Transmission amount from Dehiwala to Moratuwa will need to increase for the additional supply to Keselwatte, if no other source is provided. Such increase may be difficult to achieve due to the existing constraints in these transmission mains as described above.

It should then be considered that the transmission to Kesbewa and Keselwatte will have to be made from the other sources. For not using these two transmission mains, Dehiwala, Jubilee and Maharagama reservoirs are considered as alternative sources for comparison. Transmission to these two service areas are jointly studied rather than independently due to its close location.

(3) Recommendation

The original scheme (Jubilee to Kesbewa) was proposed before the commencement of this feasibility study on the Kalu Ganga Project so that it does not suit the development planning of the Kalu Ganga Project.

As an alternative, water transmision from Maharagama reservoir to Kesbewa shows the minimum cost. Although this plan will need construction of a pipeline for temporary transmission from Maharagama to Piliyandala, this pipeline may be able to transfer to distribution main when the Kalu Ganga water will be supplied to Kesbewa. With this water transmission, the two existing transmission mains to Dehiwala reservoir (from Ambatale and from Kalatuwawa) will be used as it is presently used without increase in transmission amount for Kesbewa or Keselwatte.

This alternative transmission plan is therefore recommended to be implemented in the Towns South Project for the economical investment although it will have a booster pumping station which will become idle or removed in future when the Kalu Ganga water comes.

CHAPTER 8

SCOPE OF WORK FOR THE FEASIBILITY STUDY

8. SCOPE OF WORK FOR THE FEASIBILITY STUDY

8.1 Service Area

Areas to be covered in the Feasibility Study are as follows:

Dehiwala-Mt. Lavinia M.C.
Moratuwa U.C.
Panadura U.C.
Kesbewa P.S.
Panadura P.S. (including Keselwatte)
Homagama P.S.
Bandaragama P.S.
Horana U.C.

8.2 Population Served by the Project

Served population which will receive direct benefit from the Kalu Ganga Water Supply System is projected as follows:

Service area	Year		
	2005	2010	2020
C.M.C.	0	134,311	450,131
Dehiwala M.C.	11,426	160,779	243,750
Moratuwa U.C.	168,171	175,950	182,610
Panadura U.C.	38,600	38,600	39,300
Kesbewa P.S.	26,226	43,169	91,388
Panadura P.S. (including Keselwatte)	23,508	34,084	59,460
Bandaragama P.S.	2,908	5,546	19,426
Horana U.C.	12,298	13,552	16,290
Total (rounded)	283,000	606,000	1,102,000

Note: Populations of C.M.C. and Dehiwala are projected from the proportion of water demand to be supplied from the Kalu System.
Homagama is excluded from the served population projection since it will be supplied from Kalatuwawa.

8.3 Water Demand

The water demand for the Kalu Ganga Water Supply System is projected at:

67,000 m^3/d (14.7 mgd) for 2005
182,000 m^3/d (40 mgd) for 2010

8.4 Facilities to be Involved

The facilities to be examined in the Feasibility Study shall be as follows:

1) Intake Facilities

Intake station with a intake capacity of 191,100 m^3/d for the 2010 water demand

2) Raw Water Transmission Facilities

Pipelinedia. 1,500 mm x approx. 7,670 m

3) Treatment Facilities

Water treatment plant with a treatment capacity of 182,000 m^3/d for the 2010 water demand

4) Clear Water Transmission Facilities

	From	To
a)	Water Treatment Plant	High Level Reservoir
b)	High Level Reservoir	Pokunuwita Junction
c)	Pokunuwita Junction	Piliyandala
d)	Piliyandala	Dehiwala
e)	Piliyandala	Moratuwa
f)	Pokunuwita Junction	Panadura

5) Storage Facilities

High level reservoir

Tower for Moratuwa low zone

Tower for Kesbewa main area

Ground reservoir and tower for Kesbewa sub area with pumping station

Ground Reservoir and high level reservoir for Homagama with pumping station

Tower for Keselwatte

6) Distribution Facilities

CHAPTER 9

DESIGN OF FACILITIES

9. DESIGN OF FACILITIES

9.1 Intake Facilities

9.1.1 Considerations in Designing

The following are considered in designing of intake facilities.

1) The grit chambers, pumping station and power receiving equipment are composed of two modules. One module will be constructed for the 2010 intake amount and the other will be added for the 2020 intake amount.

2) The intake mouth and intake gates will be constructed once for the 2020 intake amount, since the construction works require high engineering technology to temporarily isolate the site from the river flow.

3) As the maximum flood levels are 9.32 m and 9.96 m for return periods of 50 and 100 years, respectively, around the intake station, the top elevation of the intake mouth and grit chambers and the floor level of the pumping station are 10.0 m to prevent the flood from entering.

4) The grit chambers are covered with soil as the measures for buoyancy.

5) The stop logs are provided in front of the intake mouth to adjust the intake water level at the time of salinity intrusion. The bottom elevation of the intake mouth is set at -0.900 m same as the upper level of the salinity wedge based on the result of salinity intrusion analysis for the 2020 intake amount.

6) The screens are provided with the intake mouth to prevent the floating from entering.

7) Four units of staff house for station operators are constructed in the premises.

9.1.2 Outline Of Major Facilities and Equipment

The outline of major facilities and equipment at the intake station for the 2010 intake amount is as shown in Table 9.1.

Table 9.1 Outline of Major Facilities and Equipment at the Intake Station

Name	Dimensions/Spec.	No. of Units	Remarks
Intake Mouth	3.0mW x 10.1mH x 14.6mL x 4 lanes	1	w/ stop logs and screens, v = 27 cm/s
Intake Box Culvert	box culvert 3.0mW x 2.4mH x 26.9mL x 2 lanes	1	v = 27 cm/s
Grit Chamber	parallel flow, rectangular 7.0mW x 7.0mH x 54.7mL	2	v = 5.3 cm/s
Pump Well	30.8mW x 13.4mH x 8.0mL	1	
Pumping Station	32.0mW x 9.2mD x 7.5mH	1	
Intake Pump		8	
Power Receiving Equip.			

W : width H : height D : depth L : length v : velocity

9.2 Raw Water Transmission Facilities

One pipeline will be constructed for convey the amount of 2010 demand.

(1) Design Criteria and Specification

Maximum flow : 1.05 times the production capacity of the treatment plant for 2010 demand (40 mgd)
5 % added for in-plant use at the treatment plant

Maximum velocity in pipe : 1.5 m/sec
to be limited considering water hammer caused by pump operation

Pipe material : Mild steel with welding joint

Inside lining : Mortar cement lining

Outside coating : Coal tar epoxy coating with asbestos felt wrap

(2) Pipeline Details

Number of pipeline : 1 line

Diameter : 1500 mm

Length : 7,670 m

Pipe material : Mild steel

Flow : Pumping from the raw water intake

Maximum velocity : 1.3 m/sec

9.3 Treatment Facilities

9.3.1 Considerations in Designing

(1) Design Principles

In carrying out the design of treatment facilities in this Feasibility Study, practical computation and drawings are made for the target year 2010, and conceptual design for the target year 2020.

The treatment process units have been selected and designed in view of economic construction and easy operation and maintenance aspects. Simple structures will bring flexibility to the variation of raw water quality and unpredictable constraints on operation and maintenance works. The construction materials have been considered to enable acquisition from the domestic markets as much as possible.

The design of unit process has been carried out with due consideration of functionally appropriate layout and environmental aspects.

(2) Design Conditions

The proposed treatment plant site which is currently being registered as the government land amounts to 13.2 hectare and has become necessary to expand hopefully up to 14.6 hectare to establish facilities for the target year 2020. The detail programme for the future land acquisition may be made in the detail design stage.

The land reclamation work is recommended to implement approximately 60 % of the entire proposed area of 14.6 hectare prior to the commencement of construction of the facilities for the target year of 2010.

On the basis of the analysis of flood water level conducting the return period of 50 years as set out at +10.18 m MSL, land reclamation up to the level of +11.00 m MSL shall be carried out.

Topographic and soil conditions of the proposed treatment site utilized for designing purpose is based on the results of the topographic survey and borings carried out by the Study Team. The survey carried out includes leveling and cross sectional survey for the entire area and borings at selected nine locations within the area.

Staff housing area is arranged within the treatment plant site, however the residents currently being allocated in the area are to be relocated to the suitable location under the responsibility of the NWSDB.

9.3.2 Topographic and Soil Conditions

(1) Topographic Conditions

The proposed site is situated at about 4 km southeast of Horana town and lies along the Kalawellawa road extending to Malugama-wards. The site is close, about 300 m, to a small stream which runs to the Kalu Ganga and the aerial distance from the treatment plant site to the Kalu Ganga is approximately 4 km. The site currently consists of residential area where 76 houses exist and uncultivated land. The land is possessed by the Government with an area of 13.2 ha.

The site consists of moderately gentle hill with a highest mound at around the center of the area declining to the peripheral boundary which lies between the national land and surrounding private paddy field.

The altitude of the area extends from around +15.3 to +5.0 m MSL at the site of paddy field, so that approximately 10 m difference of altitude should be taken into consideration for planning of the treatment plant.

The topographic condition surveyed is given in Data Report (Volume IV).

(2) Soil Conditions

Boring tests including nine bore hole drillings and soundings have been carried out within the site by the Study Team. In addition, physical and mechanical soil tests by the sampling obtained from nearby test pits. The relevant data regarding location of bore holes and geological conditions are given in Data Report (Volume IV).

Soil strata as a whole constitutes four layers. The surface layer consists of medium dense well graded sands and gravels. The second layer consists of medium stiff sandy clays and gravels and/or medium stiff clay and clayey silts with N-values extending 5 to 15. The third layer consists of very dense to extremely dense silty sands and/or hard clayey silts with N-values extending 30 to 50. The fourth layer consists of hard rocks called Garnet-ferrous Biotite Gneiss with N-value of more than 50. The rock layer appears +4.0 to +5.0 m MSL and this feature outstandingly appears on the east half of the site. To the west-wards of the site, the rock layer drops suddenly and is likely to appear at around -10.0 m MSL. On this rock layer, medium stiff to hard clayey silts lies with 20 m in thickness. Three small rock mounds appear above the ground surface on the east half of the site.

The groundwater levels inside the bore holes have been observed and their levels are at around +8.0 m MSL at bore holes Nos.1, 2, 4 and 7 and +6.0 m MSL at bore holes Nos.3, 5, 6 and 9. This is assumed to be caused by the difference of implementation date of boring on which were affected by the rain fall

9.3.3 Outline of Major Facilities and Equipment

(1) Layout Planning of Major Facilities

Layout planning of the treatment plant facilities is arranged within the proposed site of 14.6 ha as given in Figure 9.1. The proposed entire area shall be reclaimed up to the level of +11.0 m MSL in order to protect from an inundation during the flood of 50 year return period.

The major facilities for the target year of 2010 are planned to establish in two stages, namely Stage I and II, within two third of the entire area. Whereas, the proposed facilities for the target year of 2020 will be established in the remaining remote area in order to avoid nuisance which will be generated during construction period in the future.

The sludge drying beds are considered to lay out at the east end to enable easy discharging of supernatant water to the nearby stream situated approximately 300 m eastwards.

(2) Flow Diagram of the System

The proposed water treatment system is planned to establish on the basis of the maximum day demand 182,000 m^3/d for the target year of 2010 as given in the previous section. Figure 9.2 gives the flow diagram required to compute and determine the capacity of each process unit in the system.

Solids settled in the sedimentation basin will be separated in the sludge thickeners and subsequent supernatant water is re-circulated to the receiving well together with dirty washwater generated in the filter.

(3) Design Details of Major Facilities

1) Receiving Well

The receiving well is located at the west end of the area. Raw water conducted through the side gate of the treatment plant yard is led to this receiving well. The structure is a reinforced concrete rectangular type with dimensions of 3 m (W), 2 m (L) and 4 m (D). Because of the land reclamation on a flat level, the structure will be raised above the ground to adjust water level at +16.0 meter MSL as given in the hydraulic profile in the Drawings (Volume V).

2) Hydraulic Rapid Mixer

The hydraulic rapid mixer is located following the receiving well. The structure is reinforced concrete and parallel twin rectangular channel type. The structural dimensions are 2 x 1.0 m

(W), 5 m (L) and 1.1 m (D). In order to effectuate rapid mixing, hydraulic jump is facilitated. This structure is also successively raised above the ground level to adjust water levels as given in the hydraulic profile in Drawings (Volume V).

3) Flow Splitting Chamber

The chamber consists of four inner splitted chambers provided with stop logs to separate/stop water runs to the subsequent flocculators. The structure is a reinforced concrete rectangular type with dimensions of 4.45 m (W), 4.65 m (L) and 1.5 m (D). The structure is raised in like manner as shown in the hydraulic profile in Drawings (Volume V).

4) Hydraulic Flocculation Basin

The hydraulic flocculation basin consists of reinforced concrete vertical baffle walls (up-and-down type) installed in 1.5 m interval with gradually increasing channel width forward. Width, length and depth of the flocculation basin are 10.0 m, 13.8 m and 3.5 m, respectively. Mean velocity of the flocculation basin should be kept within a range of 15 to 30 cm/sec to grow flocs gradually. The retention time is planned with more than 25 minutes.

For easy maintenance, drain holes on every other wall and drain pipes of 150 mm in diameter are provided as given in Drawings (Volume V).

5) Horizontal Flow Sedimentation Basin

Due to comparatively good condition of raw water through years with 20 NTU turbidity concentration in average, horizontal flow sedimentation is selected as described in detail in the previous section. The structure of the basin is a reinforced concrete type with dimensions of 10 m (W), 68 m (L) and 4.5 m (D) as shown in Drawings (Volume V).

For the removal of sedimented sludge, traveling sludge scraper is prepared to remove sludge effectively and to save manpower. The scraper is generally operated in a speed of 12 m per hour in order not to stir up the sedimented sludge at the bottom, so that approximately one day will be necessary to accomplish desludging in each basin. This routine work will be necessary once a day.

In the sedimentation basin, two rectification walls at the both inlet and outlet side and collecting troughs with a length of 5 m at the outlet are provided. The collected water is transmitted to the filters by two channels.

6) Rapid Sand Filters

The filter is a constant rate, rising-level with self backwashing type having 20 filter units consisting of 78 m2 each. Filter basin is located immediately after the sedimentation basin connected by the channels. The dimensions of the structure consists of 50.5 m (L), 72.2 m (W) and 5.4 m (D), respectively as given in Drawings (Volume V).

The influent is conducted through the two parallel channels with water level of +13.5 m MSL at the inlet and discharged into the filter passing over the weir. The filter is planned to operate by a filtration rate of 120 m/d. The filter media comprises a single-media of 0.6 mm sand with 600 mm in depth.

When the inflow water level rises above the sand surface due to clogging, inlet gate will be closed. The water level, thereafter descends gradually until the same level as outlet weir. The water is discharged away through the wastewater channel by backwashing from the clear water channel, and at the same time surface washing will be started. Backwashing rate is set at 0.8 m/min and surface washing rate 0.3 m/min.

7) Clear Water Basin

Filtered water is conducted by two connection pipes of 1,000 mm in diameter to the clear water basin located downstream of the filter basin. The structure is reinforced concrete (beam type). The measurement of the structure is 46.5 m (W), 60.5 m (L) and 4.0 m (effective D) as given in Drawings (Volume V). The clear water basin is attached with pump house provided for clear water transmission to the storage reservoir apart from the treatment plant.

8) Sludge Thickening Basin

Volume of dried sludge has been calculated as 6.6 t/d on the basis of Japanese design criteria and British conventional formula presented by Water Research Center and P.C.I. as given in Supporting Report (Volume III). The structures of sludge thickening basin and sludge drying beds are determined on the basis of this calculation.

Sludge thickening basin is provided downstream of the clear water basin consisting of three units of 20 m (W), 30 m (L) and 3.0 m (D), respectively as given in Drawings (Volume V). The structure is a reinforced concrete type provided with outlet gallery.

It is generally seen that separation of supernatant water is a problematic routine work using such as stop logs. In this study, a float is prepared to effectuate the function of water and sludge separation. The thickened sludge is pumped to the sludge drying beds provided further downstream.

9) Sludge Drying Bed

Sludge drying beds are facilitated at the east end of the site. On the basis of the computation of sludge volume, eight beds are provided to dry up sludge. The structure of drying beds is a reinforced concrete type with dimensions of 20 m (W), 60 m (L) and 2 m (D), respectively as given in Drawings (Volume V).

10) Other Facilities

The plan of main building and chemical building are shown in Drawings (Volume V). The main building is arranged to provide adequate rooms for efficient works for the staff and for good operation and maintenance works. The structure is a reinforced concrete two stories building located at the entrance of the treatment plant yard. The area of the main building is provided with 1000 m^2

The chemical house and chlorine house with areas of 600 m^2 and 200 m^2, respectively to provide lime, alum and chlorine are provided. The structure of these houses are constituted by bricks and plates with supporting frames.

The capacity of units of in a chemical storage house is given in Supporting Report (Volume III).

11) System Flow Sheet

System flow sheet for the preparation of chemical, mechanical and electric devices and instruments are shown in Drawings (Volume V).

Major facilities and equipment of the treatment plant for Phase 1 (2010 demand) are summarized in Table 9.2.

Table 9.2 Major Facilities and Equipment of Treatment Plant (Phase 1)

Facility	Dimension/Spec.	No. of Unit	Remark
Receiving Well	3.0mW x 2.0mL x 4.0mD	1	
Hydraulic Rapid Mixer	1.0mW x 5.0mL x 1.1mD	2	
Flow Splitting Chamber	4.45W x 4.65mL x 1.5mD	1	
Hydraulic Flocculation Basin	Vertical flow type 10.0mW x 13.8mL x 3.5mD		mean velocity = 15 - 30 cm/s
Sedimentaion Basin	Horizontal Flow 10mW x 68mL x 4.5mD	8	with mechanical sludge scraper
Rapid Sand Filters	constant rate, variable head, self-backwashing type	20	filtering rate = 120 m/d = 5.0 m/h
Clear Water Basin	46.5mW x 60.5 mL x 4.0mD	1	

Table 9.2 Major Facilities and Equipment of Treatment Plant (Phase 1) (cont'd)

Facility	Dimension/Spec.	No. of Unit	Remark
Sludge Thickning Basin	20mW x 30mL x 3.0mD	3	
Sludge Drying Bed	20mW x 60mL x 2.0mD	8	
Main Building	1000 m^2	1	
Chemical House	600 m^2	1	
Chlorine House	200 m^2	1	
Transmission Pump	Horizontal centrifugal pump 21.1 m^3/min, H=104m	8	Motor 560 kw, 6kV

9.4 Clear Water Transmission

One pipeline will be constructed for convey the amount of 2010 demand.

(1) Design Criteria and Specification

Maximum flow : 1.15 times the daily average demand
15 % added for seasonal fluctuation

Maximum velocity in pipe : 2.0 m/sec for gravity flow pipe
1.0 m/sec for pumping main to High Level Reservoir (Velocity in this pumping main is limited considering a high pump head (100 m) and water hammer caused by pump operation)

1.5 m/sec for other pumping main

Pipe material : Mild steel with welding joint for pipes larger than dia.1200 mm

Ductile iron for pipes dia.1200 mm or smaller

Inside lining : Mortar cement lining

Outside coating : Coal tar epoxy coating with asbestos felt wrap for steel pipe

Coal tar epoxy coating for ductile iron pipe

(2) Pipeline Details

1) Treatment Plant to High Level Reservoir

Number of pipeline	:	1 line
Diameter	:	1650 mm
Length	:	3,000 m
Pipe material	:	Mild steel (welding joint)
Flow	:	Pumping from the treatment plant
Maximum velocity	:	0.99 m/sec