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FEASIBILITY STUDY
TECHNICAL FINAL REPORT

VOLUME II (APPENDICES)

WATER SUPPLY

LIPA CITY WATER DISTRICT

JUNE 1976

FOREWORD

Volume II (Appendices) of the Technical Final Report on the Lipa City Water District Water Supply Feasibility Studies contains detailed information relating to specific sections of Chapters VII, VIII, IX, X and XI in Volume I.

Appendices A, B, C and D refer generally to several chapters of Volume I. Appendix A, Design Criteria, is fundamental to studies in all chapters of the Technical Report. Appendix B, Basis of Cost Estimates, has been used in the preparation of cost calculations in Chapters VIII and IX. Appendices C and D are related to Appendix B and to the construction of the recommended plan described in Chapter IX.

Appendices VII-B and VII-C provide additional data on water resources. Appendices VIII-C, VIII-D and VIII-E are discussions of alternatives for water treatment, distribution, and water conservation and augmentation, respectively. Appendix IX-B is a list of miscellaneous improvements included in the Early Action Program. Appendix IX-C covers projections of areas to be served by service connections and internal network piping, distribution system costs and computer studies. Steps in the management of groundwater resources and the updating of the water supply master plan are given in Appendices IX-H and IX-I. Appendix IX-J assesses the possible positive and negative effects of the water supply project on the environment. The project's financial and development costs projected from 1976 to the year 2000 are tabulated in Appendices X-B, X-E, X-F and X-G. The values of economic benefits and the economic costs are explained and tabulated in Appendices XI-C and XI-E.

The appendices are numbered according to the Volume I chapter and section to which they refer. Thus Appendix VIII-D refers to Chapter VIII, Section D of Volume I. The figures and tables are numbered consecutively after the chapter and section designation.

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APPENDIX A

DESIGN CRITERIA

APPENDIX A DESIGN CRITERIA

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APPENDIX A

DESIGN CRITERIA

General

The following may be considered as design criteria for the long-range facilities for water supply purposes. However, in view of scarcity of funds and financial feasibility, the criteria for the initial and emergency stage may be of somewhat lower quality.

Study Area

The study area will be determined for the present and future water service area of the water district. General topography, natural barriers, municipal boundaries, zoning plans will be taken into account in the determination of the future service area limits.

Population Projections

The total service area population would be projected on the bases of separate projections for the core city or poblacion and for the barrios within the present and future service areas. Transient population such as students, tourists, refugees, will be included in these estimates.

A percentage of population served will be estimated for the present and future systems. This estimation of percentage population currently served in the study area will be based on number of piped water connections and average number of people per urban household as obtained from the official census books. In the estimation of future population served, cost and availability of the water from sources other than the water district would be considered.

Land Use Projections

Residential, institutional, industrial, commercial and public area within the water service area will be designated either from the existing master development or zoning plans of the community or from data on other cities with similar characteristics. A projection of the land use pattern for the study area will be shown on a map and summarized in a table.

Pressure Zones

Depending on general topography of the water service area there may be one or more service pressure zones in the water distribution system. The maximum difference in ground levels in any pressure zone will not be more than 50 meters.

Separate supply lines from the source will be provided for each zone where economically feasible.

Unit Water Demands

a. Domestic: Average per capita domestic water consumption will be estimated for the study area. Past water district records and records from similar cities will be used for early estimates. When using the water district records, the actual metered customers and borrowers would be considered separately. For borrowers, an average unit consumption of 20 lpcd shall be assumed. However, the final estimates will be based on actual field measurements. Field measurement will be done by direct meter reading by isolating certain service area sections which will represent different economic classes of customers. This measurement will be conducted in areas which have adequate supply.

Unit domestic consumption will be increased by 1 to 2 per cent each year to account for economic growth within the community.

b. Institutional and Commercial: Institutional and commercial water demands will be estimated as a percentage of domestic demand based on available past records of the water districts or similar cities. If no records are available a unit flow of 5 cum/day per gross hectare will be used in the estimates for this purpose.

c. Industrial: At present, there is no heavy industry in the study areas. However, available zoning plans designate areas for future heavy industrial developments.

Past records on the water consumption of existing light industrial establishments will be studied to establish unit flows required per unit area. If no records are available, a unit flow of 10 cum/day/ha (gross) will be used in the estimates.

d. Parks, golf courses: Water demands for the public parks and golf courses will be estimated from the past records.

Unaccountable Water

With a review of the available water district records or consumption pattern in similar cities, a tentative percentage of the total supply requirement will be established for unaccountable water for the early studies. The final estimate of unaccountable water will be based on actual field measurements. Unaccountable water may include waste and unrecorded usage. It is assumed that the amount of unaccounted-for-water will be reduced gradually with the implementation of staged improvements to the system.

Total Supply

A total of various water demands and unaccounted-for-water will be the total water supply requirement for the study area. If there is more than one pressure zone in the study area, the required supply in each pressure zone will be estimated.

Demand Variation

Maximum daily and peak hourly demands in each study area will be estimated from the available records for service areas with adequate supply. If no data are available the demand factors would be obtained from other similar areas. An attempt will be made in the field to record hourly fluctuations for a minimum period of 24 hours for checking these assumed values.

For preliminary studies a maximum-day to average-day ratio of 1.2:1 and a peak-hour to average-day ratio of 1.5:1 - 2.0:1 will be used.

The present and future projected water demands will be tabulated.

Population and Demand Distribution

The study area will be divided into several sub-areas representing different population densities and demand patterns. Locations of the existing large demand customers (e.g., industry, military base, university campus, airport, etc.) and their water usages would be obtained through the water district records or field measurements. With these data, a demand load distribution will be made for the existing and future water distribution systems.

Existing Water System Analysis

After gathering all pertinent data, the existing system will be analyzed through a computer program. All the pipelines, 100 mm and larger, will be included in this study. Regular and large demands will be distributed at relevant nodes of the system skeleton.

Average-day demands will be included in computer input data. Demand factors will be applied for maximum-day and peak-hour flow conditions. About 5 per cent of unaccountable water will be allocated to transmission line and the remaining unaccountable water will be evenly distributed in the distribution system. The primary system (pipeline 4-in and larger) will be checked for only peak hourly demand condition to find out about areas with capacity shortage and low pressures. Any high level area which is being served by a booster station would be studied separately after establishing its hydraulic grade line (HGL).

If there is a storage tank floating in the system, the water level in the tank will be assumed to be at the middle of the operational storage portion, during peak hourly demand condition. In the computer application of the system, either the input flow or HGL at the source will be fixed. The following "C" values will be used for pipe friction losses.

a) Asbestos Cement Pipe

Size (mm)	100-150	200-300	350-500
"C" value	100	110	120

b) Cast Iron Pipe

Size (mm)		100-150	200-300	350-500
Age:	new	100	110	120
	10 years	90	100	110 1/
	20 years	80	90	105 1/
	30 years or more	70	80	100 1/

The internal distribution system would be checked for fire flow plus maximum-day demand. After computing the node pressures in the primary system for the maximum-day demand, a typical commercial residential area will be checked for fire protection. A fire flow demand of 15 lps (liters per second) will be applied at each one of two adjacent hydrants.

Computer Studies for Future System

The proposed system will be studied for the design year 2000

1/ Subject to field verification.

first and the economy of construction staging for 1990 shall be checked specially for supply, treatment and transmission facilities.

A system skeleton will be prepared for each pressure zone. Future pipelines will follow existing roads or proposed roads as much as possible. The maximum spacing between feeder main lines will not exceed 1,000 meters. For strengthening the system hydraulically all the pipelines will be looped as much as practical and economically feasible. The primary system which will be checked hydraulically first will include (200 mm) and larger pipelines. The projected average day demand loads will be distributed at nodes. For computer input, the pipe data will include a pipe number, connecting node numbers, diameter, length and "C" value; the node data will include a node number, ground elevation, and average day demand for the design year.

The maximum hydrostatic pressure in the system will not exceed 70 meters. If the existing water supply facilities were to be used, the pre-established HGL elevation would be evaluated carefully for deciding whether to continue to use them or to phase them out.

If a feasible storage tank site can be located in the system, a system input at a rate of maximum-day demand will be required. If no storage tank site is available then the system input will be at a rate of peak hourly demand. (In the case of well supply this means the total safe yield from the wells has to meet peak hourly demands.)

In the proposed system asbestos cement, cement lined cast or ductile iron, cement lined steel or prestressed concrete pipe will be used. The following "C" values will be applied throughout the studies:

Pipe Size (mm)	"C" Value
200-300	110
350-500	120
600-larger	130

A field cleaning and lining of existing large size mains will be considered as part of the improvement program. An operational storage volume of 15-20% of maximum-day demand at the design year will be provided (19% for Ozamiz and Clarin). The maximum operational level fluctuation in the tank will be 7 meters. If there is more than one storage in the system the operational volume required at each site will be determined through computer analysis.

Well pump capacities will be based on an evaluation of the pumping test of the well for yield and drawdown. In determining pump head characteristics the estimated minimum water level in the well, head losses through suction pipe assembly and the head required in the system would be investigated.

Booster pumps will be selected either to meet peak-hourly demands if there is no distribution storage or to meet maximum daily demands if there is an adequate storage. Each booster zone would be studied separately. The primary system (pipes 200 mm and larger size) will be checked for:

a) Peak-hour demand condition by applying a demand factor of 1.5 - 2.0. (For this condition it will be assumed that the system storage tank level is 2-3 m below the overflow elevation. The selected pipe sizes will be adequate for not creating a pressure less than 14 m at any point of the primary system).

b) Minimum flow plus tank filling if the storage tank site is located too far from the demand center. (The minimum flow is 30 per cent of the average daily demand).

The internal distribution network will be checked for fire flow plus maximum-day demand, at least at two typical areas: (1) a high value commercial area (for a fire flow of 20 lps from each of two adjacent hydrants); (2) a residential area (for a fire flow of 10 lps from each of two adjacent hydrants).

Computer runs will be repeated with revised pipe sizes until the system meets the design criteria.

Special effort will be made to utilize all or portion of the existing facilities as much as feasible. Data which would be required on the existing facilities for this purpose are as follows:

Supply facilities	:	HGL elevation and variation Flow input capacity
Pipelines	:	sizes, locations, "C" values
Pump Stations	:	pump curves, rated head and discharge values, HGL elevations on the suction and discharge sides, pump age, condition
Storage Tanks	:	overflow elevation, side water depth, operational depth, type, condition
Wells	:	safe yield, water level

Hydrants : inlet-outlet characteristics, locations

Valves : check valves, closed or throttled valves

Pipes

In evaluating and selecting the pipe material for use in the proposed improvement program of the study area waterworks system, careful consideration should be given to the following:

a. The pipe strength to resist both internal and external pressures;

b. Service life of the pipe material (resistance to corrosion, erosion and disintegration);

c. Pipe laying and jointing (simplicity, reliability);

d. Operation and maintenance problems; and

e. Economic consideration

Pressure class requirement for major transmission lines will be investigated on a pipeline profile. Working pressures will include additional allowances for surges and water hammer. Minimum pressure class of pipe will not be less than 7 kg/sqcm.

Generally, concrete pipe and cement-lined pipe have a better average coefficient of friction than unlined cast iron, ductile iron or steel pipe.

Because of the brittleness of the material, the use of cast iron pipe and asbestos cement pipe is generally limited to the smaller sizes. In addition to the inability to take large bending loads, with brittle pipe, sudden failures can occur and discharge large volumes of water that not only cause extensive damage, but may also put the water system out of operation for a long period.

A high sulfate content of the soil will limit the use of concrete or asbestos cement pipe or require special protective coating. When the sulfate concentration in the soil exceeds 0.5 per cent (or 300 mg/l) unprotected concrete pipe should not be used. Many types of soil can be corrosive to ferrous metal pipe. A corrosion survey along the pipeline routes will be necessary to locate extremely corrosive areas so that suitable types of pipe material and protective systems can be selected.

A minimum trench width of 0.60 m would be specified for new pipelines. Trench width will increase with the pipe size as shown in the following formula:

$$\text{Trench Width} = 0.50 + D \text{ (m)}$$

The minimum cover on a pipe shall be 0.60 meters. If there is a traffic load, the minimum cover shall be increased to 0.90 m. If this can not be accomplished, the pipe shall be encased in concrete.

Recommended Pipe Materials

A final pipe selection can be based on economic cost comparison, which may be made among the recommended pipe materials for the required service and capacity as shown in the following table:

Pipe Material	Diameter (mm)				Service	
	10-100	150-400	450-600	700-1200	Distribution	Transmission
Prestressed Concrete	-	-	-	x	-	x
Steel	x[2]			x	-	x
Cast Iron	-	x	x	-	x	x
Ductile Iron	-	x	x	x	x	x
Asbestos Cement	-	x	x	-	x	-
Polyvinyl Chloride or Polyethylene pipe[3]	x	-	-	-		

Pipe class should be in accordance with the required operational pressures in the system.

Pipe Cleaning and Lining in Place

It is possible to increase capacity of old transmission and distribution pipelines by 20 to 50 per cent with cleaning and lining process. This is specially true where extensive internal scaling has occurred in the pipeline. Experience shows that cleaning and cement lining in place of 150 mm diameter and larger water mains are more economical than installing new mains to obtain the same capacity increase. Therefore, cleaning and cement lining in place will be included in the improvements program where extensive capacity losses in the water mains are observed.

Valves

To isolate and drain pipeline sections for test, inspection, cleaning and repair a number of valves are generally installed in the line. The most commonly used valves are gate and butterfly valves followed by check, cone valves, blow-off and air release valves. Despite the wide range of designs, all valves have only one purpose: to slow down or stop the flow of water. In a distribution system, large numbers of shut-off valves (gate and butterfly) are utilized. Gate valves are more applicable to pipe sizes up to 300 mm in diameter. For larger size pipelines butterfly

[2] Galvanized steel pipe.
[3] Service connections only.

valves will be used.

Valves in a distribution system will normally be located at street intersections. The valve spacing in high consumption areas would be closer than low consumption areas. A maximum valve spacing of 300 to 500 m will be considered in preliminary layouts. However, the final determination will depend on judgement of conditions in a particular system.

Valves shall be equipped with handlever, handwheels, chains or hand, phematic or electric operations.

The minimum working pressure of valve will be in accordance with the service requirements. Valve design and manufacture will conform to the current AWWA or ASTM specifications.

Fire Hydrants

Fire hydrants will be located at street intersections as much as possible. Spacing and sizes will be as follows:

a) High value residential, commercial and industrial areas:

Spacing	:	150 m, maximum
Connecting pipe size	:	100 mm, minimum in looped systems 150 mm, minimum in dead-end systems
Hose outlet	:	1 x 60 mm ($2\frac{1}{2}$-in)
Pumper outlet	:	1 x 100 mm (4-in)

b) Normal single family residential areas:

Spacing	:	250 m, maximum
Connecting pipe size	:	100 mm (4-in)
Hose-Pumper outlet	:	1-100 mm (4-in)

The exterior surface of fire hydrant will be painted for protection and easy location.

Flow Meters

A. Differential Head Meters

The flow of fluid through a constriction in a pressure conduct results in lowering of pressure at constriction. The drop in piezometric head between the undisturbed flow and the constriction is a function of the flow rate. The venturimeter, flow nozzle and orifice meter are constriction meters which make use of this principle. The head loss through a venturimeter is considerably less than for the other two types of meters. Pitot tubes and pitometers may also be typed as differential head meters.

B. Mechanical Meters

Mechanical meters are widely used in water distribution systems. Two types of mechanical meters in common are positive displacement and propeller meters. The positive displacement type meters are more accurate in measuring small flows. This type of meter is not recommended, however, for waters having fine particles as it is likely to become inoperative due to clogging.

Plumbing Code

The Philippine National Plumbing Code shall be applicable.

Distribution Storage Tanks

Distribution storage tanks are used to provide storage volume to meet fluctuations in water use, to provide fire storage, and to stabilize pressures in the distribution system. The tank in relation to the service area, should be located as much on the opposite site from the source as possible; on the other hand, the tank location should not be too far away from the demand center. A storage tank is normally located at a sufficiently high point so that water level in the tank can control the hydraulic grade line and fluctuate with the variation of system demand. A tank refills when the demand is low and feeds into the system when the demand is high. With an adequately sized storaged tank it is possible to have supply and transmission facilities operating, more or less, at a steady rate which is normally to be around maximum daily demand for the design year.

The total effective storage volume required in a service zone should be at least equal to the required operational storage. Fire and emergency storage may be provided if economically justified. As mentioned previously, the equalizing or operational volume is to be equal to 15 to 20 per cent of the maximum daily demand in any design year. A maximum side water depth (or level fluctuation) of 7 m will be assumed for the operational storage.

The maximum hydrostatic pressure in any pressure zone in the distribution system should not exceed 70 meters. The tank overflow elevation, therefore, will be set at a level which will be a maximum of 70 m higher than the lowest ground level in the service area. A storage tank can be a ground type, elevated or a stand-pipe, all covered. A tank shape can be rectangular or circular. Roof slab of a tank will be supported on interior columns. The tanks are normally constructed from reinforced concrete or steel. Reinforced concrete tanks would have less maintenance costs and also will not require foreign exchange.

Piping in a tank will consist of incoming flow pipe, overflow and drain pipe, and outgoing flow pipes. From those, all of the pipes, with the exception of overflow pipe, are valved.

For large cities within a same pressure zone there may be a need for more than one tank site. In this case volume distribution at each site can be determined through computer analysis. Sufficient land should be taken for the tank site to accommodate short-term as well as future storage units for the service area.

Booster Pump Stations

A pump station structure and related piping will be designed for a period of 25 years. On the other hand, the equipment including pumps and motors shall be designed for about 15 years.

Selection of pumps will be based on system-discharge curve. With development of composite pump curve for the number of existing and proposed pumps at a station and application of this curve on the systems, the head-discharge curve indicates rated flow and head for the pumps. Where pump is pumping directly into distribution system, the system curve shall be studied through computer analysis.

To prevent excessive pressures in the pumped supply system (specially during minimum demand periods), pumps will be selected with a shut-off head which will not be greater than the rated head more than 10 per cent.

If the water has to be pumped through a long transmission line before it reaches the distribution system, an economical study may be necessary before deciding on pumping head versus transmission pipe size. In this study, pressure limitation in a distribution system has to be taken into account. If there is an adequate storage in the system, the pump station can be designed for the maximum daily demands estimated for a particular design year.

If there is no feasible site available for a storage tank, pump stations will have to be designed for peak hourly demands.

The total design head of a pump should include static head and dynamic head which consists of friction and turbulence losses in suction and discharge piping.

Pump drive will be either an electrical motor or a diesel engine. Economy and practicality of electric or diesel power will have to be studied for the study area.

Electricity is 480 volts, three-phased with 60 cycles. Local needs for additional power transmission line and a substation will have to be investigated.

In addition to manual start-stop, each pump station shall be equipped with high pressure sensing device to automatically stop the pump on a high discharge or low suction pressure. For proper operation, maintenance and safety of a pump the following quipment generally provided on the discharge line are: shut-off valve, check valve, surge relief valve, pressure gage, flow meter and air and anti-vacuum valve.

Each pump station should have a superstructure constructed from locally available material to help provide security from theft and vandalism and to minimize the noise problem in residential areas.

Sufficient land should be taken for a pump station to accommodate short-term as well as future facilities.

Pumps are to be constant speed single stage, horizontal or vertical centrifugal type. The minimum number of pumps in any station will not be less than two. Pump ratings, make and model in the system will preferably be the same for simplification in operation and maintenance. A stand-by capacity equal to the largest pump in a pump battery will be desirable for assuring the firm capacity of the station even when one pump is out of service. Where electricity is not reliable, consideration should also be given to having one of the larger pump motors close-coupled with a diesel or gas driven engine. By this, at least part of station capacity will be available in the event of power failure.

Water Quality Criteria

The water provided by a public water supply system should be free from substances harmful to human health and should be of the highest quality that is economically feasible. An acceptable water should have the following general qualities:

a. Water should be free from pathogenic organisms and at all times free from suspicion of being a means of conveying disease;

b. Toxic substances in the water should be below the concentration that would be injurious to health.

c. The water should be free from encrusting or corrosive properties and should be clear, colorless, tasteless and odorless.

Maximum acceptable concentrations of some of the most significant constituents of water, as established by the Philippine National Committee on Drinking Water Standards, and by the World Health Organization (WHO) are to be used as guideline.

Surface Water Sources

The treatment of water from a surface source must be economically feasible and should meet the current requirements of Philippine Drinking Water Standards with respect to bacteriological, physical, chemical and radiological qualities.

The quantity of water at the source(s) shall be adequate to supply or supplement the water demand of the service area at least until the year 2000.

Hydrological Studies

Daily or monthly streams flow records, if available, should be used in the safe yield estimates. If stream flow records are not available, correlations with similar basins with longer period of records, based on drainage areas, should be made. Recommendation should be made for the establishment of stream gauging stations for use in design. For extension of stream flow records for a longer period, rainfall-runoff correlations can be used. The minimum recorded flow minus riparian rights would give the minimum amount of water available from a particular source for the study area. This minimum flow rate will be compared with the estimated total gross water demand in the study area to decide whether any impoundment is needed and when needed. Without a need for impoundment reservoir a water diversion and intake structure would be required for the supply. If an impoundment is necessary to meet the demand an investigation will have to be carried out on possible dam sites. Dam sites can be located, first, on 1:50,000 topo-maps. As a result of a preliminary field investigation covering area geology, accessibility and major relocation due to reservoir impoundment, some of the sites can be eliminated. For the selected sites mass inflow curves will be plotted covering at least one significant dry period. Mass inflow curve should be adjusted for evaporation and riparian rights. Demand lines drawn tangent to the high points of the mass curve represent rate of withdrawal from the reservoir. Assuring the reservoir to be full wherever a demand line intersects the mass curve, the maximum departure between the demand line and the mass curve represents the net reservoir capacity required to satisfy the demand. Usually some volume in the reservoir, which may be also called dead volume, is allocated for siltation.

In order to determine the basic dam height, area-volume curves are drawn up for each feasible dam site. From these curves a dam height can be selected which would create enough volume of reservoir to satisfy the demand and dead volume requirements. Estimated reservoir volume will be increased by 25% for supply safety. In some cases one stream may not yield sufficient quantity of water. Then it will be necessary to look into other river basins or ground water, for supplementary supply.

In other cases the same stream may be considered for multipurpose basin development including power, irrigation and navigation. This will require close cooperation with the other authorities to make sure that adequate amount of water will be available for municipal usage. In accordance with the governmental requirements in the Philippines any proposed dam 60 m or higher must be communicated to the National Power Corporation.

Raw Water Pump Stations

Location, arrangement, type of equipment and structure are important aspects of a pump station to be investigated in the design. Before deciding on a raw water pump station, an economical cost comparison will be made for gravity flow though a tunnel alternative if area topography is suitable. A raw water pump station usually requires an approach channel, intake structure which will be equipped with stop logs, bar screen and control gates, and pump wet well.

The station will be designed for the maximum daily demand in the design year. Pumps will be capable of delivering the design flow at the maximum head which is the sum of differential static head, suction lift (if any), and suction and discharge head losses. Selection of pumps in the station will be based on the application of pump curves on system head-capacity curve.

Electric motor or diesel engine driven vertical turbine pumps will be used for the raw water pumpage.

Staging of Source Development

During source development studies a demand versus supply chart will be prepared to show a timely staging of facility construction. A demand-supply chart will include the supply from the existing sources which, in some cases, may be phased out if economically justified, following the development of new sources.

The staging of construction will be in accordance with the following demand conditions:

Facility	Demand
Dams	Average-Day
Water Treatment Plant	Maximum-Day
Diversion and Intake	Maximum-Day
Transmission Lines	a) Peak-Hour if no distribution storage b) Maximum-Day if there is an adequate storage

A sufficient time shall be allowed for planning, design and construction of future facilities.

Surveys

Water quality surveys are important as they would indicate cost of treating the proposed source of water. Water samples will be taken from all the sources and laboratory analysis will be made. Topographical surveys at 1:2000 scale will be required at dam sites for facility layouts.

Groundwater-Springs

Springs can be developed as gravity or pumped supply. In both cases sufficient period of flow measurements will be needed for determining the minimum yield. The yield of some springs may be increased by direct pumpage; however, before doing this a careful evaluation of aquifer and recharge area should be made to avoid possible damage to the spring. The major works needed for spring development would be construction of a collection chamber with necessary piping arrangement. Water quality must be checked to see whether any treatment would be required; the most likely quality problem with spring water being either excessive hardness, or iron and manganese. The spring recharge area must be protected from pollution.

In the construction staging of spring development, the measured minimum yield should meet the maximum daily demand of the study area, if adequate storage is provided for peak-hour demands. With no storage in the distribution system construction staging should correspond with peak-hour demands.

Groundwater Wells

All the available data pertinent to existing wells in the study area will be collected and evaluated for the purpose of determining well and aquifer parameters including water table elevations, well yields and drawdowns, well geometries, interference between wells, and water quality. In addition, geologic, hydrologic and meteorologic data will be evaluated with information on current withdrawals to estimate recharge to aquifers and to estimate the overall safe yield of the source.

In many cases, it may be necessary to construct and test several wells to obtain the necessary data. Test well sites and depths will be chosen to provide data on unexplored important sections of the aquifer. For each test well, a minimum number of two observation wells would be desirable.

Based on available information and test well results, the aquifer coefficients will be estimated. With this and hydrogeological appraisal of the area, practical design yield, well size, depth and spacing can be planned. Water quality analysis will indicate treatment requirements of the source.

Construction staging of wells should follow the same criteria as explained for spring development.

Water, in general, has to be pumped from a groundwater well with the exception of flowing artesian wells with adequate yield. Pumps normally used for this purpose are either multi-stage vertical turbine pumps which are shaft driven by motors or engines located on top of well or submersible pumps in which the pumps and electric motors are combined in one unit placed below the water surface of the well. The pump bowls may be set at approximately 5-10 m below the lowest anticipated pumping level. The lowering of the water table in a given aquifer and the specific capacity of the well must be taken into consideration when calculating the anticipated ultimate pumping level.

Where the source of electric power is not reliable, diesel engines will be considered for pump drive units. Pumping head will be determined by pumping level in the well and minimum pressure requirements in the distribution system during peak-hour demands or by tank filling operation during minimum hour demands.

Water Treatment Works

Objectives of Water Treatment. In the design of water treatment plants, the provision of safe water is the prime goal. The treated water must be clear and colorless and pleasant to the taste. Water quality obtained at the plant should be preserved in the distribution system. The control point for the determination of water quality is the consumer's tap and not the outlet of the treatment plant. Another basic objective is that water treatment be accomplished using facilities that are reasonable with respect to capital and operating costs. In plant design the various alternatives will be investigated including plant performance and cost studies.

General Design Considerations. Where previous experience with treatment of the same or similar source is lacking, special studies would be necessary for design purposes. These special studies may include tests conducted in the laboratory, in existing plants or in pilot plants. The rated or nominal design capacity of the treatment plant will be the maximum daily water demand of the system for the design year. Using water demand projections, a logical program for development of treatment facilities may be established. Decision will have to be made about which units to be built initially for ultimate needs or to provide for development in stages. The following are factors which have a bearing on the period of design of treatment facilities: (1) the useful life of facilities, (2) the ease of extension, (3) the rate of growth of the service area, (4) the rate of interest on the loan, (5) the change of purchasing power during the debt period, and (6) the performance of facilities during the early years.

Pumping station and chemical building structures are to be constructed for ultimate capacity; pretreatment and filter facilities are to be built in stages as the need develops.

For operational safety, even in the initial stage of construction, none of the important units such as flocculation, settling and filter basins is to be less than two. Stand-by units will be provided for specially when the plant treats a water that is highly contaminated.

An evaluation of available sites will be made to determine the most favorable location for the plant. An accurate estimate of the area required for the ultimate development of the site is specially important.

In plant sizing and layout, the following points will be considered:

(1) Frequency of basin cleaning, length of filter runs and effluent quality will be carefully evaluated.

(2) An economic but durable construction: outdoor type filters can be adopted in the Philippines. Construction items will be selected for a minimum service period of 50 years.

(3) The smallest number of units that is feasible will be chosen, but the number will be sufficient to provide stand-by capability.

(4) Operation of filters, flocculators and chemical feeding equipment requires the most attention of operators. It is therefore desirable to arrange the plant so that these functions are close together, rather than widely separated.

(5) Chemical feed lines are to be as short and direct as reasonably possible. For this reason, it may be necessary to place the rapid mix basin in the chemical building.

(6) Chemical handling and feeding system will be simplified.

(7) Unessential instrumentation will be avoided.

Types of Water Treatment Plants

The quality of raw water varies greatly from source to source. Accordingly, the type of treatment to produce a safe and palatable water will vary. The World Health Organization has established treatment requirements in relation to the coliform bacterial content of raw water.

Application of treatment methods in relation to raw water characteristics is shown in **Appendix Table A-1.**

Classification of treatment plants according to raw water quality is a useful guide to the designer. However, such classification is not a substitute for engineering studies including, in some instances, experimental and pilot plant work as the basis for plant design.

In a modern conventional plant, rapid mixing, flocculation, sedimentation, filtration and chlorination are employed to remove color, turbidity, tastes and odors, and bacteria from surface water supplies. Bar racks and coarse screens are provided if floating debris and fish are a problem; aeration is beneficial and economical for treatment of tastes and odors; presedimentation would be required if the water is highly turbid.

Water filters can be designed hydraulically as slow or rapid, depending upon the rate of flow per unit of surface area. The processes of a treatment plant are briefly explained in the following sections.

<u>Aeration</u>. Aeration is used to reduce the concentration of taste and odor producing substances in the water and to remove iron and manganese from the water by oxidation. Aeration can be accomplished by waterfall aerators, spray nozzles, cascades, multiple trays, diffusion of compressed air through the water, and mechanical aerators. Approximate area requirements for different types of aeration are shown in the following table:

Type of Aeration	Area Requirement sqm per 1,000 cum/day
Spray	2.50
Multiple Tray	1.25
Cascade	1.25
Diffuser	1.75

Inclusion of aeration process can be useful and economical in the treatment of ground water which has a high content of carbon dioxide, iron and manganese and hydrogen sulfide.

<u>Mixing</u>. Coagulation of particles in the water with the addition of chemicals is accomplished during mixing processes. Where only a coagulant is used or where sequence of application

APPENDIX TABLE A-1

APPLICATION OF TREATMENT METHODS[4]

Water Quality		Pretreatment				Treatment				Special Treatments				
Constituents	Concentration	Screening	Prechlorination	Plain Settling	Aeration	Lime Softening	Coagulation and Sedimentation	Rapid Sand Filtration	Slow Sand Filtration	Postchlorination	Superchlorination[5] or Chloramonia-tion	Active Carbon	Special Chemical Treatment	Salt Water[6] Conversion
Coliform MPN per 100 ml (monthly average)	0–20									E				
	20–100			O			O	O	O	E				
	100–5,000		E				E	E	O	E				
	>5,000		E	O[7]			E	E		E	O			
Turbidity-units	0–100	O							O					
	10–200	O					E	E						
	>200	O		O[8]			E	E						
Color-mg/l	20–70						O	O			O			
	>70						E	E			O			
Tastes and odors	noticeable		O		O				O		O	E		
Calcium carbonate-mg/l	> 200					O	E	E	E				E	
Iron and manganese-mg/l	< 0.3		O	O				S						
	0.3–1.0				O		E	E	O					
	>1.0		E		E		E	E	O				O	
Chloride-mg/l	0–250													
	250–500													O
	500[5]													E
Phenolic compounds-mg/l	0–0.005						O	O			O	O		
	>0.005						E	E			O	E	O	
Toxic chemicals							E	E				E	O	
Less critical chemicals							O	O				O	O	

[4] E—essential; O—optional; S—special justification required.
[5] Superchlorination shall be followed by dechlorination.
[6] As alternate, dilute with low-chloride water.
[7] Double settling shall be provided for coliform exceeding 20,000 M.B.N.
[8] For extreme muddy water, presedimentation by plain settling may be provided.

Source: Water Treatment Plant Design, ASCE, AWWA, CSSE, 1969 edition is not critical, chemical mixing may be obtained by injection of chemicals into a point of high velocity flows, such as the suction of a low-lift pump, a parshall flume, or a hydraulic pump. In other cases power may be put into water to secure mixing either by mechanical agitators or by use of gravity in baffled basins. The rectangular baffled basins are usually designed for horizontal flow with a detention time of 60 seconds at the design flow. Basins with mechanical agitators may be designed for a detention time of 30 seconds. Design of mechanical rapid mixing basin is based on the rate of power input into the water as measured by the velocity gradient. Because the best velocity gradient may vary from time to time at given location, variable speed equipment is desirable for agitators. Power requirement is about 1.3 hp per 10,000 cumd flow. A recent trend in chemical mixing favors use of in-line blenders.

Coagulation and flocculation are greatly influenced by physical and chemical characteristics of water, including particle size and concentration, pH, water temperature, exchange capacity and electrolyte concentrations. The behaviour of water to be treated in a proposed plant can be best determined by: (1) laboratory testing using "jar test" technique, followed by laboratory filtration or (2) pilot plant.

The sequence of addition of chemicals for coagulation is often important and multiple points of application of the chemicals are therefore required. The chemicals ordinarily used are a pH-adjusting compound, such as lime or an acidic substance, the coagulant (normally aluminum sulfate or a ferric compound), and a coagulant or flocculation aid. Prechlorination treatment is commonly applied to water before or after a coagulant. Activated carbon for taste and odor control is usually applied at raw water intake to provide sufficient period of detention time.

Flocculation. Flocculation process follows chemical mixing. Detention time used for the design of flocculation basins will be 60 minutes. To increase floc strength, usage of chemical agents such as activated silica and polyelectrolytes may be considered. For the provincial areas in the Philippines non-mechanical type of baffled flocculation basins may be economical. A distinct advantage of baffled flocculation basins is elimination of short circuiting of flow. However, the mixing intensity in this type of basin is dependent on flow rate.

The easiest way to manage flocculated water is to build the flocculation and sedimentation basins integrally, with a permeable baffle discharging the flocculated water into the sedimentation basin to assure uniform horizontal and vertical distribution of settling tank influent.

Sedimentation. This process usually finds application in two principal ways in water treatment: plain sedimentation and sedimentation following coagulation and flocculation. Plain sedimentation is usually used to reduce heavy sediment loads prior to complete treatment; therefore it is often referred to as presedimentation.

Sedimentation following chemical coagulation and flocculation is used to remove color and turbidity by adding coagulants, and to remove hardness by adding lime and soda ash. This type of sedimentation follows presedimentation (if used) and aeration and precedes filtration.

In the design of sedimentation tanks, ideally, four zones are considered:

a) an inlet zone to provide smooth transition from the influent flow to the uniform, steady flow desired in the settling zone. In general, the flocculation and settling basins are located in the same rectangular tank to eliminate the need for a channel inlet.

b) a settling zone to provide tank volume for settling, free from the other three zones.

c) a sludge zone to receive the settled material and prevent it from interfering with the sedimentation of particles in the settling zone.

d) an outlet zone to provide smooth transition from the settling zone to the effluent flow. The water level in settling tanks is usually controlled at the outlet. Basin outlets are often of v-notch weir type, and these are quite often provided with means for vertical adjustment to aid in control of the overflow.

Most sedimentation tanks used in water purification today are of the horizontal flow type. The other types are known as upward-flow solids contact units and upward-flow sludge blanket type clarification basins. Because of simplicity in construction, operation and maintenance the horizontal-flow type basins are expected to be applicable in the Philippines.

Horizontal-flow tanks may be either rectangular or circular in plan. Circular horizontal-flow tanks are usually center feed type with radial flow. In a rectangular tank the flow lines are parallel and all in one direction. The flow usually enters one end of the tank through a perforated or diffusion wall, travels the length of the tank, and then exits over some type of effluent weir. The choice of rectangular or circular horizontal-flow type is usually based on designer's preference and site limitations. Many sedimentation basins are equipped with mechanical equipment for the continuous removal of settled solids.

The standard approach in designing a sedimentation basin is to satisfy design criteria that have been arrived at through experience with full-scale plant operations and from pilot-plant research. Raw water quality varies from one source to another, therefore, only tentative design criteria can be established for preliminary design works.

The temperature of the water, the specific gravity of materials in suspension, and the size and shape of the suspended particles influence sedimentation process. Experience has shown that higher tank overflow rates can be used in warm waters. A particle with higher specific gravity will settle faster. The time of retention in the sedimentation tank is important, because longer time permits more floc contacts and, hence, more floc growth.

The purpose of the settling tank is to hold the water for a period of time during which the velocity of flow through the tank has been greatly decreased to allow sedimentation to occur. The main characteristics of sedimentation tank involved include the tank surface area, which is dependent on the surface overflow rate, the tank depth, which is dependent on the detention time, the velocity of flow through the tanks, which is a function of the cross-sectional area of the tank, which in turn is a function of the length/width ratio of the tank, its surface area, and depth.

Preliminary design parameters of settling basins are shown in Appendix Table A-2.

APPENDIX TABLE A-2

DESIGN PARAMETERS OF SETTLING BASINS

Raw Water	Treatment	Overflow Rate (cum/day/sqm)	Detention Time (hr)	Velocity Through Basin m/min	Tank Depth (m)
Surface	Alum floc[9]	25–50	2–4	0.15–0.50	3–4
	Ferrous floc[9]	30–50	2–4	0.15–0.50	3–4
Surface or ground	Lime softening	40–60	1–3	0.20–0.60	3–4
	Without subsequent filtration	10–20	8–12	0.05–0.20	4–5
	Plain sedimentation	100	1–4	0.3 –1.0	3–5

Rectangular tanks can be constructed with practical lengths up to a maximum of about 80 meters. Generally, a length to width ratio between 3:1 to 5:1 is used. Rectangular tanks will have a minimum depth of about 2.5 m and a recommended depth range from 2.5 to 5 meters. Where area is available, the shallower depths are preferable. In addition to the calculated settling basin, a provision for inlet, outlet and sludge collection zones, will be made.

The number of tanks to be provided is determined by the total flow, desired degree of flexibility of operation, and economy of design. A minimum of two basins must be provided. In larger plants, the number of units provided may be determined by the maximum practical size of a single tank.

The calculated width or diameter of a tank would, later, be adjusted to the next standard size of tank, for which mechanical collectors are available, and for rectangular type the length would be adjusted accordingly. Basins not provided with sludge removal equipment will be made deep enough to provide sufficient volume of sludge storage capacity. Typical basin outlet overflow rates are shown in Appendix Table A-3. In rectangular tanks, the overflow weir length required cannot usually be obtained with a single weir across the end of the tank. The required length is usually provided by a weir extension in the third outlet of the tank.

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[9] With subsequent filtration.

APPENDIX TABLE A-3

TYPICAL WEIR OVERFLOW RATES

Type of Treatment	Weir Overflow Rate cum/day/m
Light alum floc (low turbidity water)	150
Heavier alum floc (higher turbidity water)	200
Heavy floc from lime softening	300

If gravity discharge of the sludge from the mechanically cleaned sedimentation tank is not feasible, sludge pumps of sufficient capacity must be installed.

Filtration. The goal of water treatment is to obtain the greatest clarity (or lowest turbidity) of the filter effluent. Water filtration is a physical and chemical process for separating suspended and colloidal impurities from water by passage through a porous medium, usually a bed of single or multi-layer granular material.

Filtration may be classified hydraulically as slow or rapid, depending upon the rate of flow per unit of surface area. Slow sand filters operate at a rate as high as 9 cum/day/sqm, and rapid or high rate filters operate as high as 20 cum/hr/sqm. One of the principal drawbacks to the use of slow sand filters is the large land area required. Another is the difficulty of getting good results under all raw water conditions. Slow sand filters are cleaned by scraping a surface layer of sand and washing the removed sand and returning it to the bed. Algae growth is another problem with slow sand filters specially in hot climates. As slow sand filters require minimum amount of mechanical equipment it may be considered in the provincial areas of the Philippines where plenty of land is available and when it is justified economically.

In the design of new plants, the gravity rapid filter with coarse-to-fine media (dual media) is the obvious choice for the great majority of installations. The best example of this is the coal-sand filter with a coarse coal layer of about 18 in deep above a fine sand layer of about 8 in thick. The filter media are supported by an underdrain system. The most important function of the filter underdrain is to provide uniform distribution of backwash water. It also serves to collect the filtered water. With many types of filter bottoms or underdrains, a supporting bed of gravel is used to keep the sand out of the underdrain and clearwell during filtration and to assist in uniform distribution of washwater during cleaning of beds. A gravel depth of 12 in is usually adequate. The silica sand used in the filter media is specified to have an effective size of 0.35–0.50 mm and uniformity coefficient of about 1.7. Crushed anthracite coal has a specific gravity of 1.5, as compared to 2.65 for silica sand. Effective sizes of coal up to 0.7 mm are used in filters.

Efficiency of dual media filters can be increased by the use of polyelectrolyte filter aid in small dose, usually 0.01–0.05 mg/l.

Warm water is easier to filter than cold water. Filtrability is the most important property of the applied water. Pilot plant studies are strongly recommended in preparation to filter plant design not only for filtrability of raw water and filter design but also for the measurements of wash rates and expansion required to fluidize the proposed bed.

The usual number of filter units is four, except in small plants where it may be two. The maximum size of individual filter units is governed principally by the rate at which washwater must be supplied and by problems in securing uniform distribution of washwater that increase with larger areas. The largest filter unit normally employed is about 200 sqm. A unit of this size would be divided into two units of equal size, so that each half could be backwashed separately. For the preliminary design a filtration rate of 10 cum/hr/sqm will be used.

Filters are usually laid out side by side in rows along one side or along both sides of a pipe gallery. One end of the row of filters should be kept unobstructed to permit future expansion. In proposed plants in the Philippines the filter tops will be open as there will be no freezing problem. Clear-well storage will be located not underneath the pipe gallery but in an area adjacent to the filter basins.

Depth of water over the filter media for warm water may be about 1.5 meters. This much of adequate water depth above the media would reduce the possibility of air binding during loss of head operation.

Filter backwashing is done to remove from the bed all of the foreign material collected in the bed during the preceding filter run. In warm climates a maximum upward backwash flow of 50-60 cum/hr/sqm must be provided. Wastewater from backwash is collected in washwater troughs and conveyed into a waste drain.

Filters are equipped with a means of controlling the rate of flow through each bed.

Bacterial removal by filtration is never 100 per cent, and the filtered water must be chlorinated for satisfactory disinfection. Provisions should be made to chlorinate filter influent and effluent.

Appendix Table A-4 shows the recommended velocities for water filtration units:

APPENDIX TABLE A-4

RECOMMENDED VELOCITIES FOR FILTRATION UNITS

Location	Velocity (m/sec.)
Influent	1.0
Effluent	1.5
Backwash	3.5
Waste	2.0

Cost Estimates

The construction cost estimates of proposed improvements will be based on projected July 1976 unit prices. The estimates will show foreign and local cost components of the project cost. Construction cost projections will be made for all items which will be included in a water supply project. When using a source information outside the Philippines necessary adjustment will be made to reflect the local labor cost. All estimates will be based on an exchange rate of ₱7 to 1 US dollar. It will be assumed that no customs duty will be charged on items imported for public water supply projects. Separate cost indices for local and foreign cost components will be developed. Cost tables will be prepared to show a breakdown of the estimated construction cost for major items.

The total project cost of any alternative scheme will be computed in the following manner:

1.	Construction Cost:	A
2.	Engineering and Contingencies	0.25 A
	Sub-total	B
3.	Land Cost	C
	Sub-total	D
4.	Administrative and Legal Fees:	0.03 D
	Sub-total	E
5.	Interest During Construction (at 12%)	F
	Total Project Cost	G

Economic Cost Comparison

In the determination of the least cost water supply scheme present worth cost comparison will be utilized. The present worth cost estimates will be based on the following criteria:

Base Year: 1976
Discount Rate: 12%
Service Life of Facilites:
a) Structures and Pipelines: 50 years
b) Mechanical Equipment: 25 years
c) Land: infinite

Total project cost will include construction cost, engineering and contingencies, land cost, administrative and legal fees and interst during construction. Present worth of capital costs will be calculated backward from completion time of construction.

Construction period will be estimated on the basis of similar type of facility construction in the Philippines.

Annual costs will include personnel, power, chemicals, and maintenance costs. These estimates will be carried out for the years 1975, 1990 and 2000. Present worth cost of annual expenditures will be based on gradient series at 12% interest rate.

Cost of any facility to be replaced during design period (1975-2000) will be included in the present worth cost analysis.

No escalation factor will be applied to July 1976 prices as all of the schemes will be affected in the same rate.

Salvage value of a facility will be estimated by using linear depreciation for its value throughout its service life.

Economic comparison of alternative schemes and selection of the least cost scheme will be based on present worth of net disbursements during the period of 1976-2000.

A P P E N D I X B

BASIS OF COST ESTIMATES

APPENDIX B BASIS OF COST ESTIMATES

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APPENDIX B

BASIS OF COST ESTIMATES

General

Cost data presented here refer basically to estimated construction costs, which include all materials and labor together with some allowance for related miscellaneous work and contractor's overhead and profit. The cost data have been converted to unit prices in table or curve form for easy application during feasibility studies. In developing the estimates, data and information from various sources including local engineering consulting firms, materials and equipment manufacturers and suppliers, and construction contractors have been utilized. In some cases, prices and cost estimates from the U.S., modified and adjusted to suit local conditions, were also used. The cost figures have been projected to prices likely to prevail in July 1976.

Construction costs undergo short and/or long-term changes reflecting fluctuation in the local (national) economy and world prices. In the United States, construction cost trends are printed weekly in the Engineering News Record (ENR) and used extensively as a guide for construction cost projections. Based on price movements of structural steel, portland cement, lumber and common labor, and beginning with base of 100 in 1913, this index has risen steadily and had a value of about 2,100 in mid-1974.

Cost analysis includes the development of construction cost indices (CCI) for local and foreign exchange component (FEC) of the cost. Price indices furnished by the Department of Economic Research, Central Bank of the Philippines (CBP) were applied to labor (skilled and unskilled), local materials, contractor's overhead, and profit. The CBP Consumer Price Indices for all items were applied to the labor and profit components of construction work. For local materials, the Retail Price Indices for construction materials were used. For imported mechanical and transportation equipment the ENR cost index was adopted. The resulting projections to July 1976 are shown in Appendix Figures B-1 and B-2.

The unit costs which are developed for this study are for construction costs only. The total project cost would include other items as surveys and engineering, contingencies, land and easement costs, administrative and legal costs, and interest during construction. A typical breakdown of the total project cost is shown in Appendix Table B-1.

APPENDIX TABLE B-1

TOTAL PROJECT COST

Item	Construction Period	Cost in Pesos		
		Local	FEC	Total
1. Source Development	1978-81			
Material and Equipment		...	...	...
Civil and Structural Work		...	...	...
Construction Cost:		...	...	...
15% Contingencies:		...	...	...
Sub-Total		...	...	...
10% Engineering		(35%)	(65%)	...
Sub-Total		...	...	...
Land Costs		...	...	...
Sub-Total		...	...	...
3% Administrative and Legal Fees		...	...	...
Total Project Cost 1/		...	...	...
2. Water Treatment Plant	...			

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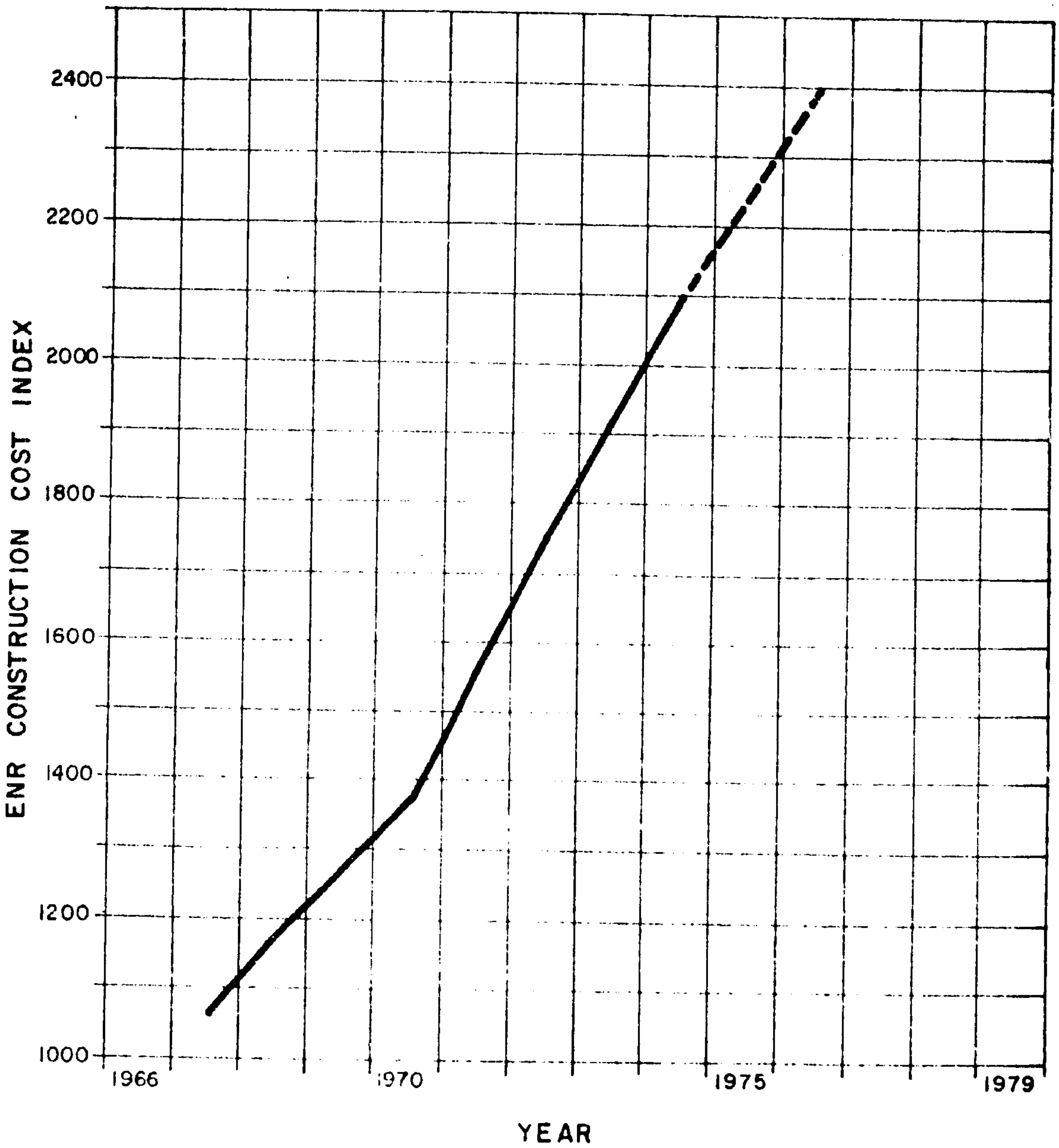
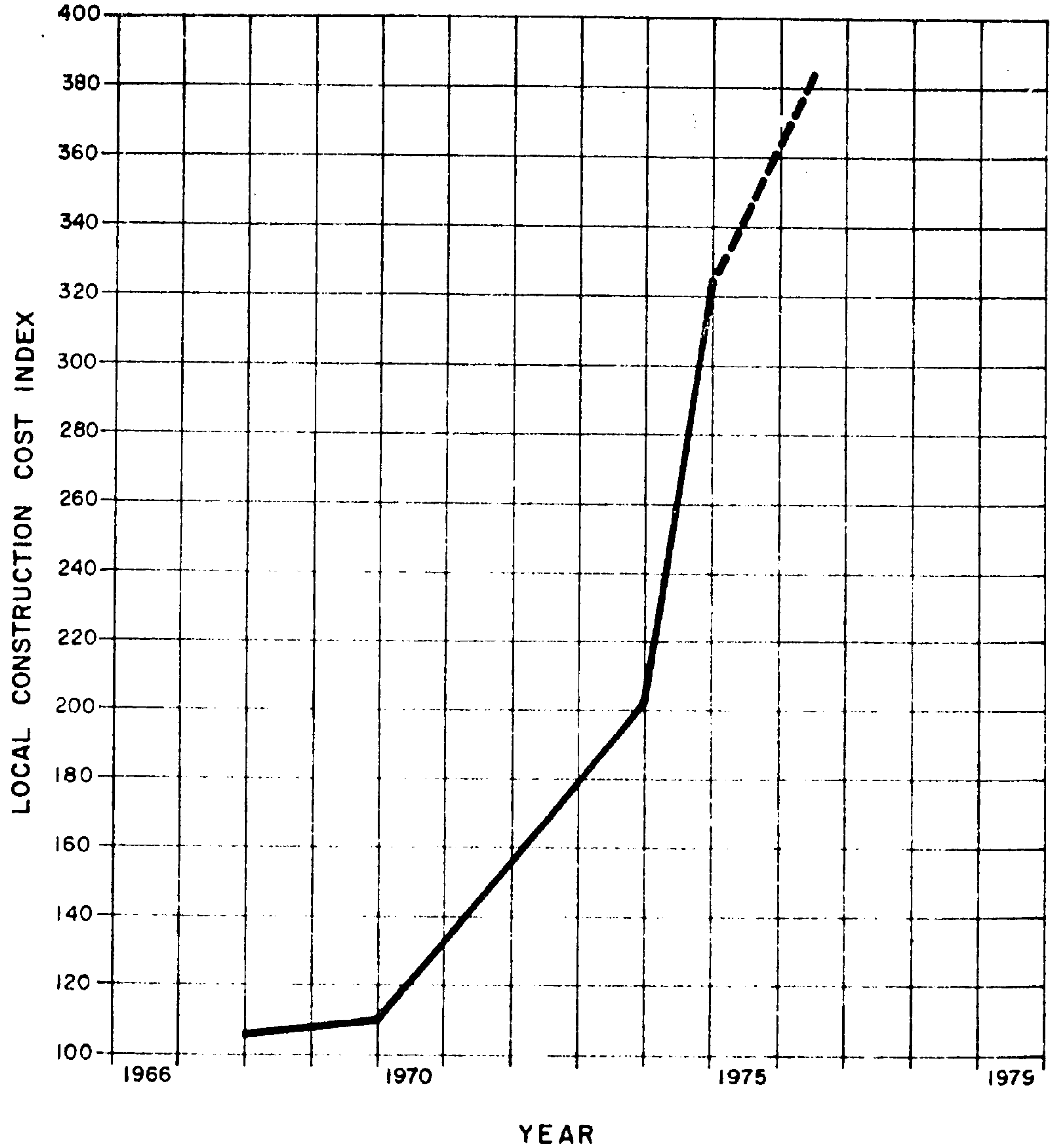
1/ Excluding interest during construction.



NOTE :

BASE YEAR IS 1913, WITH
CONSTRUCTION COST INDEX = 100

APPENDIX FIGURE B-1
ENGINEERING NEWS RECORD CONSTRUCTION COST INDEX



NOTE :

BASE YEAR IS 1965, WITH
CONSTRUCTION COST INDEX = 100

APPENDIX FIGURE B-2
LOCAL CONSTRUCTION COST INDEX

Dams and Appurtenances

Dams and appurtenances are special structures and as such, they must be treated individually in developing estimates for construction costs. Unit costs for items of work that normally enter into the construction of earthfill dams and appurtenances are listed in Appendix Table B-2. Application of the unit costs to estimated quantities for a given dam project will yield estimates of the construction cost for the project or components thereof.

Tunnels

The construction costs of tunnels are heavily dependent on a large number of variables including the types of rock or other material encountered, the physical or structural defects of the rocks, the extent to which water is present in the formations along the route of the proposed tunnel, length of tunnel to be driven, the size and shape of tunnel, the method of attacking the tunnel headings or faces, method of drilling (conventional vs. machine), ventilation and dust control requirements, the mucking operation employed, timbering, steel supports and rock bolts required, design and thickness of concrete lining, the skill and ability of workmen, and on the knowledge and experience of their supervisors. Reasonably accurate construction costs of tunnels are difficult to estimate, more so in the absence of cost data on existing installations. Reliable estimates can be made only after thorough investigation of the tunnel route by borings, geological study and consultation with specialists in tunnel construction. The unit price approach, i.e., cost per unit length of tunnel, to tunnelling cost estimates is risky and can result in substantial errors.

For the purposes of this study, estimating prices developed for tunnels are those for component or appurtenant work for tunnelling rather than for the completed tunnel. The cost figures are presented in Appendix Table B-3. Construction cost for each tunnel project must be estimated individually.

Deep Wells

Cost data for deep well construction are presented in Appendix Figure B-3. The costs are based on actual construction costs, bid prices, and contract prices for deep wells.

The estimating prices include materials and labor costs and are for non-gravel packed wells with perforated casing in lieu of a well screen. Costs of materials are based on the use of imported Schedule 40 black iron pipe casing. Labor costs include mobilization and demobilization charges, drilling, installation of casing,

APPENDIX TABLE B-2

UNIT COSTS FOR DAM AND APPURTENANCES[2]

A. Dam Embankment

Item	Unit	Unit Cost (July 1976) (P)	Remarks
Clearing and grubbing	ha	1,500	Under water add 15%
Common excavation	cum	16	Under water add 15%
Hard pan excavation	cum	20	Under water add 15%
Rock excavation	cum	25	Under water add 15%
Rockfill for embankment			
quarry excavation	cum	65	
Hauling and placement	cum/km	8	
Placement of coarse aggregate	cum	12	
Place of fine aggregate	cum	12	
Impervious earth core			
hauling	cum/km	8	
placement	cum	7	
Backfill			
dump	cum	8	
compacted	cum	60	
Crushed rock (material)	cum	50	
Riprap (placement)	sqm	30	
Steel sheet pile in place	ton	10,000	

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[2] Foreign exchange component of dams and appurtenances is 30 per cent of total construction cost.

APPENDIX TABLE B-2 (Continued)

UNIT COSTS FOR DAMS AND APPURTENANCES

B. Spillway

Excavation	(see previous unit costs)	
Concrete (Plain)	cum	500
Reinforced concrete	cum	900

C. Mobilization and Demobilization : 5% of Total Construction Cost

APPENDIX TABLE B-3

TUNNEL CONSTRUCTION COST[3] ESTIMATES

(July 1976 prices)

Item No.	Work Description	FEC (% of total)	Total Unit Cost (pesos)
A.	Items with Unit Quantities		
1	Open Excavation		
	a) Rock	45	25/cum
	b) Hard pan	45	20/cum
	c) Soil	40	16/cum
2	Tunnel Excavation	50	200/cum
3	Tunnel-Concrete Lining	35	1,000/cum
4	Tunnel-Steel Supports	35	See page B-7
5	Rock Bolts	20	See page B-7
6	Grouting	45	See page B-7
7	Drainage	25	See page B-7
8	Miscellaneous	50	See page B-7

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[3] Does not include engineering and contingencies, land cost, administrative and legal fees.

APPENDIX TABLE B-3 (Continued)

TUNNEL COST ESTIMATES

B. Unit Prices Variable With Tunnel Inside Diameter[4]
(All unit prices in pesos per meter of tunnel)

Item No. (From previous page)	Work Description	Tunnel "D" in meters 2.5	3.0	4.0	5.0	7.0
4	Steel Supports[5]	800	900	1,100	1,300	1,550
5	Rock Bolts[5]	350	400	450	500	550
6	Grouting[5]	400	500	650	800	900
7	Drainage & Ventilation	500	550	600	650	650
8	Miscellaneous	500	600	750	900	1,000

4/ For foreign exchange components see page B-6.
5/ For required length only.

perforating, developing the well, test pumping, well disinfection, and grouting the upper 15 to 30 m of the well.

Deep Well Pump and Pumphouses

Construction cost estimates for deep well pumps and pumphouses are shown in Appendix Figure B-4. The estimates in Appendix Figure B-4 are based on the use of diesel engine driven deep well turbine pumps and include discharge piping and valves, controls, miscellaneous materials, and installation. The pumphouse is assumed to be constructed of masonry or cast-in-place reinforced concrete walls and roof of wooden members and corrugated galvanized iron roofing sheets. Alternatively, cast-in-place reinforced concrete flat slab roof may be employed. Costs do not include the cost of the land and other site improvements.

Water Pump Stations

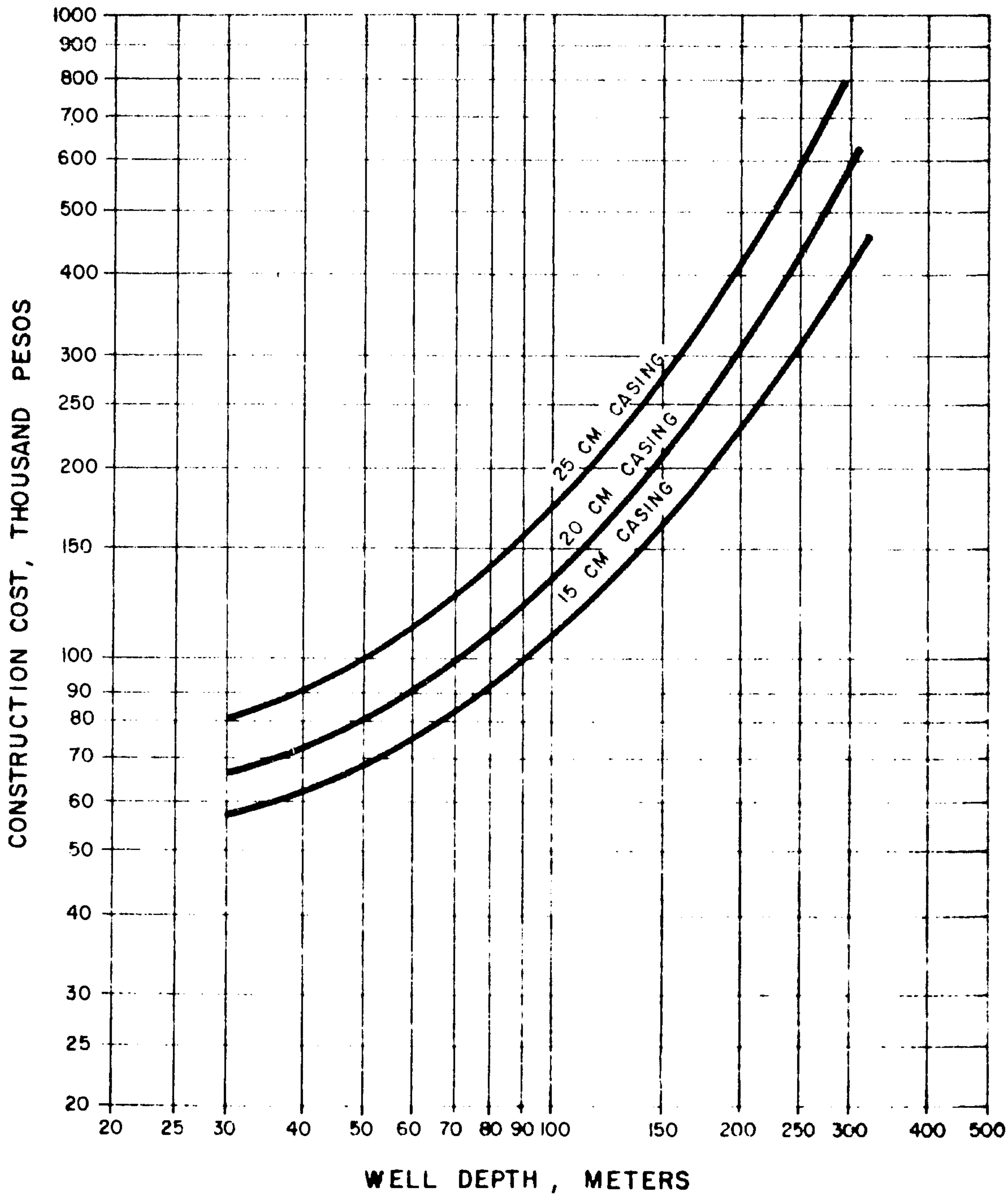
The cost curves which are shown in Appendix Figure B-5 are for a pump station adjacent to a river or lake. The cost of this type of pump station includes an approach channel, intake structure and a pump wet well. A superstructure for housing pump, motors and controls and necessary piping are also included. Cost of land, power transmission and substation, access road must be added to the cost obtained from Appendix Figure B-5.

Water Treatment Plants

Numerous water treatment plants with various capacities have been built in the United States. Therefore, it was possible to develop cost curves for the treatment plants based on plant capacities used in the preliminary cost estimates. However, it was necessary to modify U.S. costs to reflect differing construction costs in the Philippines. The resulting construction costs are shown in Appendix Figure B-6. Costs related to land purchase, access road and power facilities will have to be added to the costs obtained from these curves.

Water Mains

Cost studies have been made on pipe of various materials including cast iron, asbestos cement, steel, ductile iron and prestressed concrete. The unit costs of pipelines are based on the assumption that the least cost pipe, whether locally manufactured or imported, will be utilized. The estimated unit in-place costs based on lower limit of cost envelope, are presented in Appendix Table B-4. The costs include pipe, fittings, jointing materials, excavation, pipe

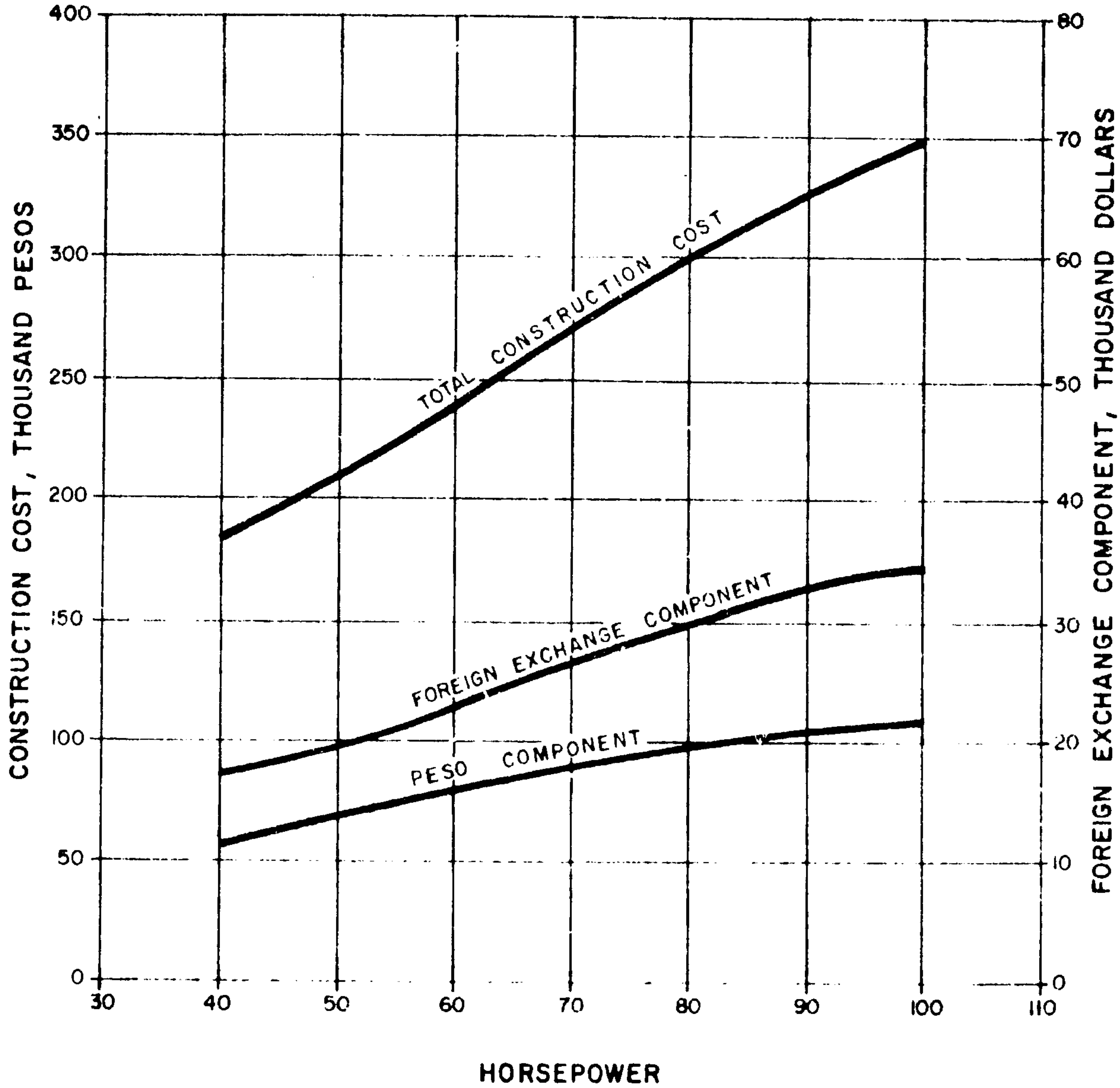


NOTES:

1. COST INCLUDES MOBILIZATION AND DEMOBILIZATION, DRILLING, CASING, SCREEN, DEVELOPING AND GROUTING.
2. FOREIGN EXCHANGE COMPONENT IS ABOUT 25 % OF TOTAL COST.

APPENDIX FIGURE B-3
DEEP WELL CONSTRUCTION COSTS
(JULY 1976 PRICES)

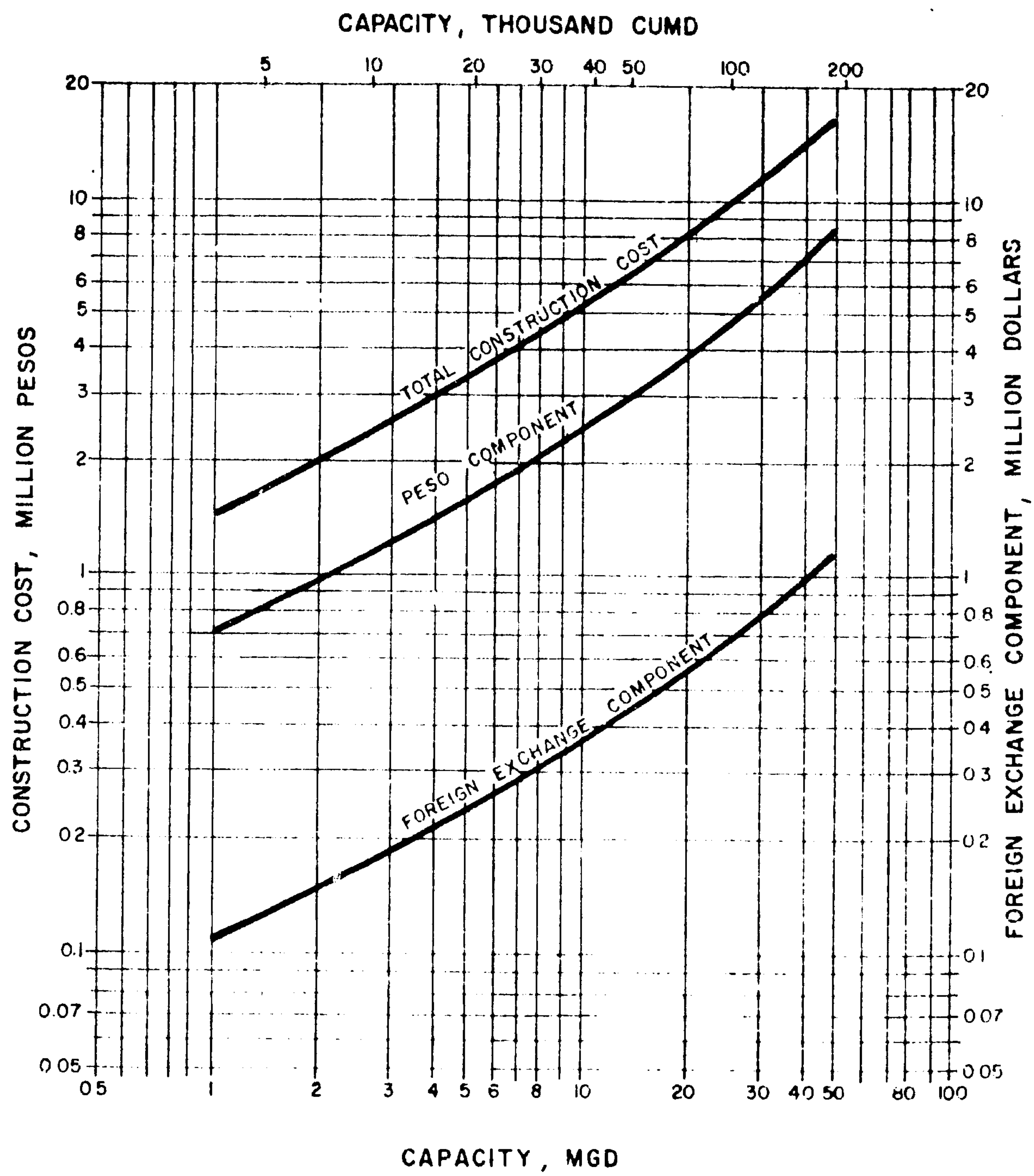
FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM - LWUA



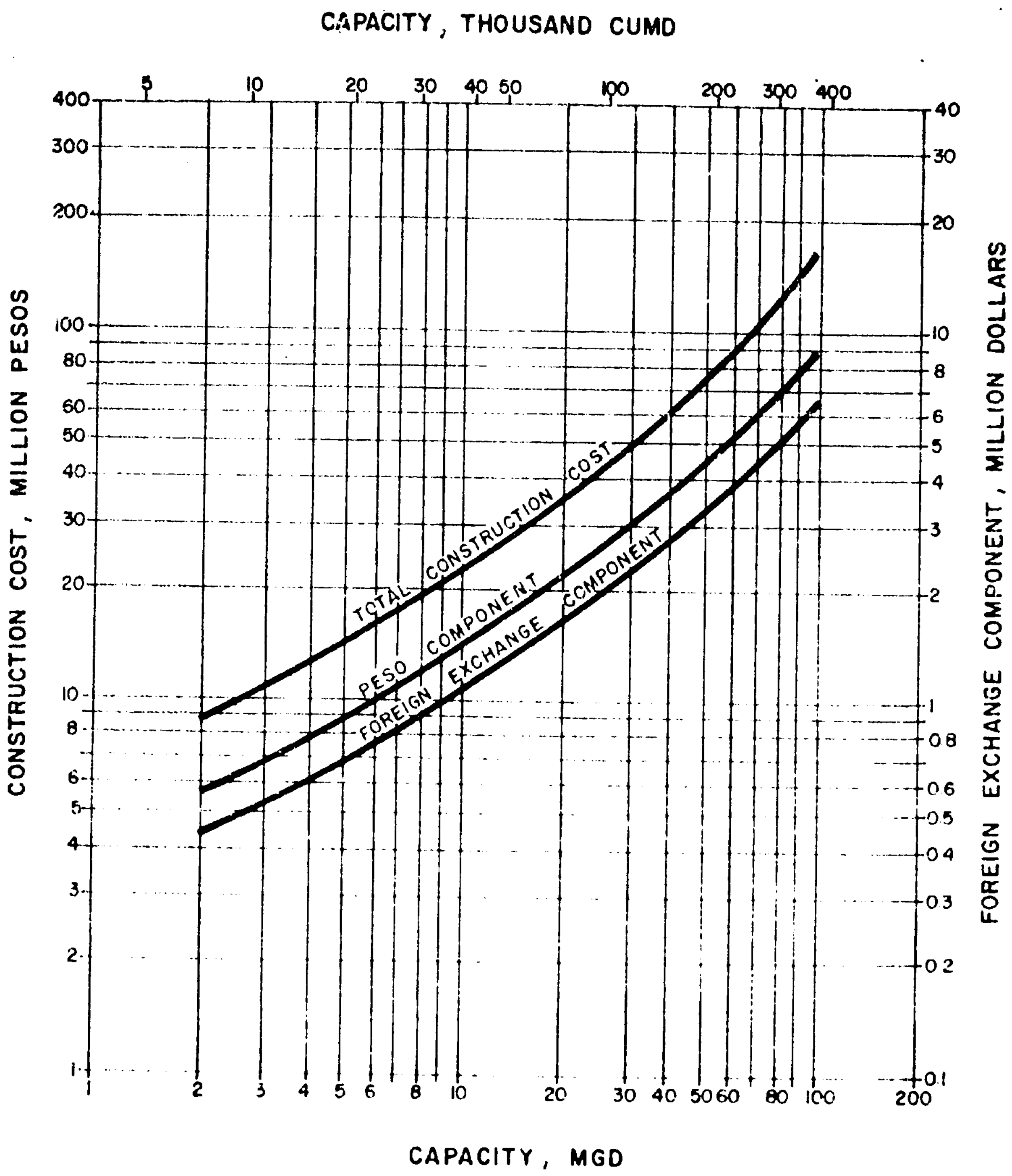
NOTE :

1. COST INCLUDES PUMP AND ENGINE DRIVE, CONTROLS, VALVES, FITTINGS, PUMP HOUSE, AND INSTALLATION.

APPENDIX FIGURE B-4
DEEP WELL PUMP STATION COSTS
(JULY 1976 PRICES)



APPENDIX FIGURE B-5
WATER PUMP STATION CONSTRUCTION COSTS
(JULY 1976 PRICES)



NOTE:

1. COST INCLUDES CHEMICAL MIXING, FLOCCULATION, SETTLING BASINS; RAPID SAND FILTERS, CHLORINATION, SITE WORK; STRUCTURES AND EQUIPMENT.

APPENDIX FIGURE B-6
WATER TREATMENT PLANT CONSTRUCTION COSTS
(JULY 1976 PRICES)

APPENDIX TABLE B-4

PIPELINE COSTS (₱/m)

(July 1976)

Size (mm)	Material	Unit Cost Local	FEC[6]	Total
100	AC, CI	47	33	80*
150	AC, CI	78	72	150
200	AC, CI, DI	96	104	200*
250	AC, CI, DI	148	182	330
300	AC, CI, DI	190	250	440*
350	AC, CI, DI	216	324	540*
400	AC, CI, DI	264	396	660*
450	AC, CI, DI	277	453	730*
500	AC, CI, DI	296	504	800*
600	AC, CI, DI	342	608	950*
700	PSCP, S, DI	448	672	1,120
800	PSCP, S, DI	520	780	1,300
900	PSCP, S, DI	588	882	1,470
1,000	PSCP, S, DI	672	1,008	1,680
1,100	PSCP, S, DI	780	1,170	1,950
1,200	PSCP, S, DI	912	1,368	2,280
1,300	PSCP, S, DI	1,000	1,500	2,500
1,400	PSCP, S	1,160	1,740	2,900
1,500	PSCP, S	1,260	1,890	3,150

*Based on contractor's bid prices for San Pablo and Bacolod City water supply system improvements in November and December 1975.

6/ US $1 = ₱7.00

bedding, backfill, laying and jointing, concrete thrust blocks, pressure and leakage testing, disinfection and flushing, pavement replacement, clean up, transportation, contingencies, and contractor's overhead and profit. Cast iron pipe costs assume AWWA class 150 pipe with inside cement lining, outside tar coating, and bell and spigot lead caulked joints. Costs for asbestos cement pipe are for Class 25, 1SO R160 specifications, with sleeve type coupling joints. Costs for steel pipes are based on pipe with a wall thickness of 0.25 inch, with inside cement-lining and outside double enamel coating.

Booster Pump Station

Cost curves for booster pump station are shown in Appendix Figure B-7. Development of these curves is based on available local information and U.S. costs with some adjustment for the labor component. Booster pump station costs include pumps and motors, necessary controls, piping and a superstructure. Depending on location of the pump station, cost of access road, power transmission line and a substation and land would have to be added to the costs obtained from this curve.

Ground Storage Reservoirs

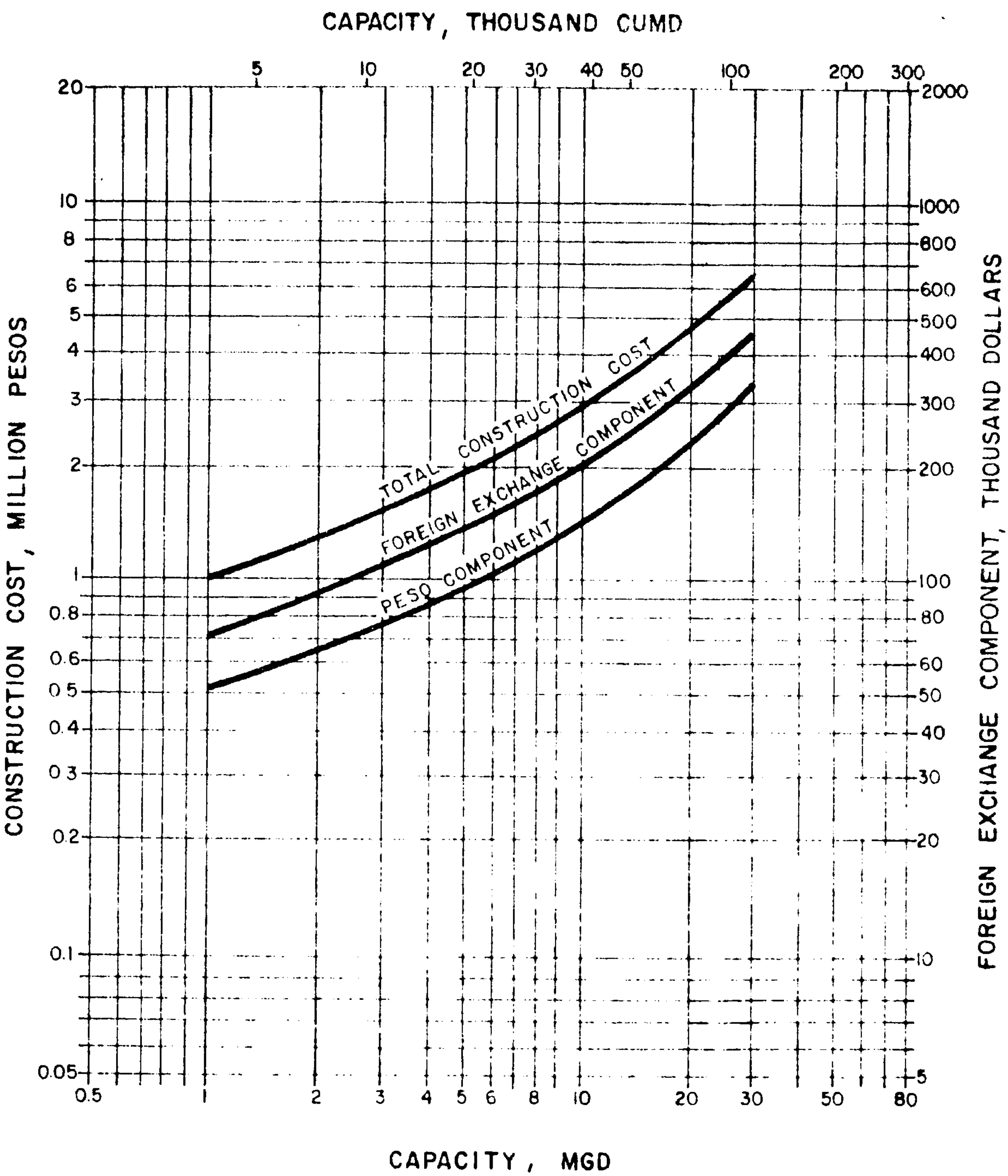
Construction cost estimates of ground storage reservoirs are presented in Appendix Figure B-8 including steel, reinforced concret and prestressed concrete tanks. The costs for steel and reinforced concrete tanks are based on updated costs of actual construction in the past in the Philippines and in other parts of the world.

For tanks constructed of prestressed concrete, the costs were based on prices of similar tanks constructed in the United States adjusted to reflect local prices of materials and labor and on the assumption that local expertise, equipment and facilities for such construction are available. At present, prestressed concrete tanks are not constructed in the Philippines.

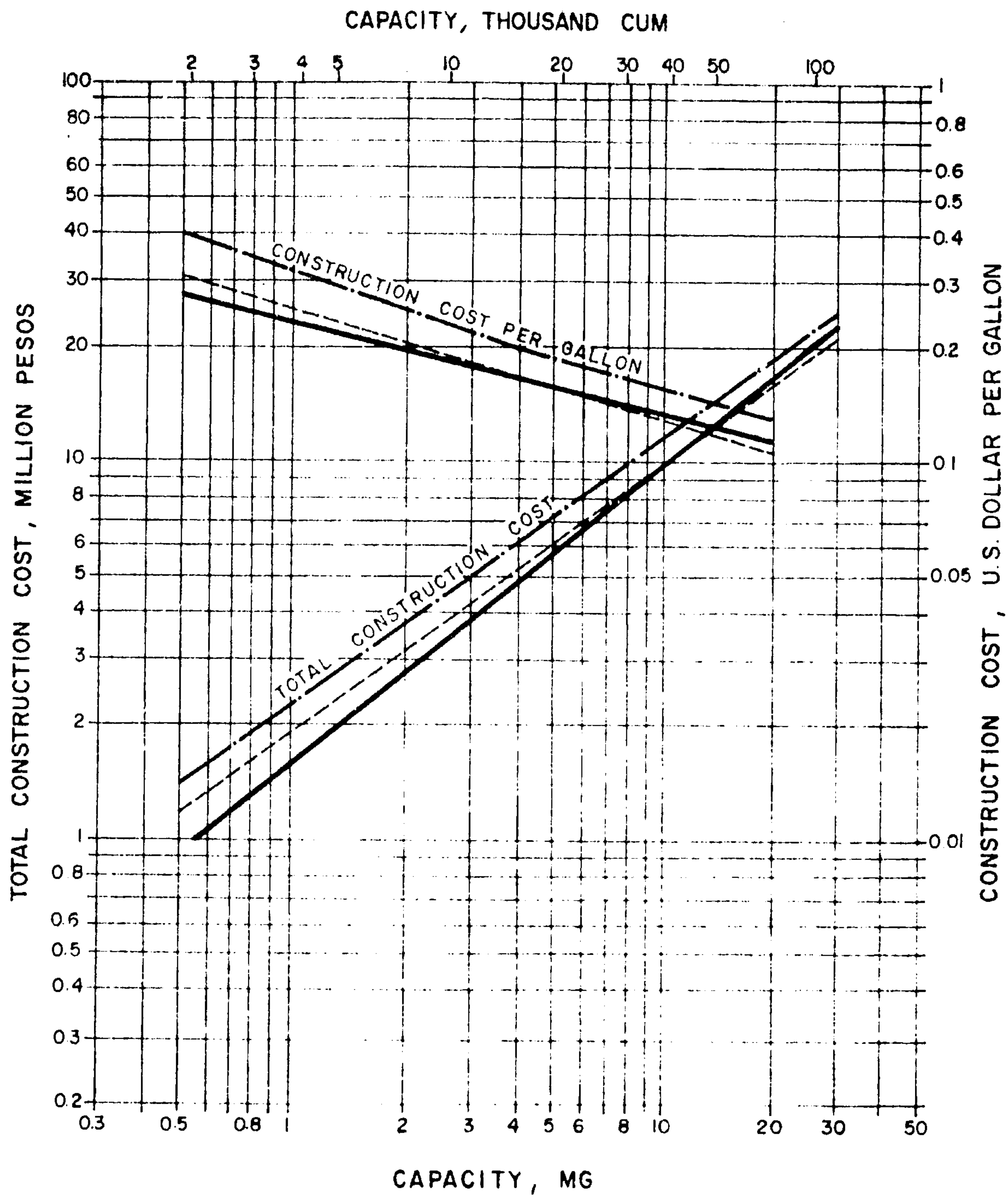
Tank costs include ordinary piping, valving, and tank accessories such as vent, access manhole, ladder rungs, etc. The costs do not include special valves and controls, land taking and access road.

Gate Valves

Gate valves up to 600 mm diameter can be manufactured in the Philippines. Unit costs for gate valves are based on the prices of locally manufactured valves. However, studies indicate that the prices of imported (U.S.) gate valves conforming to AWWA Standard



APPENDIX FIGURE B-7
BOOSTER PUMP STATION CONSTRUCTION COSTS
(JULY 1976 PRICES)



LEGEND:

- STEEL
- PRESTRESSED CONCRETE
- REINFORCED CONCRETE

NOTE :

FEC OF STORAGE TANK COST IS ESTIMATED TO BE ABOUT 20 %.

APPENDIX FIGURE B-8

CONSTRUCTION COSTS FOR COVERED GROUND STORAGE TANKS

(JULY 1976 PRICES)

0500 are only slightly higher than the locally manufactured valves. The in-place estimating prices for gate valves up to 300 mm diameter are shown in Appendix Table B-5. The unit prices include a locally manufactured cast iron valve box and cover.

Butterfly Valves

Current local practice uses butterfly valves instead of gate valves for sizes 350 mm and larger. Butterfly valves are not manufactured in the Philippines and therefore cost data for this type of valve are based on the assumption that these valves will be imported. The unit in-place costs are given in Appendix Table B-5.

Fire Hydrants

The unit in-place costs for fire hydrants assume the use of dry barrel, compression type, traffic model hydrant with 2½-in hose connection and one 4-in pumper connection. The cost figures are shown in Appendix Table B-6 and include fire hydrant, gate valve, tee fitting, jointing materials, concrete thrust blocks, miscellaneous materials, and installation.

Service Connections

Cost data for service connections developed for this work are for two types of service lines. In the first type, the service line consists essentially of flexible polyethylene (PE) plastic pipe without a "gooseneck". The other type consists of a service line made up of GI pipe and employs a PE plastic pipe gooseneck.

The unit in-place estimating prices are for service connections from ½ in to 2 in as shown in Appendix Table B-7. The cost figures are based on the assumption that all materials and components of the service connection would be locally manufactured. The unit costs also assume connection to asbestos cement water distribution mains and include a service clamp in all cases.

Not included in the unit costs are curb stops, curb boxes, and water meters. The in-place prices of these items should be added to the tabulated unit costs should it be desired to include them in the installation and estimating prices.

APPENDIX TABLE B-5

IN-PLACE VALVE COSTS

A. Gate Valves

Size (mm)	In-Place Cost (P) Local	FEC	Total
100	630	770	1,400
150	760	1,140	1,900
200	990	1,610	2,600
250	1,300	2,400	3,700
300	1,580	3,220	4,800
350	3,040	6,460	9,500
400	3,900	9,100	13,000

B. Butterfly Valves

Size (mm)	Local	FEC	Total
300	2,035	3,465	5,500
350	3,370	6,260	9,630
400	4,370	8,870	13,240
450	5,083	11,315	16,398
500	5,890	14,410	20,300
600	6,700	18,100	24,800
700	7,500	22,500	30,000
800	8,800	27,600	36,400
900	9,600	32,400	42,000
1,000	11,200	39,800	51,000
1,100	12,600	47,400	60,000
1,200	14,200	56,800	71,000
1,300	15,200	64,800	80,000
1,400	16,200	73,800	90,000
1,500	17,300	84,700	102,000

APPENDIX TABLE B-6

FIRE HYDRANTS 7/

Size (inlet connection)	In-Place Cost 8/ (Pesos)		
	Local	FEC 9/	Total
100 mm	1,572	2,202	3,774
150 mm	2,304	3,173	5,477

7/ Hydrants are imported.

8/ Costs are for July 1976.

9/ Based on P7 to $1.

APPENDIX TABLE B-7

COST OF SERVICE CONNECTIONS
(July 1976)

Diameter (in)	In-Place Cost 10/ (₱)		
	Local	FEC 11/	Total
½	150	216	366
5/8 - 3/4	160	240	400
1	180	330	510
1½	360	840	1,200
2	450	1,350	1,800

10/ The above estimated costs include all the material and work necessary for a service connection from water main to the meter (5 to 15 m long) with the exception of pavement replacement and the meter.

11/ Foreign exchange component is based on contractor's bid prices for San Pablo and Bacolod City water supply system improvements in November and December, 1975, with an exchange rate of US $1.00 - ₱7.00.

A P P E N D I X   C

CONSTRUCTION MATERIALS AND METHODS

APPENDIX C CONSTRUCTION METHODS AND MATERIALS

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APPENDIX C

CONSTRUCTION MATERIALS AND METHODS

General

The construction of water supply system components such as source of supply facilities, transmission mains, treatment and distribution system works requires a wide range of construction procedures and specific materials designed for each purpose. Construction may vary from the laying of small underground pipelines to the construction of relatively large structures including the construction of buildings. installation of complicated mechanical and electrical equipment, excavation of all types, construction below ground level, pavement removal and replacement, and a host of other types of construction depending on the nature, magnitude and complexity of the waterworks project. This chapter describes certain materials and methods of importance in obtaining the class of construction needed to carry out the intent of preliminary design. Construction must be such that proper and economical operation is assured in order to protect the large investment that must be made to achieve the goal which is to make an adequate supply of safe and potable water available to the people.

Factors Affecting Construction

Factors affecting the facility and cost of construction of water system components include climate and weather conditions, availability of construction materials, availability of skilled and common labor, special construction equipment requirements, existing developments, and soil conditions.

The climate of the area will influence the construction methods to be used and the speed with which work can proceed. For example, protracted periods of intense rainfall will cause interruption and delay in construction work and may require shoring or bracing trenches for water mains to prevent their collapse and trench dewatering facilities. Adverse weather conditions will also affect the logistics of construction as the delivery and transportation of materials may be prevented or delayed.

A significant climatic factor in many parts of the Philippines is the frequency of tropical cyclones. An average of 19 tropical cyclones form in or enter the Philippine area of responsibility annually. Some areas in the Philippines are more susceptible to tropical cyclones than others. Aside from preventing prosecution of the work, tropical cyclones may also inflict significant damage on work already completed or in progress.

Other physical factors that could greatly affect the construction of water system facilities, particularly water distribution main construction procedures, are the width of streets; presence or absence of sidewalks, curbs, and gutters; traffic density; and other existing or proposed underground utilities.

Soil conditions are expected to vary for different areas and from place to place in any given area. Pertinent soil information for the construction of the various components of the water system improvements should be gathered and evaluated in order that any special construction problem or requirement can be properly determined and provided for. For example, soils with high sulfate content may eliminate consideration of asbestos cement pipe for water mains. In cases where transmission and distribution mains are to be laid in unstable soils, across streams, swamps, or marshlands, the soil conditions should be thoroughly checked that they can withstand the load, or the pipeline materials and joints should be selected and designed with provision for any excessive settlement that may occur.

There is a large reservoir of labor and skills in the Philippines to carry out the vast construction work involved in water supply system development or improvement. It may be necessary, however, to bring in to the project area certain technical personnel and specialists to supervise the work and installation, and to instruct in the maintenance and operation of complicated items of machinery and equipment.

In some large Philippine cities, there may be local construction contractors with the competence and resources to undertake all or portion of a waterworks project. In the event that local construction expertise and capability are not available or are deficient in some respects, several Metropolitan Manila - based construction firms can be utilized for any and almost all of the work needed for water supply projects.

Other types of work require the use of specialized equipment not only because it is virtually impossible or extremely difficult to accomplish the work with human power but also for faster, more efficient, more economical, and better quality of work. In general, however, the use of equipment-intensive construction procedures for waterworks improvements in the Philippines should be avoided if possible. Common construction equipment such as trucks, cranes, etc., may be available in some project areas. Government-owned construction equipment for infrastructure projects assigned to highway regional or district offices may be available for use by private contractors on a rental basis.

Existing and proposed developments in a project area would normally create some problems with respect to the construction of water supply facilities. For economy and ease in construction, the implementation of waterworks projects must be planned with due consideration of other utilities and public works construction programs.

Construction Materials and Methods for Waterworks Projects

In any construction work, materials and procedures are two of the most important items needed for the successful prosecution and completion of the project. Many construction materials and procedures are common to several types of construction. Others are more specialized in nature and apply only to certain types of structure or work. In the following sections are discussed some of the materials and procedures that are normally needed and employed in the construction of water supply systems. Information is presented on materials that go into concrete work, various pipe materials and valves, fire hydrants, service lines, pumps, and water meters. With the expanding activities and programs in water supply development in the Philippines, the engineering and construction of large capacity water supply works, such as transmission tunnels, water mains, water treatment plants, pumping stations, and storage reservoirs are expected to increase. Common practices in the construction of these facilities are discussed briefly in this report.

Sand and Gravel

Sand and gravel may be needed in large quantities in a water supply development project area for use as concrete aggregates, pipe bedding, road surfacing, etc. Unavailability of these materials in the amounts needed within reasonable hauling distance to a project area could add materially to the construction costs. In any water supply feasibility study and construction program, investigation should be carried out to locate sources of sand and gravel and determine their suitability for the various works.

Cement

Cement is manufactured in large quantities in the Philippines and in recent years has been one of its export products. As of 1974, there were 18 operating cement plants in the Philippines, 11 located in Luzon, two in the Visayas, and five in Mindanao. The majority of the existing cement plants started original operation or underwent expansion within the last decade. In addition, 24 cement projects were registered with the Securities and Exchange Commission. The operating plants have a total capacity of 173.4 million bags of cement of 43 kg each. Total production in 1974 amounted to about 85 million bags, or about 3.6 million metric tons, of which approximately 20 per cent was exported.

No serious or special problem is likely to arise with respect to cement requirements of any water supply project in the Philippines.

Reinforcing Steel

For reinforced concrete construction, steel reinforcing bars are fabricated by 27 steel mills in the country. Reported production of reinforcing steel of the plants for 1974 amounted to 240,000 metric tons. Steel manufacturing normally conforms to ASTM standards. Reinforcing bars in sizes from 6 to 25 mm are readily available. For the larger sizes, bars are available in plain and deformed sections.

Concrete

From the foregoing, it can be concluded that all the principal materials needed for good quality concrete can be furnished from local (Philippine) sources. The quality of concrete needed for the various components of the development plan will have to be determined during the final design stage of the project.

Asbestos Cement Pipe

Asbestos cement pipe was first made in Europe in 1913, and was introduced in the United States in 1929. However, its extensive use for water system piping in the Philippines started only in the early sixties.

Asbestos cement pipe is manufactured from simple ingredients: asbestos fiber, silica sand, and cement. Asbestos fibers make up the smallest percentage of the total volume of pipe material ingredients but their high tensile properties add significantly to the overall pipe strength. The amount of each element used varies but is usually in the following ranges: asbestos, 15 to 20 per cent, silica, 32 to 34 per cent, and cement, 48 to 51 per cent. By virtue of its methods of manufacture, asbestos cement pipe is smooth on the outside, and due to the polished mandrel used in its formation, it normally has a very smooth interior bore. Therefore no coatings of any kind are used. Because of its chemical composition, asbestos cement pipe is not easily affected by corrosive waters; however, it requires a special outside coating for soils with high sulfate content. With its smooth bore, it has a high "C" value at installation that can be expected to remain high throughout use. The low content of uncombined calcium hydroxide ensures that the leaching effects of soft waters will be at a minimum. Purchasers may specify a limit for uncombined calcium hydroxide. Disadvantages of this pipe include low strength, brittleness, disintegration, leakage, and low ductility.

Asbestos cement pipe which has been used for over a decade for water mains in the Philippines is widely accepted in this country and often has been the pipe material of choice for small sizes (80 mm to 300 mm) primarily because of its relative economy compared to ferrous pipes. The pipe is produced by two manufacturers with factories in Metropolitan Manila, and under the trade name Eternit and Italit, respectively. Pressure pipe is readily available in sizes from 80 mm to 600 mm for rated working pressures up to 130 mm. Pipes are generally manufactured according to ISO R-160 specifications and supplied in 4-meter lengths. A significant feature of asbestos cement pipe manufactured under the ISO specifications is that the required test pressure is only twice the rated working pressure.

Inquiries as to whether asbestos pipe conforming to AWWA standard C-400 can be manufactured by the local plants revealed that the pipes can be manufactured but at higher costs than ISO pipes because of the stringent requirements of the AWWA standard. For example, the AWWA standard requires a hydrostatic test pressure of 3½ times the rated working pressure.

The AWWA standard covers two types of asbestos cement pipe: Type I - for use where contact with aggressive waters and/or soils with sulfate content is not expected, and Type II - for use where contact with aggressive waters and/or soils with sulfate content is expected to occur. The standard limits the uncombined calcium hydroxide (free lime) for Type II pipe to one per cent. To meet this requirement, the local manufacturers indicated that the cement to be used might have to be imported if locally produced cement would not prove suitable. For Type I pipe, there is no prescribed limit for uncombined calcium hydroxide.

Locally produced asbestos cement pipes are normally joined with a coupling of the same composition and strength as the pipe and joints are sealed with double "O" rubber rings. Mechanical joints (Gibault joints) for joining asbestos cement pipes, or asbestos cement-to-cast iron pipe are also produced locally.

In recent years a question has been raised with respect to the possible health hazard that may be associated with drinking water which has flowed through asbestos cement pipe. In an effort to determine the scope of the problem, the A/C Pipe Producers Association (U.S.) contracted with the American Water Works Association Research Foundation to study the problem of asbestos in water, specifically with relation to the use of asbestos cement pipe. One conclusion of the recently completed study is that though asbestos in water has become a potentially serious health hazard the proper use of asbestos cement pipe for water does not pose a hazard to health by reason of ingestion of asbestos fibers. Highlights of the other findings and conclusions of the study are:

(1) Asbestos can cause granulomatous and fibrotic reactions in the lungs but there is no evidence that it does so in the gastro-intestinal tract.

(2) The general prevalence of asbestos in soil results in its presence in most waters of lake, river, and well origin, and in distribution systems whether fabricated of asbestos cement or other materials.

(3) Asbestos cement pipe systems have serviced large populations for 40 or more years in Europe and the United States with no apparent increase in peritoneal mesotheliomas among the public during this period despite the fact this tumor has been the focus of great interest among the pathologist for the past 10 years.

(4) No firm evidence shows that the proper use of asbestos cement pipe poses a hazard to health by reason of ingestion of asbestos fibers. Calculations comparing the probable ingestion exposure in occupational groups to that likely to [illegible] as a result of ingestion of potable water from asbestos cement pipe systems suggest that the probability of risk to health from the use of such systems is small approaching zero.

Based on the above, it is safe to assume that asbestos cement pipe is still an acceptable material for conveying and distributing public water supplies.

Cast Iron and Ductile Iron Pipe

General. There are two types of cast iron available for water systems: gray cast iron and ductile iron. Gray cast iron has a history of use that dates back more than 300 years. Ductile iron was developed in 1948, and its use has been increasing since 1960.

Gray Cast Iron. Gray cast iron has characteristics of long life, toughness, imperviousness, and ease of tapping, that are provided by the chemical composition of the metal. Carrying capacity is ensured by proper lining.

The production of gray cast iron pipe consists of melting the metal in a furnace (cupola), the addition of such other materials as needed for the final desired composition, and the actual casting, usually by a centrifugal process. As a molten iron is withdrawn from the cupola to a ladle, small amounts of graphite and ferrosilicon are added to adjust the carbon and silicon content; this is termed inoculation. The amounts of carbon, silicon, manganese, etc., although small, materially affect the structure of the iron. Each of the chemicals added is controlled in amounts to produce the desired qualities in the castings.

In gray cast iron, the major part of the carbon content occurs as free carbon or graphite in the form of flakes interspersed throughout the metal. An appreciable volume of graphite flakes makes gray cast iron more resistant to corrosion than the purer forms of iron because graphite does not corrode. Graphite in cast iron also affects the machinability of the pipe, that is, it makes the pipe more easily tapped and threaded for insertion of a corporation cock.

Cast iron pipe has been used for water supply systems in the Philippines for more than half a century. Prior to the introduction of asbestos cement pipe, cast iron dominated the market for water supply piping. Until locally manufactured cast iron pipe became available in the 1950's, all cast iron pipes used were imported.

At present, centrifugally cast iron pipe is manufactured by the Filipino Pipe and Foundry Corporation and marketed under the trade name "Silva Pipes". This company's plant is located in Mandaluyong, Rizal and has an annual capacity of about 33,000 metric tons. Pipes are centrifugally cast in metal molds and are available in sizes from 150 to 600 mm unlined or cement lined. The pipe is manufactured with bell and spigot ends for lead caulked joints. Bell and spigot iron pipes are made in conformance with (U.S.) Federal Specifications or AWWA Standards. The Silva plant also manufactures cast iron fittings, and Gibault joints for asbestos cement pipe to cast iron pipe.

Ductile Iron Pipe. Ductile iron pipe is stronger, tougher, and more ductile than gray cast iron. Its characteristics are due to the configuration of the free carbon or graphite in the iron. Ductile iron is defined as cast iron with graphite in spheroidal (nodular) form. It is produced by adding an inoculant, usually magnesium, to molten iron.

Ductile iron is chemically akin to gray cast iron of low phosphorous and low sulfur content, the latter obtained by desulfurizing in the cupola. Magnesium can be added, after the removal of sulfur, in a post-inoculation treatment, with a silicon-base magnesium alloy.

Ductile iron pipe is centrifugally cast in the same manner as gray cast iron, but the melting and inoculation phase of the process is more complex; the casting phase is the same. At present, this type of pipe is not manufactured in the Philippines.

Steel Pipe

Early use of steel pipe for carrying water was in large, long, and exposed transmission lines in relatively dry areas where corrosion was not a problem. Other applications in other areas, became more common as coal-tar coatings became available. Steel pipe is used in the Philippines in many distribution and transmission lines as well as in inplant systems. The American Water Works Association (AWWA) has prescribed standards for steel pipe for use in water systems. The Local Water Utilities Administration (LWUA) of the Philippines has adopted (U.S.) Federal Specifications SS-P-385a dated January 31, 1964 and Amendment I dated February 27, 1968, with some modifications thereof, as its standards for steel pipe and specials.

As described by AWWA Standards, there are two types of steel water pipe: fabricated, electrically welded steel pipe and mill-type steel pipe. Both types may be coated and lined.

Fabricated electrically welded pipe may be produced by automatic welding machines or by manual operations. AWWA Standard C201 gives detailed specifications for this type of pipe. Mill-type steel pipe may be furnace welded (continuous butt-welded or furnace butt-welded), electrically welded, or seamless. AWWA Standard C202 sets forth the specifications for mill-type steel pipes. An AWWA committee has been working to combine the above two standards into a single standard.

Large and small diameter steel pipes are manufactured in the Philippines. The International Pipe Industries Corporation with plant in Pasig, Rizal manufactures spiral welded pipe from 100 to 1,200 mm diameter. As of January 1975, this plant had a capacity of 15,000 metric tones per year but was undegoing expansion to double its present capacity. Pipe can be manufactured and cement lined according to AWWA Standards C202 and C205, respectively.

Five other plants produce small size pipe from 10 to 200 mm diameter. Both black and galvanized iron pipe can be produced according to ISO or ASTM Standards. In 1974, the total production of these five plants amounted to 31,600 metric tons.

Prestressed Concrete Pressure Pipe

There are four usual types of concrete water pipe, classified according to the method of reinforcement. These types are: cylinder, not prestressed; steel cylinder, prestressed; non-cylinder, not prestressed; non-cylinder, prestressed.

AWWA has set forth design requirements for the first three types of pipe including minimum wall and lining thickness, reinforcing spacing, and core coat thickness specifications.

The steel cylinder, not prestressed concrete pipe is covered under AWWA Standard C300.

The prestressed concrete embedded cylinder pipe consists of a water tight steel cylinder, steel joint rings, a concrete core, high tensile wire reinforcing and a cement-mortar or concrete coating. Ranging in diameter from 16 to 144 inches, it is considered highly suitable for major water supply and transmission lines. This type of pipe is also recommended for unusually high pressure distribution lines. AWWA Standard C301 covers this type of pipe.

The non-cylinder, not prestressed reinforced concrete pipe is normally produced in diameters from 600 to 3,500 mm. It is a vertically cast pipe with dense concrete walls reinforced by one or more steel cages. AWWA Standard covers this type of pipe.

The fourth type of concrete pipe (prestressed, non-cylinder type) is not covered by AWWA Standards. This pipe consists of a concrete core manufactured by centrifugation, both longitudinally and circumferentially prestressed by high tensile wire, and protected by a dense coating of premixed cement-mortar.

Although prestressed concrete pipe is not yet manufactured and used in the Philippines, it is recommended that this type of pipe, where it is applicable, be considered in the final design of facilities. Unofficial information has revealed that two Philippine companies are planning to put up factories to manufacture prestressed concrete pressure pipes.

Plastic Pipe

Plastic pipe as a commercial product was first intoduced in Germany in 1930 and in the United States in 1940. Polyvinyl chloride (PVC) was the first type produced. Later came cellulose acetate butyrate (CAB) and polyvinyladine chloride (Saran). Volume production of plastic pipe began in 1948, when polyethylene (PE) was accepted for various water uses.

Early production of plastic pipe was in sizes below 50 mm, and most of the plastic pipe sold was for service lines and household plumbing systems. As developments in the plastics industry progressed, larger pipe sizes became available, and plastic pipe is today used for water distribution mains in many localities throughout the world, as well as for services and in plant piping systems.

There are about a dozen plastic materials that are, have been, or may be used in water systems. Only three, however, are in common use: PVC, PE, and ABS (Acrylonitrile Butadiene Strynene). ABS has been used primarily for drainage, waste, and vent (DWV) pipe and fittings for interior application. ABS has been popular a few years ago for water systems, but because it has only half the available hoop stress of PVC when subjected to internal pressure, the latter product is considered to be a better material for water lines.

Available U.S. standards for the manufacture of plastic pipe for use in water systems include ASTM, Department of Commerce, Commercial Standards, and USASI Standards.

PVC and PE pipes for use in water systems are manufactured in the Philippines. A PVC plant in Iligan City supplies most of the raw materials for PVC pipe to the local manufacturers. PVC pipe is available in sizes from 10 to 300 mm in 3 to 6 m lengths and standard thermo-plastic pipe dimension ratio (SDR) from 9 to 32.5. The SDR is the ratio of pipe diameter to wall thickness. In the case of ABS and PVC pipe, the outside diameter is used; for PE, the inside pipe diameter is used. The SDR and hydrostatic design stress of the pipe affects its pressure rating which is defined as the estimated maximum operating internal pressure at which the pipe can function without failure.

Classes of PE pipe available include Medium Density, Schedule 40; and High Density, Schedule 40, 80 and 120. Pipe sizes are from 10 to 40 mm, 60 m rolls for sizes 40 to 60 mm, and 25 to 30 m length for pipe 75 to 300 mm in diameter.

To date, plastic pipe has not been used extensively in the Philippines for water mains. Limited experience with PVC pipe water mains used in a high pressure distribution system has not been satisfactory because of frequent failures and leakage particularly at the joints. One problem that has discouraged or deterred some engineers to specify PVC pipe is the non-standardisation of fittings and connections among local plastic pipe manufacturers.

Thus, a project becomes a captive market for a particular brand of pipe once the project starts to use the particular brand.

Plastic pipe materials acceptable to LWUA are PVC, PE, and PB (Polybutylene) and tentative standards therefore have been adopted by this organization. PB pipe, however, is not currently manufactured in the Philippines.

Valves and Fire Hydrants

Gate Valves. Gate valves for water systems are normally of the double-disc type, with parallel bronze-mounted seats, cast iron body, gate rings, wedges, and a non-rising stem with or without handwheel, or outside screw and yoke (OS & Y) type. Valves used for small lines (100 mm to 300 mm) in distribution systems are frequently furnished with an operating nut and installed with a valve box extending to the ground surface, providing accessibility to the operating nut. For valves, 400 mm or larger, which are in general power operated, vaults with manhole access are generally provided to facilitate operation and maintenance. Also valves larger than 400 mm are often equipped with smaller by-pass valves, to reduce the pressure differentials and the power required during opening and closing operations. Gate valves for water service are covered by AWWA Standard C500. At present, most of the gate valves used in the Philippines are imported mostly from the U.S. or Japan. Valves up to 300 mm diameter conforming to AWWA requirements, however, can be manufactured in the Philippines.

Butterfly Valves. In recent years, butterfly valves have been increasingly used for water systems. Advantages of this type of valve are: driptight shut off, little maintenance, low head loss, small space requirement, reliability, and generally less expensive than gate valves, particularly of the larger sizes. The AWWA has two standards for butterfly valves: AWWA Standard C504 which covers rubber-seated valves from 100 to 1,800 mm diameter for pressures up to 10 kg/cm^2, and AWWA Standard C505 which covers metal seated valves from 100 to 1,800 mm diameter for pressures up to 15 kg/cm^2.

Butterfly valves are not currently manufactured in the Philippines.

Air Valves. Air valves should be installed at high points in transmission lines, to permit the escape of air when the pipeline is being filled and to admit air when the pipe line is being emptied for maintenance or repair. It is usual to install air valves of the automatic type which open to release air accumulating during normal operation of the pipeline.

Blow-off Valves. Blow-off valves are generally installed at low points of transmission pipe lines and at low points and dead-ends in distribution systems to provide an outlet for removing sediments that may accumulate in those places. Ordinary gate valves may be used for this purpose, with provision for conveying the water and sediments "blow-off" to a suitable point of discharge.

Miscellaneous Valves. Miscellaneous and special valves for water systems include check valves to permit only one direction flow of the water, surge relief valves for surge and water hammer protection, altitude valves for controlling water levels in reservoirs and/or pump operations, and pressure reducing or regulating valves for dissipating excess pressures. None of these valves is manufactured in the Philippines.

Fire Hydrants. Fire hydrants that are in common use in Philippine towns and cities are of two types. One is a wet barrel type consisting of a 60 mm or 75 m riser pipe, usually GI pipe; a 60 or 75 mm GI tee or 90° elbow; and a 60 mm fire hose valve. A shut-off valve is generally installed between the hydrant and the water main to which it is connected. This type can be fabricated and assembled in the field, or in the shop ready for installation and connection to the water main.

The other type of hydrant is similar to that commonly used in European and North American communities. This hydrant is a dry barrel type, with compression type main valve, 100 mm or 150 mm inlet connection, and one or two 60 mm hose outlets and one 115 mm pumper connection.

The first type of hydrant has a disadvantage in that unless sufficient pressure in the main for the fire flow can be provided, it will not be effective for fire-fighting. For this reason the second type of hydrant appears to be advantageous.

Water Service Lines

Water services or service connections are pipes of usually small diameter that run from distribution mains or branch mains to customer premises. The water service connection is usually attached to the street main by means of a corporation stop which may be inserted while the main is in service and under pressure. Where the service connections are expected to be larger than 50 mm in diameter, tees, wyes or special branches are installed, along with the water main construction. Ordinarily, water service to the customer's premises is turned on or off at a curb stop, accessible through a curb box. Various pipe materials have been and can be used for the service lines. Non-flexible materials require a flexible "gooseneck" connection to the corporation cock. Gooseneck connections may be lead, copper, or flexible plastic.

At present, galvanized iron pipe is used in the Philippines for most water service connections. Galvanized iron pipe has a relatively short life because of its susceptibility to the corrosive action of soil on the outside and the water inside the pipe. The use of plastic pipe material for service connections may reduce this corrosion problem to a minimum.

Water Meters

Any modern water supply system should be equipped with the proper type of water meters so that the water produced and delivered can be accurately measured. Key locations in the system, at supply sources, treatment plants and pump stations should be provided with venturi tubes, orifice plates or other types of metering devices. Because such metering devices are not currently manufactured in the Philippines, these items will have to be imported.

Every service connection to a distribution system should be equipped with a meter to reduce wastage and to obtain the proper billing. Small-size turbine type water meters are manufactured by the Liberty Manufacturing Corporation in the Philippines. Another local company, Domingo S. Jose, Inc., is in the process of putting up a factory to manufacture various sizes of meters under the trade name "KIMMON" under license by the Kimmon Manufacturing Company Ltd., a Japanese firm. Kimmon water meters of the turbine or rotary piston type are available in small sizes 10 to 50 mm. Propeller type meters up to 400 mm are also manufactured by Kimmon plants in Japan.

In recent years, locally manufactured meters have been the most commonly used meters for service connections. Limited information indicates that these meters can be expected to function satisfactorily for only about one year after installation and have poor registration capability. Improvements in the characteristics and performance of these meters are obviously desirable.

Construction Methods For Water System Components

In the preceding sections, common construction materials for waterworks have been briefly discussed. The remainder of this report will be devoted to a general description of construction practices for deep wells, tunnels, water treatment plant, water mains, pumping stations, and storage reservoirs.

Deep Wells

Water wells have long been used in the Philippines as sources of public and private water supplies and for small and large quantities of water. Wells that have been used for piped public water systems are generally of the drilled well type and capable of supplying several tons or hundred of gallons of water per minute. At present there are about half a dozen competent and experienced deep well drilling contractors in the Philippines. Present practice of deep well construction in the Philippines is normally by the percussion (or cable tool) or rotary method. Specifications usually call for the contractor to submit a well log. In unconsolidated formations, the well is usually cased with imported Schedule 40 black iron pipe. A telescoping casing employing two pipe sizes is commonly installed. As a rule, no well screen is used principally because of its high cost. Openings from the aquifer(s) to the well are provided by perforations in the casing. The perforations can be made in the field. Gravel packing around well screens or perforations is very rarely practiced.

After the installation of the well casing, the well is developed. Local well drilling contractors employ development methods such as pumping, surging and bailing, and development wit compressed air.

Test pumping follows well development. The purpose of test pumping is to provide information of the yield and capacity of the well, which in turn helps in determining the capacity of pumping equipment. Water level measurements are taken during pumping (drawdown measurements) as well as after the pumping test is completed (well recovery). Common practice is to specify a 24-hour or 48-hour pumping test.

Water Main Construction Procedures

Water mains are generally installed to a definite alignment and grade. In the Philippines where freezing is not a problem, the depth of cover over the pipe specified usually depends on the surface load conditions. The minimum cover for the alignments which are subject to traffic loads is 90 cm. For the areas with no traffic loads a minimum cover of 60 cm may be used. Trenches may be dug manually or with excavation machinery.

Trenches are excavated as shallow as possible but still provide enough depth for surface loading. Deep trenches are avoided since they usually require shoring and bracing and, therefore, are costly.

Trench Widths. Sufficient trench widths are provided to permit installation of the pipe, with room for the workmen to make up the joints and to tamp backfill under and around the pipe. Trench widths are governed by type of soil, pipe size, and excavating equipment. For asbestos cement and concrete pipes, unnecessarily wide trenches are avoided to minimize excessive backfill loads on the pipe. For asbestos cement pipe, the following widths are used:

Pipe Diameter (mm)	Trench Width (cm) Minimum	Maximum
100	45	70
150 or 200	50	80
250 or 300	60	90
350 or 400	75	100

For cast iron pipe 100 to 450 mm in diameter, the trench width is the diameter plus 40 cm; for the larger pipe up to 1,500 mm in diameter, the width is the pipe diameter plus 45 cm.

Wide trenches for small diameter pipe are avoided, particularly in hard clay soils. Otherwise, the weight of backfill becomes out of proportion to the beam strength of a small pipe.

Where pipe is to be laid on a curve, it utilizes the available deflection characteristics of the joint. Many joints have an inherent ability to be deflected to some small degree, permitting pipe to be laid on a long-radius curve. For pipe laid on a curve, the trench width is somewhat wider than normal.

Excavation. Whether excavation is done manually or by machine, the excavated material is piled on one side of the trench at a distance away from the trench. The distance should be sufficient to prevent excavated material from rolling back into the trench and also to provide room for walking along the trench. In congested areas, it is usually necessary to haul and stockpile the excavated material temporarily at some other location and excavated material suitable for backfill is transported back during backfill operations. Material unsuitable for backfill is disposed off the site.

Sheeting and Bracing. The need for sheeting and bracing to protect against cave-in depends on soil conditions and trench depths. They are installed where required not only to prevent delay in pipe laying but also to protect the workmen and the public.

Pipe Bedding. All types of pipe are bedded or supported properly at the trench bottom. Pipe is laid directly on the trench bottom if the bottom has been levelled properly. For greater load bearing ability by the pipe, the trench bottom is shaped to match the exterior circumference of the pipe. Care is taken to prevent voids or high spots under the pipe. High spots are shaved off, and voids filled with well tamped soil. For trenches in rock, unsuitable soil, or soft or wet soil, special bedding is provided. This is specially important for AC and CI pipes because of their lower tensile strength and brittleness.

For formations of rock or unsuitable soils, the trench is excavated to a depth of about 15 cm below the grade line of the pipe bottom, and the overexcavated material replaced with sand or good soil free of clods, levelled and tamped to grade.

Joint Holes. Provision is made in the trench to permit proper jointing of the pipe with the type of joint employed. For asbestos-cement pipe laid directly on the trench bottom, a coupling hole about 8 cm deep and 15 cm longer than the coupling is dug at the joint location. For cast iron pipe joints of the bell and spigot type which are made with lead caulking, the trench must be excavated wider and deeper at the joint location sufficient to provide room for the caulker to work.

Stringing, Laying and Jointing Pipe. To avoid unnecessarily handling, pipes and fittings and other accessories are placed as near as possible to their final location in the line, with due regard to safety requirements. Pipes are placed as close to the trench line as possible and on the side opposite where the excavated material is to be piled. Asbestos cement pipe is usually not strung in advance of laying and jointing operations but is delivered from storage to trench as needed.

The procedure for laying pipe and making up pipe joints varies with the type of pipe material and type of joint. For asbestos cement pipe, general procedures are given in AWWA Standard C603 which are followed in the Philippines. The laying and jointing of cast iron and steel pipes conform with applicable portions of AWWA Standard C600, C603 and C206, Federal Specifications and in accordance with the recommendations and directions of the pipe manufacturers. As part of the final design, detailed specifications are included in the jointing procedures for all types of pipe to be installed. Furthermore, to have trouble-free service from a pipeline the resident inspector insists on strict compliance with the specifications and construction drawings.

Leakage and Pressure Tests. All pipelines are subjected to leakage and hydrostatic tests. Such tests are usually done after the trench has been partially backfilled. Test procedures and requirements, allowable leakage, etc., vary with the type of pipe and joint. Procedures and requirements for asbestos cement pipelines and cast iron pipelines are specified in relevant AWWA standards.

Backfilling. Backfilling is an important part of proper pipeline installation and is given considerable attention. Backfilling is usually a two-step procedure consisting of partial backfilling before leakage tests and completing the backfill after the tests. Select backfill material is placed at both sides of the trench uniformly for the full trench width up to the horizontal centerline of the pipe. The backfill material usually is tamped by hand under and on each side of the pipes to provide a void-free support.

Where visual inspection during leakage tests is not required, backfill is placed to the depth indicated above and then a cushion of backfill material, hand-placed and tamped, is added to cover the pipe to a depth of 30 cm.

Where visual inspection is required, joints are left exposed or covered only by a relatively shallow layer. After leakage tests are completed, the exposed joints or couplings are covered with hand placed material to a depth of 30 cm.

The remainder of the backfill material is deposited in the trench by hand or machine in layers and tamped. This backfill should be good soil free from rocks, debris, clods and other unsuitable materials.

Disinfection. All newly installed or repaired water mains are cleaned and disinfected before they are accepted and placed into service. The main is first flushed clean of foreign matter at a scouring velocity of at least 0.75 m/sec. The flushing may be done after the pressure tests.

Suggested disinfection procedures are as described in AWWA Standard C601. The usual disinfectants are chlorine, calcium hypochlorite or sodium hypochlorite solution or chlorinated lime solution. The disinfecting solution is applied at one extremity of a pipe section and drains at the opposite extremity of a properly segregated section. The rate of application gives a uniform dose of at least 25 mg/l at the end of the section being treated. The average contact period is 24 hours and should produce not less than 10 mg/l at the end of the line after the contact period. If shorter contact periods are used, the chlorine concentration is increased to 50 or 100 mg/l.

Water Service Connections. Components of a customer's service connection include a connection to the main (corporation cock), curb stop or turn-off valve and box, and the line itself. The service connection may be installed when the water mains are laid. Installation operations consist of trenching, main tapping, laying the line, installing the valves, and backfilling.

The trench may be dug by hand or by small backbone. When dug by hand, the width must be sufficient to accommodate the digger. The trench bottom should be relatively flat and on the necessary grade. Special bedding is not required unless the soil is corrosive in nature and the pipe is not corrosion-resistant. Where the service line is made under a pavement, the pavement is removed and replaced after the installation is completed.

Methods for tapping service lines to mains vary depending on the service line size and material. Where the size and the wall thickness of the main are sufficient to provide adequate full threads for the corporation cock, small-size service lines are connected to the main by direct drilling, tapping, or by other methods, and insertion of the corporation cock into the main. If the main is under pressure, the tapping, drilling, and insertion operations are done with a special tapping device. This operation is known as a wet tap.

If the pipe wall is too thin for direct tapping or will not provide the required number of full threads, service clamps are used. In such cases, drilling is done through a corporation cock that has been screwed into the service clamp. For connecting larger service lines, tees, wyes or special branch connections and larger drilling machines are used.

Laying the service line involves not only the laying of the pipe on the trench bottom but the installation and connection of the curb stop and box near the property line. It also involves the connection of the line to the corporation cock and sometimes to the shut-off valve or meter in the customer's premises. Where water meters are set outside the building, the operation is frequently done as a part of the service line installation operation. When the final connection is completed, the installation is tested under pressure.

Backfilling of trenches may be done manually or by machine. In either case, large stones or boulders are not placed directly on the line. Backfilling without tamping is usually done to some reasonable level above grade to allow for settlement. In areas to be paved or repaved, the backfill is tamped to at least 90 per cent of the compaction value of the surrounding areas, then allowed to stand with temporary pavement for at least three months before permanent pavement is replaced.

Pipe Cleaning and Lining

General. Although pipe cleaning and lining per se may not be considered part of construction but rather of maintenance and rehabilitation of existing pipelines, many water system development projects in the Philippines will include such work as part of the initial water supply improvement program.

Pipe cleaning is the process of removing corrosion deposits and slimes from the inside of pipelines. The primary objective of pipe cleaning is to increase the carrying capacity of a pipeline, which has diminished because of deterioration effects and, if possible, restore the carrying capacity of the pipe close to its original capacity.

Lining refers to the process of placing a protective coating on the inside of a pipeline that has been cleaned. Lining of the pipe in-place after the line has been cleaned not only prevents recurrence of internal surface deterioration but also eliminates red water and stops leakage. Cleaning without lining is effective, but there can be no assurance that the pipe's carrying capacity will remain at its improved level for very long because cleaning does not remove the causes of pipeline deterioration. Cleaning alone is an expensive means of maintaining carrying capacity.

Cleaning. Three basic techniques are used for in-place pipeline cleaning. These are (1) drag, (2) hydraulic, and (3) mechanical. The choice of methods depends on the pipe diameter, water volume and pressure available, length of pipe to be cleaned, amount of encrustation or sediment, ease of access, distance between access points, provisions for disposal of wastewater from cleaning operations, and other local conditions.

1. Drag Cleaning. Drag cleaning is usually limited to pipe diameters of 100 to 600 mm. The cleaning equipment is pulled by a power winch through a line that has been removed from service. The method utilizes a spring-steel cleaning tool that is composed of a series of scrapers, followed by an assembly of tight-fitting squeegees. As the tool moves through the line, accumulated deposits are loosened by the scrapers, and then mechanically removed by the squeegees. The separate drag operations are repeated until the pipe wall is clean. Access openings are made in the pipeline at intervals of 90 to 150 m depending on pipe size, line configuration, and condition of pipe.

2. Hydraulic Cleaning. The hydraulic method of pipe cleaning is most practical in long, comparatively straight runs of transmission or arterial mains. The method requires an adequate supply of water at a given pressure. The volume of water available and the required pressure depend on pipe size. The greater the volume of water available, the lower the pressure required.

The tool used in the hydraulic cleaning process consists of spring scrapers so arranged that part of the water pushing the tool is released through it to flush the scrapings and debris ahead of it. The tool usually travels at a rate of 10 to 30 m per minute. The travel speed is controlled by regulating the rate of discharge of wastewater at the end of the pipe run being cleaned.

The operation begins by cutting out a section of the pipe, inserting the tool, replacing the removed section, and making up the joints. At the discharge end of the run, a cut is made into the pipe and a special line attached to discharge the wastewater and debris above ground for ultimate disposal to sewers, storm drains, or acceptable runoff areas. If the tool cannot be discharged through the discharge line, it is stopped in the main and a cut is made in the pipe to remove it. Hydraulic cleaning is relatively rapid, effective, and economical.

3. Mechanical Cleaning. In pipelines greater than 660 mm in diameter, hydraulic cleaning becomes less practical, and mechanical cleaning is used. Mechanical cleaning is accomplished by an electrically-driven and manually-operated machine with rotating scraping blades which remove tuberculation, debris, and existing coatings by a honing action. These machines are driven by an operator who actually observes and controls the entire cleaning operation.

Lining. There are three methods of applying cement-mortar lining to pipelines in place: (1) centrifugal method, (2) reinforced centrifugal method, and (3) Mandrel or tate process.

1. Centrifugal Process. After the pipe has been cleaned, access openings are cut every 150 to 200 m (less in small pipes where bends occur). Bends cannot be negotiated in 100, 150 or 200 mm pipe sizes. After placement, the lining in these diameters may be troweled; for pipes above 200 mm diameter, troweling is always done to provide a smoother finish and the extra carrying capacity that results.

The field equipment for centrifugal lining includes a variable speed winch for pulling the lining machine with its mortar hose and electric cable through the pipe; an electric generator to supply power to the winch and to the revolving head that dispenses the mortar; a specially-designed mortar mixer of the capacity needed to ensure ample mixing time; and a feeder to pump the mortar to the lining machine.

The lining material is usually a 1:2 portland cement-mortar, and the volume of mortar applied to the wall is controlled by the travel speed of the machine. A lining thickness of 5 to 20 mm is common on cast iron pipelines, but it may be as little as 3 mm. The thinner the lining, the smaller the reduction of the original cross-sectional area of the pipe. Thin coatings may be sufficient in smaller pipelines. The thickness of lining for steel pipe lines depends on size, plate thickness, and condition of the metal.

In large mains that contain few service taps or lateral connections, all openings are plugged prior to lining and opened after lining by men working in the pipe. In lines below 400 mm diameter, where men cannot work, very little mortar is thrown into lateral openings, and any obstruction at the corporation cock is removed by blowing out the service line before the mortar sets completely.

Small mains tapped for service lines are usually by-passed by a temporary above ground line to maintain customer service.

The cost of centrifugal in-place lining depends on a number of factors, principally: pipe diameter, pipe length, condition of the line, plan and profile of the line, bends, location and type of valves, length of section that can be removed from service during the operation, by-pass requirements, depth and type of soil cover, access, and traffic problems. The greater the length that can be lined at one time, the greater the production rate and the lower the cost.

Centrifugal in-place lining is applicable to pipe sizes up to 3,650 mm. One of its advantages is that the line can be placed in service 24 hours after the lining process. The process has also been used on newly-installed steel pipelines.

2. Reinforced Lining. When pipelines of 600 mm or greater diameter are badly deteriorated, it may be desirable to reinforce the cement-mortar lining. This reinforcing process consists of three steps. First, a course of mortar one-half the final lining thickness is placed by centrifugal machine, without troweling. Next, spirally-wound reinforcing rod is placed. (The rod spacing depends on pipe size and strength requirements of the equivalent steel area. The size of the rod varies with the size of the pipe and the required reinforcing.) After the steel rod is placed, a second course of mortar is spun into place to the final desired thickness. The spiral rod has two advantages over prefabricated cage steel: it requires less steel, and it conforms to the inside contour of the line.

3. The Tate Process. The mandrel process, commonly known as the Tate process after its Australian inventor, cleans and scours out encrustation from the pipe, then lines the pipe with cement mortar. An advantage of the Tate process is that road opening is kept to a minimum. Only two major digging operations take place at both ends of a 90 m section of main, and only small openings are required to disconnect and temporarily bypass service connections. The exact location of each service connection is obtained by electrifying the household system and sweeping the "live" area with a detector which tells the operator through headphones where the connection is located. Customers suffer only little inconvenience, with full service restorable in 24 hours.

The Tate process can be described briefly as follows: At both ends of a 90 m section, a hole is dug and a 1 m length of main is cut and removed. Flexible steel rods to which a wire rope is attached are pushed through the main from one end and drawn out from the other. An assembly of coil scrapers and steel brushes to scour the pipe, and rubber force cups to clean and dry it, is connected to the wire rope and this is pulled through about 90 m section of main from six to 12 times, until it is completely clean. A special cement-mortar mixture of a relatively high initial water-cement ratio is then introduced into one end of the section and drawn by suction along the 90 m length of main. A "cement gun" which spreads the mortar evenly over the walls of the cleaned pipe is then drawn through by winch. A smooth lining approximately 3 mm thick is left in the main, excess water escapes through the rear of the "gun", and the surplus mortar is removed and used to put a matching 3 mm lining in the 1 m length cut from the main at the start of the operation. This section is reconnected, the road surfaces at the opening are repaired, and the crew moves on to the next section to be cleaned and lined.

Pipe Cleaning in the Philippines

Until recently, pipe cleaning and lining in place have not been practiced in the Philippines. The Metropolitan Waterworks and Sewerage System (MWSS) has included these activities as part of its improvement program. A New Zealand-based company which can undertake these types of work is currently available locally. This firm employs the Tate process of in-place cleaning and lining.

Tunnel Construction Methods

Tunnels for water transmission lines may be constructed by conventional or machine tunnelling. Conventional tunnelling in rock formation involves the cyclical repetition of the following operations: drilling, blasting, loading, and removal of excavated materials; installation of primary supports where necessary; and the mixing, hauling, and placing of concrete to form the secondary lining. It is sometimes desirable to defer the installation of the secondary lining until driving operations have been completed or are remote from the lining operations.

In the machine tunnelling method, a tunnel excavating machine would be employed at one tunnel face simultaneously with conventional tunnelling at the other face. There are many variations of mechanical rock excavators. Most adopt the same principle in which the machine bores a pilot hole into which an expanding "packer" is placed to form an anchor by which the machine pulls itself forward, enabling a larger rotating cutter head to bore the tunnel. The cutter head may be moved forward from 0.5 to 1.2 m within the frame by hydraulic jacks. When the cutter has been advanced to its full distance, the cutter head is retracted and then the frame is pulled forward and locked in place ready to begin the next advance. The cutter head is fitted with teeth or rollers which cut or spall the rock faces as the cutter head revolves. Cutters must be replaced frequently depending on the hardness of the rock being excavated. Tunnel excavations are normally electrically powered. Excavated material is picked up by a series of revolving buckets, discharged into a belt conveyor and carried to rail haulage trucks. A tunnel driven by a mechanical excavator has a smooth bore as contrasted to a jagged, broken rock surface that results from conventional tunnelling methods.

The average rate of tunnelling by either conventional or machine tunnelling would depend on the nature of the materials and conditions encountered. Higher rates can be obtained with a high degree of mechanization and a carefully organized and executed procedure. On the other hand, conventional tunnelling, although it may be slower, will require less foreign exchange costs.

Pumping Stations

General. Water supply pumping stations may be classified into deep well pumping stations, booster pumping stations and raw water pumping stations. The second type may be installed as part of a treatment plant or part of the water distribution system.

Centrifugal and turbine-type pumps are the most commonly used pumping units in waterworks applications. Prime-movers may be electric motor, diesel engine, gas engine, or other suitable energy source which can develop the required power. Because of their relatively low cost compared to other types of prime movers, electric motors are the favored type where electric power is available at reasonable costs. Dual drive pumps can be used for operation by electric motor or by engine.

Pumping installations are usually housed in a structure that will provide protection from the elements and security from theft, tampering, etc. Each station is provided with the necessary suction and discharge piping and valving, controls, and a metering system with suitable indicating, totalising and recording facilities. Attention is also given to water hammer.

The structure which will house the pumps and appurtenant equipment is constructed from locally available masonry, wood and reinforced concrete materials. In some installations, deep well pumps equipped with weatherproof motors are not provided with pumphouse. The interior flanged pipes and valves are made from locally available valves and cement-lined steel or cast iron pipes, wherever possible.

Deep Well Pumps. Two types of deep well pumps in common use are the deep well turbine pump and the submersible (or submergible) deep well pump. The first type consists of impellers in series installed below the minimum expected water level during pumping. Each impeller is encased in a housing or bowl and is called a stage. The number of stages necessary for any given installation depends on the head that each stage can develop at a given pumping rate and on the total pumping head. Power is transmitted to the impellers through suitable shafting from a prime mover usually installed at the ground surface.

The submersible deep well pump is usually equipped with an electric motor drive. In this type the motor is installed in the well itself.

Booster Pumping Stations. The most widely used type of pump for booster pumping stations, whether in a treatment plant or in a distribution system, is the centrifugal pump. A centrifugal pump consists essentially of a rotating impeller which draws water into a center and a stationary casing which guides the water into the discharge outlet. Advantages of the centrifugal pump include ease of operation and repair, low starting torque, increase output with pressure drops or vice-versa, and smooth flow and uniform pressure.

In the Philippines, the manufacture of centrifugal pumps and motors is still in its infancy. For most waterworks projects, it is anticipated that pumping units will be imported items. If and when Philippine-manufactured equipment with the capability, efficiency, and quality desired become available in the future, local product should be considered in the final design and construction phases.

Raw Water Pumping Stations

Raw water pumping stations, as used herein, are intended to mean pump installations that draw water from a surface source such as a spring, river or lake. Such pumping stations are similar in many respects to booster pumping stations but may include some features and facilities not normally needed in booster stations such as intake screens, protection against flood waters, etc.

Water Storage Tanks

In the Philippines, water storage tanks, both elevated and ground tanks, are usually constructed of either cast-in-place reinforced concrete or of steel. Prestressed concrete tanks, although gaining in use and popularity in other countries, have not been used in the Philippines. The relative economics between reinforced concrete and steel tanks depends somewhat on the tank size and tower height for elevated tanks. Generally, in the larger sizes, reinforced concrete tanks are more economical than steel tanks unless steel plates and other foreign-made components can be imported tax-free. In smaller sizes, the construction costs of steel tanks are comparable to that of reinforced concrete. However, maintenance costs of steel tanks are generally higher. This factor can make the total annual costs of steel tanks greater than those of reinforced concrete tanks.

Water Treatment Plants

Water that is to be used for drinking and public water supply purposes must satisfy certain minimum quality requirements with respect to safety, potability, etc. The water is subjected to treatment to upgrade its quality if it does not meet prescribed or desirable standards. As a general rule, all water from surface sources such as rivers, streams and lakes should as a minimum be given "complete" treatment to minimize the risk from water-borne diseases.

Modern "complete" water treatment plants employ the processes of flocculation, sedimentation, filtration, and disinfection. Other additional treatment may be given depending on the quality of the raw water and other factors.

The construction of a modern water treatment plant providing at least complete treatment or its equivalent will require the building of several components utilizing a multitude of skilled tradesmen versed in certain specific fields. The major construction fields which must be utilized to build the treatment plant include:

(1) General construction consisting of all earthwork, reinforced concrete work civil works, and building construction.

(2) Mechanical work consisting of installing pumps, motors, treatment plant equipment such as mechanical feeders, sludge collectors, emergency generators, and other process mechanical equipment. Also, all large size flanged pipes and valves required within the plant may be installed by this specialty.

(3) Electrical work consisting of general wiring of the entire plant for lighting and power. The furnishing and installation of simple controls, instrumentation and communications equipment may also be included as part of the electrical works contract. Where such equipment are complicated and extensive, it may be desirable for this work to be undertaken separately from the general electrical work.

(4) Pipeline and plumbing works including piping for the in-plant water system, sanitary sewers, storm drains, and building plumbing.

With good construction supervision, all these construction work can be done by qualified Philippine contractors. Special material and equipment for the plant will have to be imported.

A P P E N D I X D

O U T L I N E S P E C I F I C A T I O N S

APPENDIX D OUTLINE SPECIFICATIONS

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APPENDIX D

OUTLINE SPECIFICATIONS

Spring Intake Structure

All spring intake structures shall be constructed of reinforced concrete. The intake structure shall be of a size sufficient to capture the maximum spring flow. The spring intake may be circular, rectangular or of other suitable shape. It shall be covered and provided with outlet pipe(s) and valve(s), overflow(s), vent(s), drain(s), covered access manhole(s) and other necessary appurtenances and site works. The intake facility shall also include a weir or other suitable device for flow measurements; security fencing; chlorination facilities (if necessary); general site improvement including drainage facilities for possible surface runoff; and an all-weather access road. Reinforced concrete construction, piping, fittings, valves, and all other materials and attendant work shall conform to LWUA Standards. (The water district shall acquire ownership of the intake structure site.)

Hydraulic Control Structure

Hydraulic pressure control structures on transmission lines for dissipating excess energy shall be impact type in which pressure dissipation is accomplished by the impact of the incoming jet of water on a vertical baffle and by eddies or turbulence formed from the directional change of the jet after it strikes the baffle. The hydraulic control chamber shall be constructed of reinforced concrete and shall be covered. It shall be designed such that it can handle the design maximum flow. The chamber shall be provided with the necessary piping, overflows, and other protective devices. The work shall include general site improvement and security fencing, if necessary. (Ownership of the land on which the control chamber will be built shall be acquired by the district.)

Dams and Appurtenances

The construction of dams and appurtenances shall be performed by firms and personnel experienced in this line of work. The Contractor shall furnish plant and equipment which will be efficient, appropriate and large enough to secure a satisfactory quality of work and a rate of progress which will insure the completion of the work within the stipulated time.

The dam construction will include the main dam structure, upstream and downstream cofferdams, tunnels, diversion channels and spillway.

The zoned embankment dam will consist of a vertical core protected by filter and transition zones, and rolled rock-fill shells. The upstream face of the dam is protected by riprap against wave action.

Materials for the dams shall be as designed and specified and shall be obtained from designated borrow areas, excavations, or manufactured from rock obtained in required excavations.

The areas to be occupied by the required permanent construction and the surfaces of all borrow pits shall be cleared of all trees, stumps, exposed roots, brush, rubbish, and other objectionable matter. Excavation shall be made to the specified lines, grades, and dimensions. All necessary precautions shall be taken to preserve the material below and beyond the established lines of all excavation in the soundest possible condition. All excavations for embankment and structure foundations shall be made in the dry.

The diversion tunnel shall be concrete lined. The portal structure will be provided with a slot for installation of stop logs for closure of the tunnel. The spillway will consist of an ungated overflow concrete structure and a concrete lined chute.

The raw water intake will be multi-ported and shall be constructed of reinforced concrete.

Diversion Dams

The construction of the diversion dam shall be performed by firms and personnel experienced in this line of work. The Contractor shall exercise care to preserve the natural landscape and shall conduct his construction so as to prevent any unnecessary destruction, scarring, or defacing of the natural surroundings in the vicinity of work.

The Contractor shall construct and maintain all necessary cofferdams, channels, flumes, drains, sumps, and/or other temporary diversion and protective works; shall furnish all materials required therefore; and shall furnish, install, maintain and operate all necessary pumping and other equipment for removal of water from the various parts of the work free from water.

All concrete work shall be in accordance with LWUA standard specifications and supplementary specifications.

Access and Service Roads

The construction of access and service roads to water supply facilities shall include all necessary clearing and grubbing, excavation, fill and backfill, roadbed preparation, installation of base course, surface finish or paving, bridges, and all drainage structures and facilities. The work will involve improvement and/or extension of existing roads and the construction of new access and service roads.

All roads shall be constructed in conformity with the specified lines, sections and grades. Materials and their installation shall be in accordance with the latest revision of the Bureau of Public Highways Standard Specifications for Highways and Bridges, local requirements, and supplementary specifications.

Water Transmission Pipelines

Raw and treated water transmission pipelines may be constructed of cast iron, ductile iron, asbestos cement, steel or prestressed concrete (with steel cylinder) pipe. Soil and corrosion studies shall be conducted prior to the final selection of pipe material. The transmission lines shall be equipped with all necessary valves and appurtenances such as shut-off and sectioning valves, air/vacuum and air release valves, blow-offs, inspection manholes, expansion joints, flexible couplings, anchorages, thrust blocking, and surge arresters.

Pipe, fittings, valves, other materials and installation, jointing, testing and disinfection shall be in accordance with LWUA Standard Specifications, where such specifications are applicable to the particular material or work. Available Standard Specifications of LWUA include those for cast iron, asbestos cement and steel pipes; gate and butterfly valves; blow-offs; air valves; and work relating to their installation.

Ductile iron pipe shall be manufactured in accordance with AWWA C151 "Ductile Iron Pipe, Centrifugally Cast in Metal Molds or Sand-Lined Molds". Fittings shall be either cast iron or ductile iron conforming to AWWA C110 "Gray Iron and Ductile Iron Fittings, 2 in through 48 in ". All pipe and fittings shall have a cement mortar lining and bituminous seal coat on the inside in accordance with AWWA C104 "Cement Mortar Lining for Cast Iron and Ductile Iron Pipe and Fittings".

Prestressed concrete cylinder pipe shall conform to AWWA C301, "Reinforced Concrete Water Pipe-Steel Cylinder Type, Prestressed". Fittings shall conform to the specifications for cast iron, ductile iron, or steel pipe.

In general, all piping shall be designed for a minimum working pressure of 10.5 kg/sqcm (150 psi). The pressure class of fittings, couplings, special castings, and valves shall be at least equal to the pressure class of the pipe to be installed. Joints shall have the same or greater strength than the connecting pipe.

Shut-off and sectioning valves shall be either gate valves or butterfly valves, depending on the size and other factors. A sufficient number of air valves shall be provided to insure full protection of the pipeline.

All pipeline installation shall be in strict conformance with applicable AWWA and/or LWUA Standards and with the respective manufacturer's instructions and recommendations.

Water Treatment Plant

Water treatment plants designed to provide complete treatment would generally include facilities for chemical mixing, flocculation, sedimentation, rapid sand filtration, post chlorination, chemical storage, backwashing, treated water storage, and waste washwater and sludge disposal.

Chemical mixing chambers, flocculation and sedimentation tanks, filter boxes and treated water storage tanks shall be constructed of reinforced concrete.

Filter materials shall consist of filter sand and anthracite conforming with specified requirements with respect to composition and grading. For each filter unit there shall be installed the necessary control valves, rate of flow controller, loss of head gage, flow meter and recorder.

Instrumentation shall include suitable equipment to vary chemical feed rates in proportion to flow.

Concrete work, yard and in-plant piping, and painting work shall be in accordance with LWUA Standard Specifications and supplementary specifications.

Piling (if required), structural steel, architectural works, instrumentation and electrical works, mechanical equipment, and all other items not covered by LWUA Standards shall be constructed as specified.

Administration Building

The construction of administration buildings shall be of the materials and workmanship called for in the drawings and specifications. The administration building will generally consist mainly of offices but may include a water analysis laboratory, meter testing and repair shop, general work shop, and storage facilities. Items of work shall include site preparation; foundations; concrete and masonry work; roofing and metal work; carpentry and joinery; plumbing, ventilation, and air-conditioning systems; lighting and power systems; architectural and other special finishes; painting work, landscaping and general site improvement work. Applicable LWUA Standard Specifications shall be employed in the construction work.

Well Construction

Deep well construction shall include the furnishing of all materials (except those that may be furnished by the Owner), equipment, tools, labor and all appurtenances and incidental work for construction of the deep wells. The work shall include drilling; installation of temporary casing, conductor pipe, well screen; developing and testing of the well; gravel packing; grouting, well completion and disinfection; and site work and clean-up.

The well shall be drilled using the cable tool (Percussion) and/or rotary process, or other process acceptable to and approved by the Engineer. Well casing and/or conductor pipe shall be of the diameters, materials and class specified, or better.

For gravel packed wells only clean, washed gravel composed of well rounded particles and of specified grading shall be used. The procedure to be employed shall be as approved by the Engineer.

The topmost 12 m of the annular space between the conductor pipe and hole shall be filled with cement grout. The mixtures, method of mixing, and consistency of grout shall be as approved by the Engineer.

Developing of the well shall be done with care and by methods that will not cause damage to the well or cause adverse subsurface conditions that may destroy barriers to the vertical movement of water between aquifers. Upon completion of well development, test pumping shall be done in accordance with a test procedure that will be furnished to the Contractor by the Engineer. The pump shall be operated continuously for specified durations and pumping rates.

Immediately following satisfactory construction and development samples of the well water shall be collected and analyzed in a laboratory acceptable to the Owner.

After completion of all construction, development, testing and related work at each well site, all equipment and residual materials shall be removed from the site. The site shall then be restored to a condition as nearly as possible to that which existed before the well construction work, unless otherwise specified.

Flow Meters (Mainline Meters)

Flow meters for mainlines shall be differential pressure type, propeller meters, or other suitable and acceptable devices. Differential pressure type meters may be venturi tubes, Dall flow tubes, orifices or nozzles. The flow meter shall include suitable instrumentation for remote indicating, recording and totalling. Flow meter and accessories shall be products of reputable manufacturers that have manufactured such devices for fluid measurement for at least five years.

The venturi meter tube shall be of standard or long form design, the included angle of the outlet cone being approximately $8^{\circ} - 10'$. The tube shall have a body of high tensile gray iron or close grain, high tensile iron. Both inlet and throat shall have integrally cast annular pressure chambers with multiple even spaced vents communicating with the interior of the meter tube.

Propeller type meter shall have the same nominal inside diameter throughout its length to offer minimum obstruction to the flow. The meterhead shall be connected to the tube by means of a flanged connection, designed for easy removal from the tube for inspection and repair. The meter shall be furnished with a propeller of plastic or other suitable material mounted in the meter tube. The meter shall register within 2 per cent of the true flow of water at all flows within the minimum and maximum rating. The propeller type meter shall conform to AWWA C704-70 "Standard for Cold Water Meters - Propeller Type for Main Line Applications".

The flow meter shall be designed for a minimum working pressure of 10.5 kg/cm^2 (150 psi). Range of flow will be specified by the purchaser. Ends shall be flanged 250 lb American Standard unless otherwise specified.

Deep Well Turbine Pump

Deep well turbine pump shall be water lubricated, line shaft vertical turbine pump, electric motor or diesel engine driven or both (dual drive), as required. Pump characteristics and operating conditions will be specified for each particular installation. Pump shall conform to ANSI B58.1 - 1971 (AWWA E101 - 71) "American National Standard for Deep Well Vertical Turbine Pumps - Line Shaft and Submersible Types". Diesel engine and accessories shall conform to the specifications for diesel engine, except as modified herein.

For motor-driven pump, the motor shall be full voltage starting where the electric power system capacity and regulations permit; otherwise the motor shall be star-delta starting. The motor shall be vertical hollow-shaft squirrel cage induction type complying with ANSI C50.2. The motor shall be of ample size to drive the pump continuously over the specified range at the ambient temperature without the load exceeding the service factor. Motor operating characteristics (voltage, phase, frequency, speed) and control and protective devices shall be as specified. A suitable base of high grade cast iron or fabricated steel shall be provided for mounting the meter, and with discharge elbow having above-ground discharge outlet with companion flange.

With an engine drive, the power shall be applied to the pump shaft through a right angle gear set. The horizontal shaft shall be connected to the engine by a flexible-shaft coupling.

Pump bowls, impellers, pump shafts, line shafts, discharge column assembly, suction pipe and strainer shall conform to ANSI B58.1 - 1971.

A suitable air line of galvanized iron pipe or copper tubing of sufficient length to extend from the surface to the top of the bowl assembly shall be furnished with altitude gage reading in meters and connections for air pump.

The pump and prime movers shall be products of reputable manufacturers which have been regularly engaged in the manufacture of these equipment for the last five years. The manufacturer shall, if required, furnish a sworn statement that the equipment furnished and installed comply with the requirements of the applicable standards and the specifications. The equipment manufacturer/supplier shall furnish the services of competent personnel to supervise the installation and testing of the equipment. Spare parts, operation and maintenance manuals shall be provided. The pump equipment and controls shall be housed in a suitable permanent structure that provides protection from the elements, damage, or vandalism.

<u>Submersible Deep Well Pump</u>

Submersible deep well pump shall conform to ANSI B58.1 - 1971 (AWWA E101 - 71) "American National Standard for Deep Well Vertical Turbine Pumps - Line Shaft and Submersible Types". Operating conditions and requirements will be specified for each particular installation.

The motor shall be of the squirrel cage induction type, suitable for across-the-line starting and shall be capable of reduced-voltage starting. It shall be capable of continuous operation under water at the specified conditions. Motor operating characteristics (voltage, phase, frequency, speed and control and protective devices) shall be as specified.

Submersible cable, surface plate, strainer, discharge pipe, pump bowls, impellers shall comply with the requirements of current ANSI B58.1.

The pump and accessories shall be products of reputable manufacturers which have been regularly engaged in the manufacture of these equipment for the last five years. The manufacturer shall, if required, furnish a sworn statement that the equipment furnished and installed comply with the requirements of the applicable standards and the specifications. The equipment manufacturer or supplier shall furnish the services of competent personnel to check the installation and testing of the equipment. Spare parts, as specified, and operation and maintenance manuals shall be be furnished.

Diesel Engine

The engine shall be of the vertical in-line, or V-type multi-cylinder, full diesel, mechanical injection, heavy duty rating type. The engine may be either two or four stroke cycle and shall have specified rotative speed and piston speed. It shall be a model which has been in satisfactory operation in similar service at the same or higher rating and speed for at least five (5) years. The engine's continuous duty rating, after deducting power consumed by all engine-driven auxiliaries, shall be not less than the horsepower required to operate the driven equipment at its specified full rated load. The engine rating shall be adjusted for operation at specified conditions of elevation and ambient temperature.

The unit shall be furnished for battery starting. Starting shall be accomplished by a 12 or 24 volt electric starter, as recommended by the manufacturer, which shall be capable of withstanding five (5) minutes' continuous cranking.

The diesel engine shall be furnished with complete fuel system, lubrication system, governor, safety devices and controls, engine instrumentation, cooling system, exhaust system and accessories as will be specified. Accessories to be furnished include starting battery, automatic battery charger, manufacturer's standard spare parts, detailed operating and maintenance manuals and parts lists, complete set of gaskets and spare set of matched V-belts, and one spare set of fuel injectors.

Diesel Generator Unit

The diesel generator unit shall be complete with excitation system, controls, steel subbases, exhaust silencer, fuel system and all essential and desirable auxiliaries for a complete installation. The unit shall be arranged for manual pushbutton starting and stopping and manual transfer of load to the unit when it has attained rated frequency and voltage. The engine-generator set shall be a factory assembled unit especially designed for operation on No. 2 diesel fuel oil.

The engine generator set shall be the standard product of a manufacturer regularly engaged in the production of this type of equipment. The diesel engine and accessories shall be as specified under Diesel Engine. The diesel engine shall be arranged for direct connection to the alternating current generator.

The generator shall be especially designed for direct connection to the diesel engine and shall be for the specified phase, frequency, and voltage. Tropical insulation with fungus protection shall be provided. Each unit shall be properly screened to prevent the entrance of rodents. The complete generator unit shall be free from critical speeds and torsional vibration that will endanger its satisfactory operation, or cause undue vibration in any part of the equipment, throughout its entire operating range of speed and load.

The generator control panel shall be either shock-proof mounted on the generator unit or a free standing enclosed unit for floor mounting adjacent to the generator unit. It shall have at least the following instrumentation and equipment: AC voltmeter, AC ammeter, frequency meter, indicating KW meter, combination ammeter-voltmeter phase selector switch, 3 pole line circuit breaker of suitable amperage, and elapsed running time meter.

Chlorination System

Chlorine gas, in 150-lb cylinder or ton containers, whichever is most suitable for the particular installation, shall be employed in all chlorination stations. (Hypochlorite solutions are an acceptable substitute.) Chlorine solution shall be added to the water to be treated through chlorination equipment and accessories specifically designed and suited for the purpose.

Chlorinators shall be the vacuum operated, solution feed type which meter the chlorine gas under vacuum and dissolve it in water forming a concentrated solution that is then injected into the water. Direct feed chlorinators will not be permitted.

Chlorinators may be directly mounted on 150-lb cylinder or ton container, wall - or floor-mounted units. Models of a design that permit enlarging the capacity by replacement of [illegible] the flow meter will be preferred to those with fixed maximum capacity. The chlorinators shall also be of a design that will permit either manual or automatic operation, the latter with the use of auxiliary equipment. At least two units shall be provided and installed, one serving as stand-by. The completed installation shall include all necessary piping, valves, controls and accessories including chlorine scales, gas masks, and gas leak detection and alarm systems.

Chlorinators and accessories shall be housed in a separate building or rooms specially designed for the equipment and their functions. (The site for the chlorination facilities shall be acquired by the district and necessary improvements and protective features shall be incorporated.)

Installation of Equipment - General

Special care shall be taken to ensure that all equipment are installed in proper alignment and level. This applies to, but is not limited to, pumps, drive units, gears, sluice gates, mechanical, electrical, instrumentation and communications equipment, and their appurtenances. Equipment contractors will be required to supply the necessary anchor bolts, drawings and templates of anchor bolts.

The general and equipment contractors shall be responsible for the equipment they supply. They shall use only competent personnel and appropriate equipment necessary to properly align, level and secure equipment in place.

The installation of the major equipment specified in the Contract shall be performed under the supervision of competent representatives of the manufacturers. The manufacturer's representative shall not only supervise the installation of the equipment, but shall also supervise the adjustments and testing of the equipment to insure that it will operate in a satisfactory manner as specified or intended. These representatives shall also instruct personnel and mechanics of the Owner in the operation, care and maintenance of the equipment. Complete sets of operating and maintenance instructions shall be furnished as required.

The Contractor shall submit a certificate from the manufacturer stating that the installation of the equipment is satisfactory, that the unit is ready for operation and that the operating personnel have been sufficiently and thoroughly instructed in the proper operation, lubrication and care of the unit.

Installation of deep-well vertical turbine pumps is particularly critical if long service-free life is to be expected. Installation should only be done by experienced personnel following specifications of ANSI B58.1 - 1971 (AWWA E101 - 71) and paying particular attention to straightness of line shafts and proper alignment of all parts.

Booster Pump Stations

Booster pump stations shall be designed and constructed to comply with established criteria and standards of the LWUA as well as other requirements peculiar to each site. Booster pump facilities will generally consist of pumphouse, pump units, suction and discharge piping, control valves, gauges, flow meter and recorder, control and protective equipment, site works and security fencing.

Pump units shall be centrifugal, turbine, or submersible type. Centrifugal and turbine type pumps shall be either electric motor or diesel engine driven. Submersible booster pumps shall be motor driven. Each pump shall have optimum efficiency at the specified duty point. Motors for electrically driven pumps shall be of adequate horsepower for the full operating range of the pump.

Storage Tanks

Elevated and ground storage tanks shall be generally constructed of reinforced concrete. For small capacity elevated tanks, steel tanks on steel towers may prove to be more economical and should be given consideration in the final design phase. Ground tanks may be circular, rectangular or other shape acceptable to and approved by the Owner. Tanks shall be designed in accordance with applicable national and local structural and sanitary codes. It shall be structurally sound with ample provisions for wind and/or seismic stresses. Concrete and reinforced concrete work including waterproofing, disinfection, painting, and all other incidental work shall be in strict compliance with LWUA Standard Specifications and Supplementary Specifications. All tanks intended for storing potable water shall be covered and watertight. For both elevated and ground tanks, available LWUA standard tanks shall be used to the fullest extent possible. Necessary piping, valves and accessories for operation, maintenance and safety shall be provided. Piping shall include inlet-outlet, overflow, drain, and vent. Shut-off valves, check valves, automatic flow control valves, water level indicators and instrumentation, shall be provided as required.

Distribution System Piping and Components

General requirements with respect to materials, installation and other appurtenant work for water transmission pipelines are applicable to distribution system pipelines. Other distribution system components, including fire hydrants, service connections and customer water meters, shall be installed according to LWUA standard details and standard specifications.

Pipe Cleaning and Lining

Pipe cleaning and lining shall include all materials, labor, equipment and all incidental work necessary to clean and line the interior of pipelines in-situ and restore the pipelines in service. The work shall be performed by trained workmen under the supervision of personnel experienced and competent in this particular line of work.

Interior lining shall be cement mortar. The interior of pipes to be lined shall be thoroughly cleaned of all rust, incrustation, dirt, oil and grease and other foreign matter. Necessary repairs, including replacement, shall be made to pipe sections that have suffered severe deterioration and/or corrosion. Any section of pipe that shall be cleaned and lined shall be restored to service in as short a time as possible, preferably within 24 hours.

All work shall be performed in accordance with AWWA Standard C602-67, except as may be modified in the specifications. The work shall include all excavation and backfill; installation and removal of temporary by-pass pipes, service connections, plugs, closure pieces; making and closing required access openings; surface restoration; clean-up and disposal of debris and other waste materials.

APPENDIX TO CHAPTER VII

APPENDIX TABLE VII-B-1

WATER WELL DATA SUMMARY

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m) Total	Cased	SWL[1]	Test PWL[2]	Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
LC-1	Pinagtongulan	100	38	36	28.9	30.5	0.6	0.4	1960
LC-2	Adya	100	37	31	9.2	9.8	0.5	0.8	1960
LC-3	Tangway	100	162	160	131.1	132.3	0.6	0.5	1958
LC-4	Fernando Air Base	150	93	90	19.8	44.2	3.2	0.1	1958
LC-5	Pagulingin Bata	100	37	36	24.4	25.9	0.3	0.2	1959
LC-6	Pangao South	75	37	32	21	21.3	0.8	2.5	1964
LC-7	Sitio Barandal San Francisco	75	45	38	30.5	32.0	0.3	0.2	1962
LC-8	San Salvador	75	77	57	44.2		0.6		1956
LC-9	Inosloban-Marauoy School		37		8.8				
LC-10	Antipolo	112	79	31	47				1955
LC-11	Malalim na Gulod	75	63	45	16.5		0.6		1956
LC-12	Mataas na Lupa	75	37	31	10.7	12.2	1.6	1.1	1964
LC-13	Malabanan	75	113	113					1964
LC-14	Bulacnin	75	98	92	85.4	87.5	0.4	0.2	1956
LC-15	Tambo School	75	51	51	18.6	20.4	0.4	0.2	1956
LC-16	Rizal	150	42	34	18.3	21.0	0.4	0.2	1956
LC-17	San Jose School		50	29	28.7	32.0	0.4	0.2	1956
LC-18	Marauoy	150	56	55	12.2		1.0		1955
LC-19	Bolboc School	100	52	35	13.7		0.6		1956
LC-20	Cumba near Chapel	75	24	21	12.2	14.6	0.3	0.2	1959
LC-21	Sto. Toribio near RR Track	100	29	29	15.9	17.4	1.6	1.1	1961
LC-22	San Carlos near RR Track	100	36	32	22.6	23.5	0.4	0.4	1961
LC-23	Anilao	100	92	86	50.3	51.2	0.6	0.6	1959
LC-24	Pagulingin (West)	100	32	25	15.2	15.9	0.3	0.5	1959
LC-25	Anilao, Labac	100	38	33	25.9	26.5	0.3	0.5	1961

[1] Static Water Level
[2] Pumping Water Level

APPENDIX TABLE VII-B-1

WATER WELL DATA SUMMARY (Continued)

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m)				Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
			Total	Cased	SWL	Test PWL			
LC-26	Labac	100	39	38	27.4	29.0	0.3	0.2	1961
LC-27	Tambo	100	32	32	15.2	18.3	1.6	0.5	1959
LC-28	Tipacan	100	75	72	58.5	60.1	0.3	0.2	1959
LC-29	Antipolo School Site	100	70	59	42.7	48.8	0.6	0.1	1962
LC-30	Paninsingin School Site	100	26	26	13.1		0.6		
LC-31	Lipa Experiment Station	150	72	63	13.7	35.1	3.8	0.2	
LC-32	Masiit Mabini	100	32	32	17.7	18.3	0.6	1.1	1961
LC-33	Bulacnin	112	159	156	131.1		0.6		1964
LC-34	Pinagkawitan	100	49	44	32.0	33.5	0.3	0.2	1961
LC-35	Camp Malvar	200	101	59	16.8	22.9	3.8	0.6	1953
LC-36	Balintawak		32		15.2	18.3	1.0	0.3	1953
LC-37	San Celestino								
LC-38	San Benito								
LC-39	Bugtong na Pulo		78						1955
LC-40	Fernando Air Base	150	92	92	18.3	25.0	9.5	1.5	1962
LC-41	Bo. Adya School Site	100	21	20	14.0	15.2	0.6	0.5	1959
LC-42	Bo. Halang		152						1961
LC-43	Fernando Air Base	200	96	89	22.9	47.3	6.3	0.2	1962
LC-44	Lipa City-City Hall	200	75	60	15.2				1958
LC-45	City Hall Compound	150	92	61	14.6	19.2	15.8	3.4	1974
LC-46	Banaybanay, Lipa City	150	61	61	18.6				1971
LC-47	Dagatan, Lipa City	150	152	146	48.8	80.5	1.3	0.1	1970
LC-48	Marauoy, Lipa City	200	92	92	11.4				1969
LC-49	Lipa Cathedral Compound	150	61	53	14.3	18.8	1.6	0.4	1969
LC-50	Fernando Air Base	200	92	88	20.1	37.8	4.2	0.2	1968

APPENDIX TABLE VII-B-1

WATER WELL DATA SUMMARY (Continued)

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m)				Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
			Total	Cased	SWL	Test PWL			
LC-51	Fernando Air Base	200	92	92	18.0	42.4	5.9	0.3	1968
LC-52	Bo. Bagong Pook, Lipa City	150	92	64	22.9				1968
LC-53	Marauoy, Lipa City	200	61	37	10.7		3.8		1966
LC-54	Bugtong na Pulo Elementary School	100	152	152	70.9		1.5		1960
LC-55	Lipa City, Batangas	200	93	43	13.7	13.7	1.3		
LC-56	Fernando Air Base, Lipa City	150	102	41	19.2	20.4	1.3	1.1	
LC-57	Fernando Air, Base, Lipa City	200-150	93	96	18.3	39.6	3.8	0.2	
LC-58	Marketplace, Lipa City	150			12.6	14.2	5.7	3.6	

APPENDIX TABLE VII-B-2

TEST WELL DATA - LIPA CITY

Data: Start Pumping Test 9:51 pm, April 19, 1976
Start Recovery Observation 6:20 pm, April 21, 1976
Original Static Water Level 51.118 m
Observation Well - None

Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)	Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)
0 (Start Pumping)	51.118	0	40	59.959	8.841
1	-	-	45	60.213	9.095
2	54.548	3.430	50	60.341	9.223
3	55.056	3.938	55	60.290	9.172
4	56.225	5.107	60	60.341	9.223
5	57.393	6.275	70	60.315	9.197
6	57.876	6.758	80	60.976	9.858
7	58.232	7.114	90	61.001	9.883
8	58.333	7.215	100	63.085	11.967
9	58.359	7.241	110	60.468	9.350
10	58.283	7.165	120	61.433	10.315
11	58.308	7.190	150	63.542	12.424
12	58.359	7.242	180	60.595	9.477
13	58.384	7.266	240	-	- *
14	58.435	7.317	300	58.181	7.063
15	58.486	7.368	360	58.461	7.343
16	58.512	7.394	420	58.817	7.699
17	58.537	7.419	480	59.045	7.927
18	58.588	7.470	540	59.197	8.079
19	58.638	7.521	600	59.375	8.257
20	58.664	7.546	660	61.484	10.366*
22	58.766	7.648	720	61.966	10.848
24	58.893	7.775	780	61.687	10.569
26	59.096	7.978	840	60.544	9.426*
28	59.273	8.155	900	59.756	8.638
30	59.375	8.257	960	61.661	10.543
32	59.527	8.409	1020	61.230	10.112*
34	59.680	8.562	1080	59.476	8.358
36	59.781	8.663	1140	59.197	8.079*
38	59.858	8.740	1200	60.442	9.324

*Recovery determined from final pumping water level

APPENDIX TABLE VII-B-2 (Continued)

TEST WELL DATA - LIPA CITY

Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)	Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)
1260	59.603	8.485	10	51.676	7.268
1320	60.137	9.019	11	51.549	7.395
1380	59.553	8.435	12	51.474	7.470
1440	59.071	7.953	13	51.398	7.546
1500	58.508	7.190	14	51.372	7.572
1560	58.105	6.987	15	51.296	7.648
1620	58.156	7.038	16	51.271	7.673
1680	57.952	6.834	17	51.245	7.699
1740	60.061	8.943	18	51.245	7.699
1800	60.696	9.578	19	51.195	7.749
1860	60.137	9.019	20	51.144	7.800
1920	59.654	8.536	22	51.093	7.851
1980	59.781	8.663	24	51.017	7.927
2040	60.747	9.629	26	50.966	7.978
2100	60.798	9.680	28	50.940	8.004
2160	59.324	8.206	30	50.915	8.029
2220	60.366	9.248	32	50.839	8.105
2280	60.188	9.070	34	50.813	8.131
2340	60.391	9.273	36	50.788	8.156
2400	60.366	9.248	38	50.762	8.182
2460	60.569	9.451	40	50.712	8.232
2520	60.366	9.248	45	50.661	8.283
2580	59.451	8.333	50	50.610	8.334
2640 (Stop			55	50.585	8.359
Pumping)	58.944	7.826*	60	50.559	8.385
0 (Start			70	50.407	8.537
Recovery)	58.944	0	80	50.356	8.588
1	53.684	5.260	90	50.280	8.664
2	53.888	5.056	100	50.203	8.741
3	53.583	5.361	110	50.152	8.792
4	53.100	5.844			
5	52.337	6.607	120	50.102	8.842
6	52.337	6.607	150	49.949	8.995
7	52.185	6.759	180	49.873	9.071
8	52.007	6.937	240	49.720	9.224
9	51.753	7.191	300	49.492	9.452

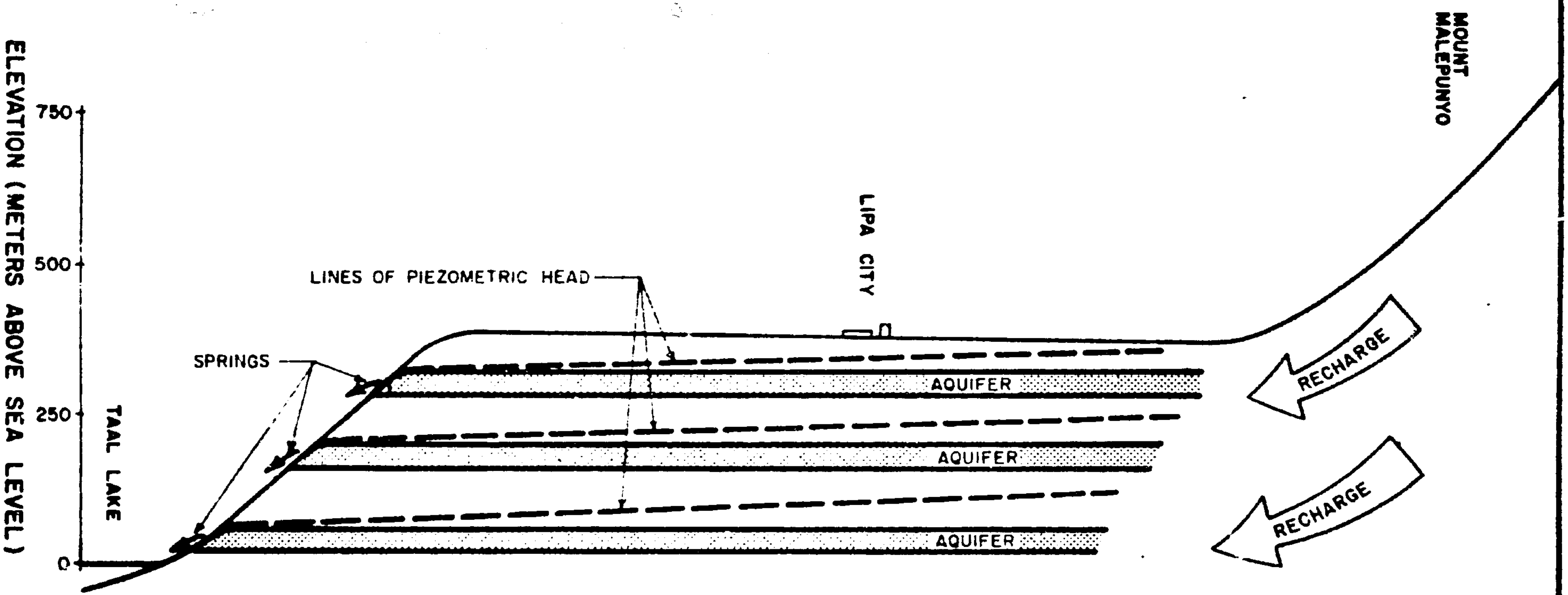
*Recovery determined from final pumping water level

APPENDIX TABLE VII-B-2 (Continued)

TEST WELL DATA - LIPA CITY

Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)	Cumulative Time From Start (min)	Water Level (m)	Drawdown/ Recovery (m)
360	49.237	9.707			
420	49.161	9.783			
480	49.110	9.834			
540	48.932	10.012			
600	48.882	10.062			
660	48.882	10.062			
720	48.806	10.138			
780	48.756	10.188			
840	48.578	10.366			
900	48.552	10.393			
960 (Stop Recovery)	48.552	10.392			

NOTE: Recovery determined from final pumping water level.



APPENDIX FIGURE VII-B-1
WEST TO EAST SCHEMATIC CROSS-SECTION NEAR LIPA CITY

DESCRIPTIVE DATA

WELL NO. (CDM) LC-3
(OTHER) BPW 17122
LOCATION BARRIO TANGWAY
CITY LIPA
PROVINCE BATANGAS
CONST. BY
DRILLER AGUILAR
STARTED MARCH 1958
COMPLETED APRIL 1958
OWNER
STATUS
CASING DIAMETER 100 MM

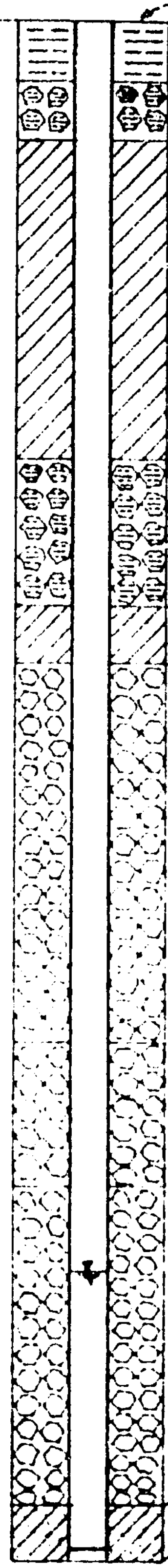
DRILLER'S TEST DATA:
DATE APRIL 1958
STATIC WATER LEVEL 131.1 M
PUMPING WATER LEVEL 132.3 M
TEST PUMP YIELD 0.6 LPS

REMARKS:
CASING DEPTH = 160.1 M
OPEN HOLE = 160.1 - 161.6 M
SPECIFIC CAPACITY = 0.5 LPS/M

WATER QUALITY DATA:
WATER IS FRESH WHEN DRILLED

GRAPHIC LOG

DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
		BROWN CLAY
6.1	20	
		ADOBE CLAY
12.2	40	
		HARD ROCK
45.7	150	
		ADOBE CLAY
61.0	200	
		HARD ROCK
67.1	220	
		ADOBE
106.7	350	
		HARD ADOBE
122.0	400	
131.1	430	
		ADOBE
155.5	510	
		SAND ROCK
160.1	525	
161.6	530	



APPENDIX FIGURE VII-B-2
WELL DATA SHEET
WELL LC-3

DESCRIPTIVE DATA

WELL NO. (CDM) LC-40
(OTHER) BPW 8-62-221
LOCATION FERNANDO AIR BASE

CITY LIPA
PROVINCE BATANGAS
CONST. BY
DRILLER ROSALES
STARTED FEBRUARY 1962
COMPLETED APRIL 1962
OWNER

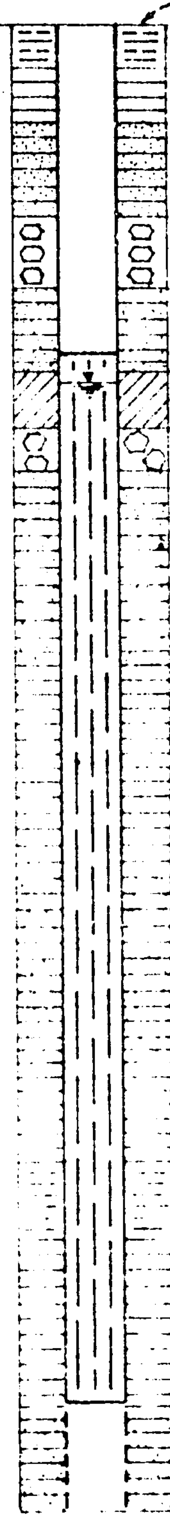
STATUS

CASING DIAMETER 200 MM

DRILLER'S TEST DATA:
- DATE APRIL 1962
- STATIC WATER LEVEL 22.9 M
- PUMPING WATER LEVEL 47.3 M.
- TEST PUMP YIELD 6.3 LPS

REMARKS:
- CASING DEPTH = 88.7 M
- PERFORATED CASING = 22.3 - 88.7 M
- OPEN HOLE = 88.7 - 96.0 M
- SPECIFIC CAPACITY = 0.3 LPS/M

GRAPHIC LOG



DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
		YELLOW CLAY
3.1	10	ADOBE STONE
6.7	22	SANDSTONE
12.5	41	HARD ADOBE
16.8	55	SANDSTONE
21.3	70	
22.3	73	SOLID ROCK
25.6	84	BLUE ADOBE
28.7	94	SANDSTONE
41.2	135	ADOBE STONE
54.9	180	SANDSTONE
61.1	220	ADOBE STONE
73.2	240	BLUE CLAY
78.4	257	ADOBE STONE
88.7	291	
89.9	295	SANDSTONE
96.0	315	

APPENDIX FIGURE VII-B-3
WELL DATA SHEET
WELL LC-40

DESCRIPTIVE DATA

WELL NO. (CDM) LC-42
(OTHER) BPW 8-60-6
LOCATION BARRIO HALANG

CITY LIPA
PROVINCE BATANGAS
CONST. BY
DRILLER
STARTED DECEMBER 1960
COMPLETED JANUARY 1961
OWNER

STATUS

CASING DIAMETER

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL

PUMPING WATER LEVEL

TEST PUMP YIELD

REMARKS:
THIS WELL WAS ABANDONED FOR LACK OF WATER

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
1.5	5	TOP SOIL
10.7	35	ADOBE STONE
24.4	80	ADOBE ROCK
32.0	105	YELLOW CLAY
42.7	140	ADOBE ROCK
67.1	220	YELLOW CLAY
91.5	300	ADOBE ROCK
109.8	360	YELLOW CLAY
122.0	400	ADOBE STONE
141.8	465	YELLOW CLAY
144.8	475	SOFT ADOBE CLAY
152.4	500	STICKY CLAY

APPENDIX FIGURE VII-B-4
WELL DATA SHEET
WELL LC-42

DESCRIPTIVE DATA

WELL NO. (CDM) LC-45
(OTHER) BPW 5868
LOCATION CITY HALL

CITY LIPA
PROVINCE BATANGAS
CONST. BY
DRILLER CAMARADOS
STARTED OCTOBER 1953
COMPLETED DECEMBER 1953
OWNER LIPA CITY WATER DISTRICT

STATUS OPERATIONAL

CASING DIAMETER 150 MM

DRILLER'S TEST DATA:
DATE DECEMBER 1953
STATIC WATER LEVEL 14.6 M

PUMPING WATER LEVEL 19.2 M

TEST PUMP YIELD 16.1 LPS

REMARKS:
CASING DEPTH = 61.3 M.
OPEN HOLE = 61.3 – 91.5 M.
SPECIFIC CAPACITY = 3.5 LPS/M

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
2.1	7	BROWN CLAY
4.6	15	CLAYEY BROWN SAND
6.1	20	PEBBLES OF SCORIA GRAY TO BLACK
14.6	48	
18.3	60	TUFF
20.4	67	DARK BROWN CLAYEY SAND, FINE TO COARSE
24.4	80	BROWN CLAY & SAND
54.0	177	TUFFACEOUS SAND, POORLY CONSOLIDATED BROWN SAND GRAINS
60.1	197	GRAY TUFF
61.3	201	
75.3	247	TUFFACEOUS CLAY, BROWN
91.5	300	BROWN TUFFACEOUS SANDY CLAY; SAND IS FINE TO COARSE

APPENDIX FIGURE VII-B-5
WELL DATA SHEET
WELL LC-45

DESCRIPTIVE DATA

WELL NO. (CDM) LC-47
(OTHER)
LOCATION DAGATAN

CITY LIPA
PROVINCE BATANGAS
CONST. BY
DRILLER KATIGBAK
STARTED 18 MARCH 1970
COMPLETED 17 MAY 1970
OWNER

STATUS

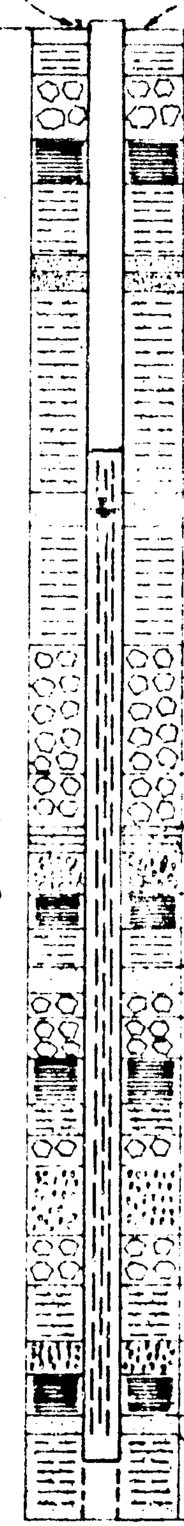
CASING DIAMETER 150 MM

DRILLER'S TEST DATA:
- DATE
- STATIC WATER LEVEL 48.8 M
- PUMPING WATER LEVEL 80.5 M
- TEST PUMP YIELD 1.3 LPS

REMARKS:
- CASING DEPTH = 146.3 M
- PERFORATED CASING = 42.7 - 143.3 M
- OPEN HOLE = 146.0 - 152.4 M
- SPECIFIC CAPACITY = 0.04 LPS/M

GRAPHIC LOG

0.3 M — GROUND SURFACE



DEPTH (M)	DEPTH (FT)	STRATIFICATION
0.9	3	TOP SOIL
3.7	12	BROWN CLAY
		HARD ADOBE
10.7	35	BLUE SHALE
15.2	50	
		BROWN CLAY
22.9	75	COARSE SAND
24.4	80	HARD PACKED SAND
25.9	85	
		SANDY CLAY
42.7	140	
47.3	155	FINE SAND
50.3	165	
		BROWN CLAY
62.5	205	
		ADOBE
76.2	250	
		HARD ADOBE
81.4	267	SANDSTONE
83.8	275	CEMENTED GRAVEL
88.4	290	BLUE SHALE
91.5	300	BROWN CLAY
96.0	315	FINE SAND
98.5	323	ADOBE
100.6	330	HARD ADOBE
105.2	345	BLUE SHALE
109.8	360	SANDY CLAY
112.8	370	ADOBE
115.9	380	
		CEMENTED GRAVEL
123.5	405	HARD ADOBE
128.1	420	SANDY CLAY
134.2	440	CEMENTED GRAVEL
137.2	450	BLUE SHALE
141.8	465	FINE SAND
143.3	470	
146.0	479	SANDY CLAY
152.4	500	

APPENDIX FIGURE VII-B-6
WELL DATA SHEET
WELL LC-47

DESCRIPTIVE DATA

WELL NO. (CDM) LC-54
(OTHER) C-45
LOCATION BUGTONG NA PULO ELEMENTARY SCHOOL
CITY LIPA
PROVINCE BATANGAS
CONST. BY BPW
DRILLER
STARTED 1 JUNE 1960
COMPLETED 22 OCTOBER 1960
OWNER
STATUS
CASING DIAMETER 100 MM.

DRILLER'S TEST DATA:
DATE 22 OCTOBER 1960
STATIC WATER LEVEL 71.9 M. VARIABLE BECAUSE OF NEARBY PUMPING
PUMPING WATER LEVEL 75.4 M.
TEST PUMP YIELD 1.5 LPS

REMARKS:
PERFORATED CASING = 67.1 – 100.6 M AND 103.6 – 115.8 M
TOTAL PERFORATION = 45.7 M
TOTAL LENGTH OF CASING = 152.4 M.
GROUND SURFACE ELEVATION = 307.1 M.
TRANSMISSIVITY DERIVED FROM TEST DATA = 50 CUMD/M (APPROX.)
SPECIFIC CAPACITY = 0.4 LPS/M

WATER QUALITY DATA:

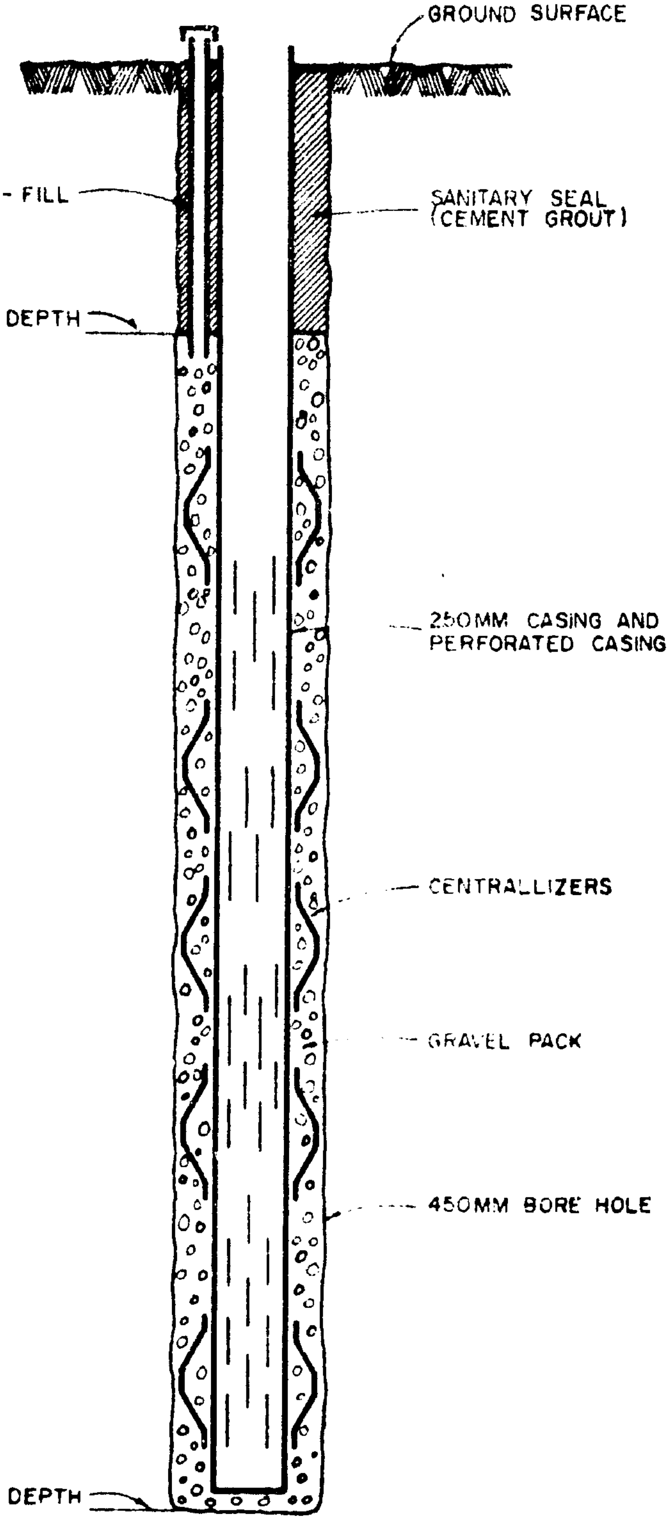
pH = 7.6	ALKALINITY = 166.00
ODOR = NIL	BICARBONATE = 168.36
COLOR = SLIGHTLY CLOUDY	ACIDITY = NIL
TASTE = BLAND	FREE CO_2 = NIL
TURBIDITY = 3.50	CHLORIDES = 10.0
$CaCO_3$ = 84.00	IRON = 0.20

PHENOLPH. ALKALINITY = 14.00
GOOD FOR DRINKING, LAUNDRY & BOILER
EXAMINED – OCTOBER 20, 1960

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
4.5	15	CLAY, LIMONITIC, YELLOW
15.2	50	TUFF, PUMICE W/ CLAY DARK GREENISH-GRAY
19.8	65	CLAY, LIMONITIC LIGHT OLIVE-GRAY
24.4	80	CLAY WITH GRAVEL YELLOW GRAY
35.1	115	SANDY CLAY YELLOW-GRAY
47.2	155	SILT, SLIGHTLY SANDY, LIGHT OLIVE-GRAY
56.4	185	TUFFACEOUS SAND, LIGHT OLIVE-GRAY
59.4	195	OLIVE-GRAY CLAY
67.1	220	
74.7	245	TUFFACEOUS SAND LIGHT OLIVE-GRAY
79.3	260	CLAY, SLIGHTLY SANDY YELLOW-GRAY
100.6	330	TUFFACEOUS SAND LIGHT OLIVE-GRAY
103.6	340	
115.8	380	
120.4	395	CLAYEY SAND TUFFACEOUS LIGHT OLIVE-GRAY
152.4	500	TUFFACEOUS SAND

APPENDIX FIGURE VII-B-7
WELL DATA SHEET
WELL LC-54



FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM - LWUA

APPENDIX FIGURE VII-B-8
GRAVEL PACKED TEST WELL
LIPA CITY

DESCRIPTIVE DATA

WELL NO. (CDM) TEST WELL
(OTHER)
LOCATION

CITY LIPA
PROVINCE BATANGAS
CONST. BY KATWELL, INC.
DRILLER R. KASTLE
STARTED 10 FEB. 1976
COMPLETED
OWNER

STATUS

CASING DIAMETER 250 MM

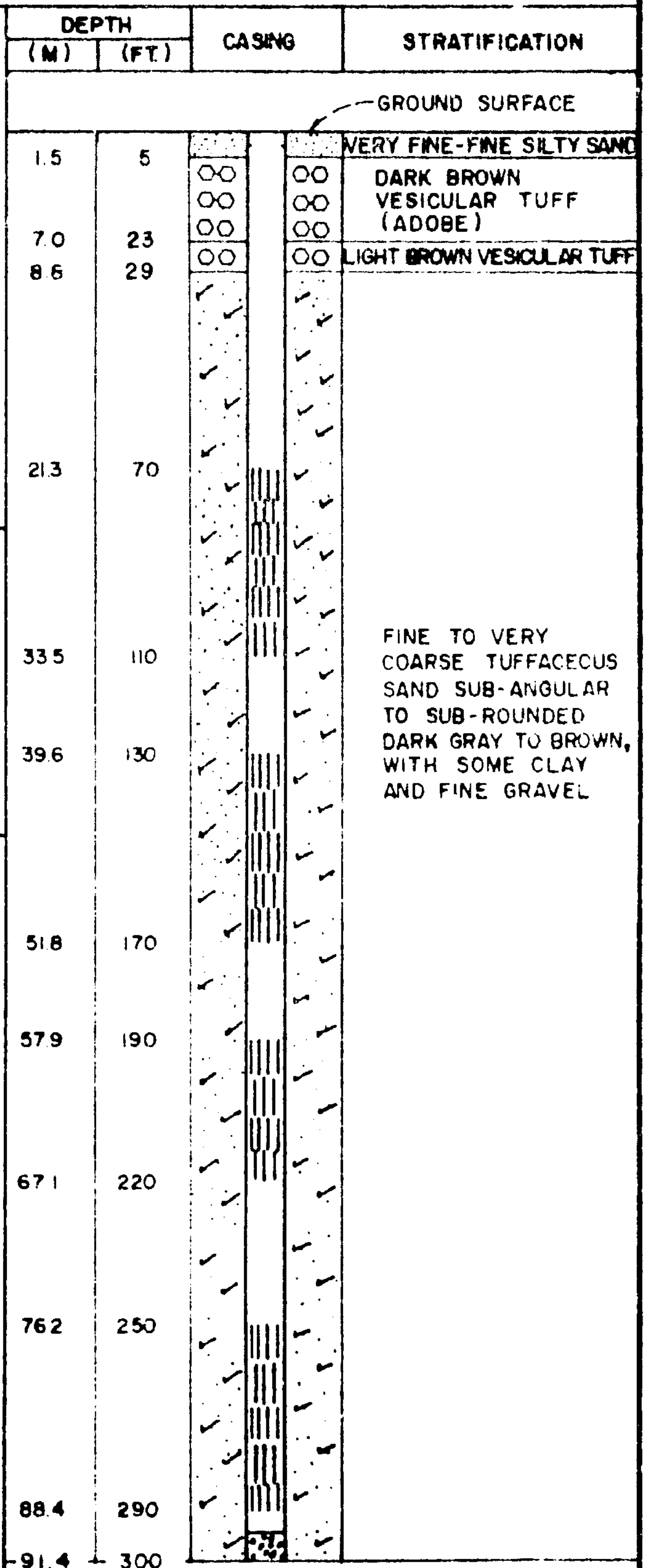
DRILLER'S TEST DATA
DATE
STATIC WATER LEVEL

PUMPING WATER LEVEL

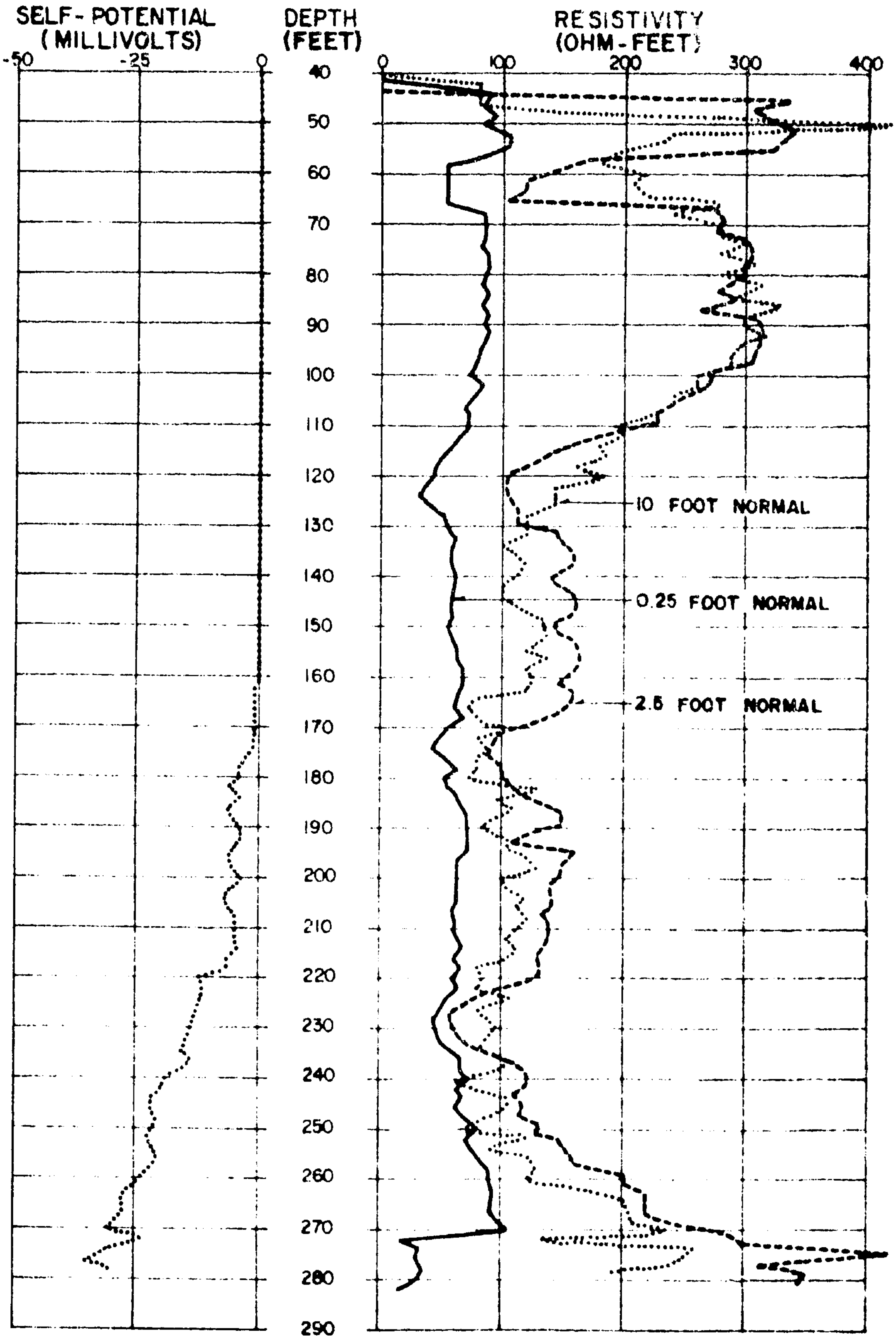
TEST PUMP YIELD

REMARKS:
CASING DEPTH = 89.0 M
PERFORATED SECTIONS:
21.3 - 33.5 M
39.6 - 51.8 M
57.9 - 67.1 M
76.2 - 88.4 M
GRAVEL PACKED
PERFORATED SECTION LOCATIONS DETERMINED FROM ELECTRIC LOG

GRAPHIC LOG

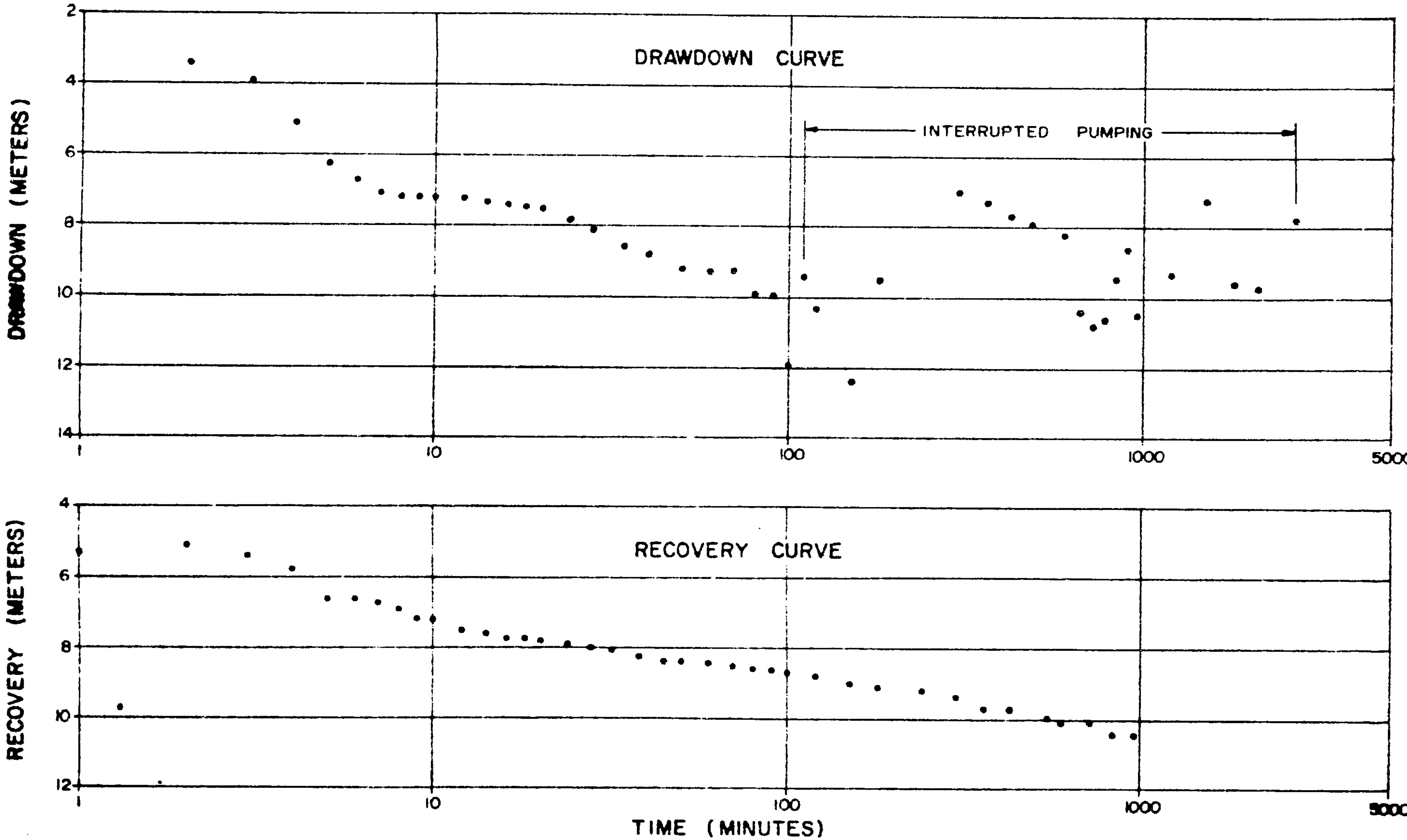


APPENDIX FIGURE VII-B-9
WELL DATA SHEET
LIPA CITY TEST WELL



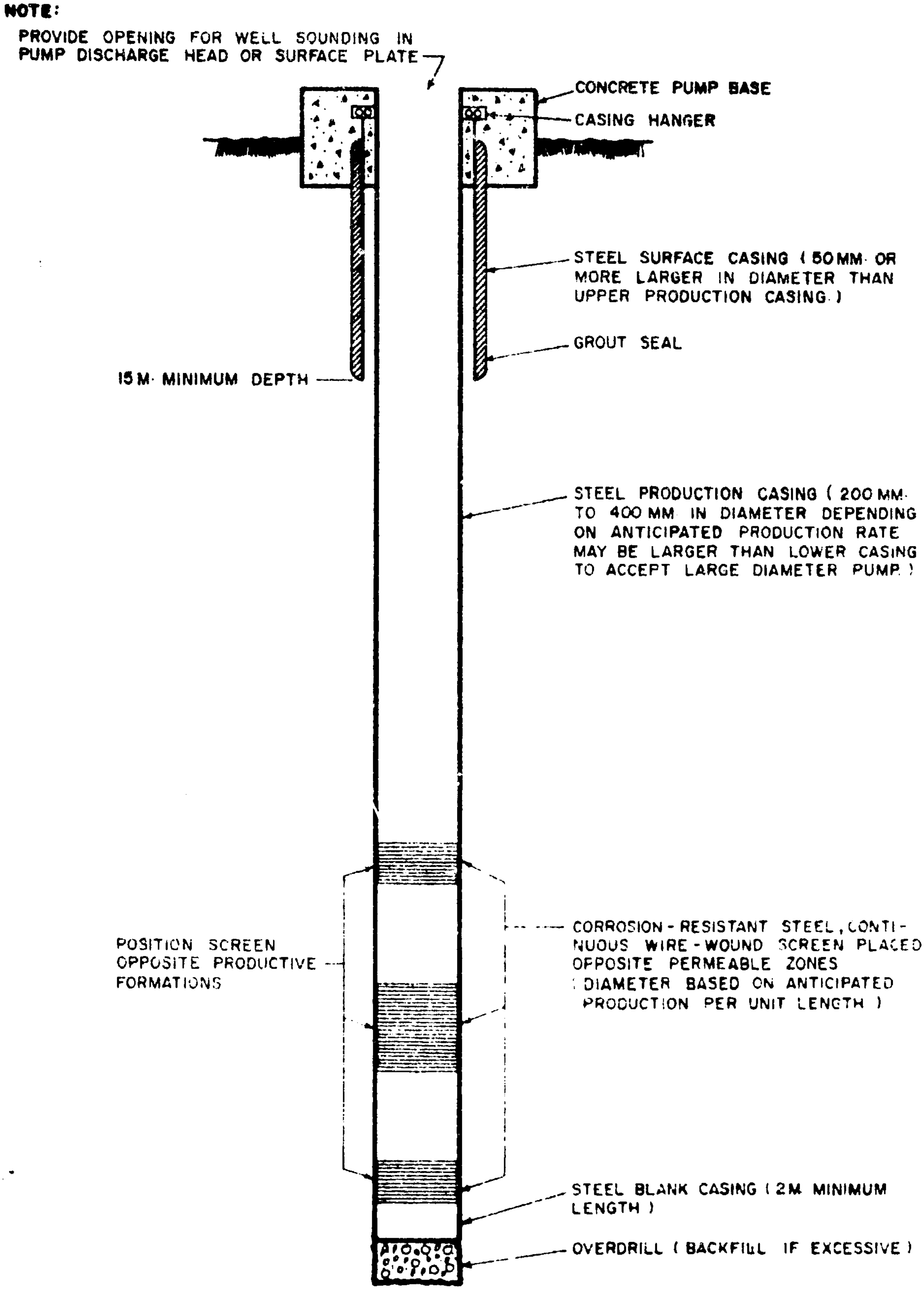
APPENDIX FIGURE VII-B-10
ELECTRIC LOG
LIPA CITY TEST WELL

FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS **CDM-LWUA**



APPENDIX FIGURE VII-B-11
DRAWDOWN AND RECOVERY IN TEST WELL

FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM-LWUA



APPENDIX FIGURE VII-B-12
GENERAL DESIGN
NATURAL DEVELOPMENT WELL
ROTARY DRILLED

SUPPLEMENT TO APPENDIX FIGURE VII-B-12
GENERAL CONSTRUCTION SUGGESTIONS

Natural Development Well - Rotary Drilled

1. Drill oversized hole to 15 m depth (more if conditions require) set and grout surface casing.

2. Drill clearance diameter hole inside surface casing to anticipated maximum total depth (or drill small-diameter pilot hole and enlarge it after logging).

3. Run electric log.

4. Examine samples and electric log to locate suitable permeable zones. Abandon site if sufficient permeable material is not found. Enlarge (ream) hole to accept casing if necessary.

5. Install string of casing and screen, with screen of slot suited to formation grain size opposite permeable zones.

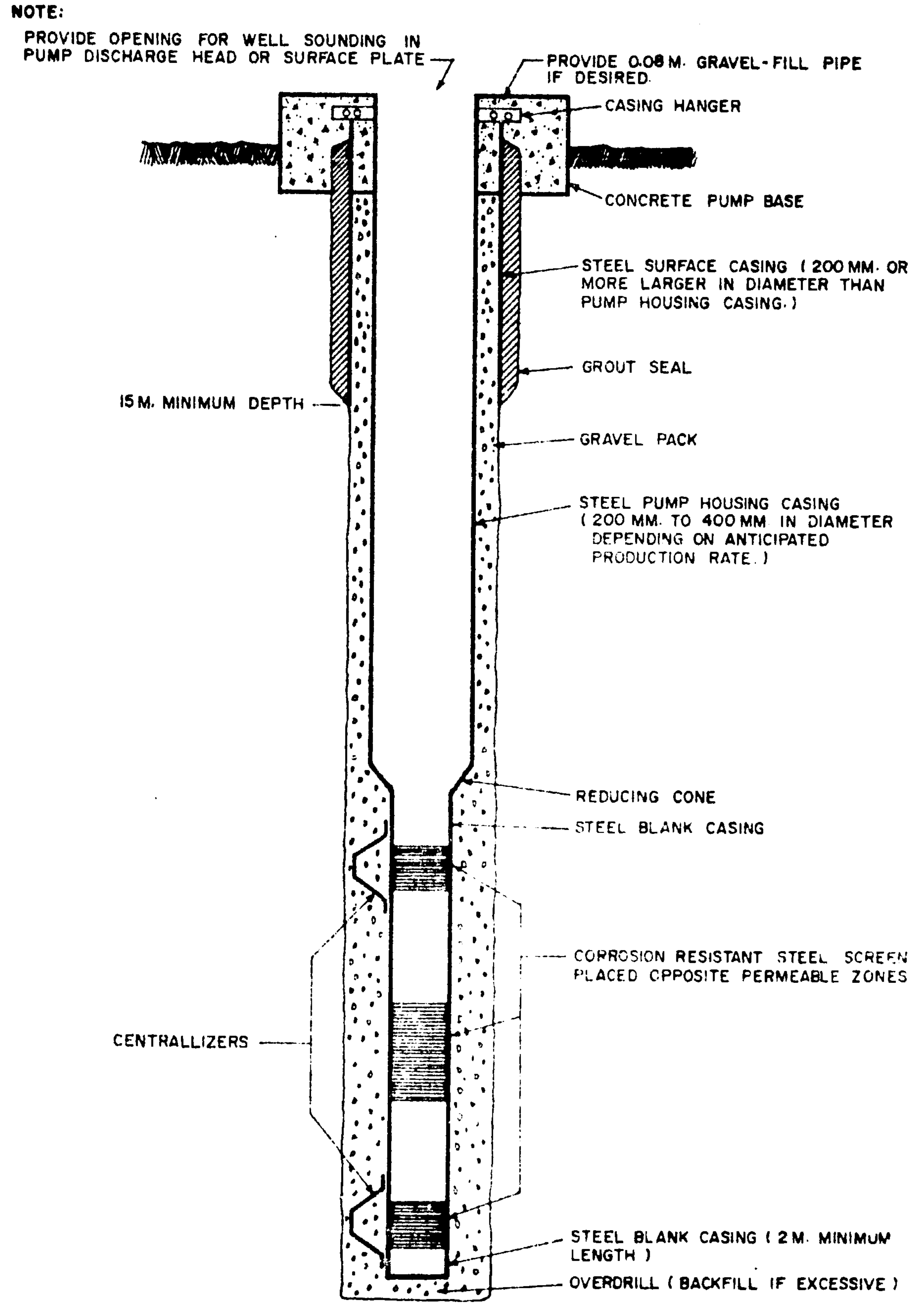
6. Clean and develop well thoroughly.

7. Test well.

8. Design pump.

9. Construct well head facilities.

10. Install pump.



APPENDIX FIGURE VII-B-13
GENERAL DESIGN
GRAVEL PACKED WELL
ROTARY DRILLED

SUPPLEMENT TO APPENDIX FIGURE VII-B-13
GENERAL CONSTRUCTION SUGGESTIONS

Gravel Packed Well - Rotary Drilled

1. Drill oversized hole to 15 m minimum depth (more if conditions require), set and grout 550 mm surface casing.

2. Drill small diameter pilot hole inside surface casing to 300 meters.

3. Run electric log.

4. Examine samples and electric log to locate suitable permeable zones. Abandon site if sufficient permeable material is not found.

5. Ream pilot hole diameter to largest diameter that can be drilled inside the surface casing to a depth about five meters below the lowest permeable zone.

6. Install string of casing and screen with screen opposite all permeable zones.

7. Place gravel.

8. Clean and develop well thoroughly.

9. Test well.

10. Design pump.

11. Construct well head facilities.

12. Install pump.

APPENDIX TABLE VII-C-1

MAXIMUM AND MINIMUM DAILY FLOWS
PANSIPIT RIVER

Year	Maximum Flow (lps)	Maximum Date	Minimum Flow (lps)	Minimum Date
1958	16,100	2[illegible] October	-	-
1959	11,150	18 November	1,460	22-23 June
1960	44,420	14 October	5,950	22 April
1961	25,140	13 October	8,250	5 May
1962	63,400	6 September	7,530	16-18 May
1963	24,900	2 October	8,170	16 May
1964	35,660	29 November	6,520	29 May
1965	34,200	28 September	10,820	23 May
1966	31,880	19-21 September	4,190	5 May
1967	49,530	10 November	6,140	16 May
1968	23,190	17 October	5,390	5 June
1969	14,520	24 October	5,390	16 May
1970	19,770	23 October	4,510	7-8 May
1971	30,530	12 October	7,080	26 April
1972	42,080	3 August	12,900	22 May

APPENDIX TABLE VII-C-2

TOTAL MONTHLY STREAM FLOW
(IN MILLION CUBIC METERS PER MONTH)

LOCATION: PANSIPIT RIVER, POBLACION SAN NICOLAS, BATANGAS
DRAINAGE AREA = 644 sqkm

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JUL	AUG	SEPT	OCT	NOV	DEC	TOTAL ANNUAL
1958											33.36	26.57	
1959	22.04	14.14	13.01	8.02	6.06	4.46	11.93	19.28	22.42	24.37	25.23	26.78	198.24
1960	28.64	22.45	20.92	16.76	18.72	26.37	37.25	50.64	60.41	86.72	69.84	56.24	494.96
1961	42.72	30.88	27.89	23.36	24.05	28.45	54.27	53.33	60.26	65.13	56.37	52.55	519.26
1962	41.00	27.10	24.99	22.68	21.33	23.19	33.30	55.75	136.08	110.40	69.98	56.41	622.21
1963	40.65	29.62	29.78	25.35	22.91	26.57	33.68	43.81	58.92	53.26	38.52	45.00	448.13
1964	38.19	29.22	27.48	20.76	18.41	22.34	40.02	50.08	51.44	66.11	68.75	83.70	516.5
1965	51.31	36.61	34.14	29.95	30.91	32.57	38.73	48.74	56.15	54.07	43.48	37.15	493.81
1966	29.22	18.91	14.66	12.90	21.04	31.60	35.89	39.46	57.64	58.37	47.95	52.02	419.66
1967	55.93	33.29	27.22	20.18	18.17	27.01	37.49	46.17	64.75	68.13	104.39	64.82	567.55
1968	40.05	29.03	23.77	18.31	15.39	17.03	24.19	36.69	36.36	53.03	45.34	41.86	381.05
1969	33.27	20.56	18.81	17.85	15.04	14.93	21.30	30.83	33.14	36.21	28.95	28.87	299.76
1970	85.38	17.81	16.33	12.74	12.53	14.66	21.85	25.20	31.22	39.75	44.74	49.65	311.86
1971	31.52	21.03	23.56	20.06	22.53	29.48	47.40	73.00	63.74	70.51	64.49	-	467.33
1972	59.17	48.28	44.63	37.93	36.33	36.88	57.93	106.35	96.84	85.18	72.48	65.38	747.00
AVERAGE:	38.5	27.07	24.80	20.49	20.24	23.97	35.37	48.52	59.24	62.27	55.75	47.18	463.4

APPENDIX TO CHAPTER VIII

APPENDIX VIII-C

WATER TREATMENT ALTERNATIVES

Disinfection Alternatives

Disinfection of water supply may be accomplished through chemical application of chlorine, iodine, ozone, ultra-violet radiation and oxidizing agents.

Chlorination is a universal disinfection process used in most municipal water systems. Chlorine, a potent oxidizing agent, destroys bacteria when mixed under certain time constraints and when applied in correct dosage.

Iodine has chemical properties that make it an effective agent against virus and certain bacterial cysts. However, research indicates that iodine treatment in excess of three weeks may have detrimental effects upon individuals afflicted with thyroid diseases.

Ozone, a blue gas and active form of oxygen, is rated to be a more vigorous oxidizing agent than chlorine. This versatile element not only disinfects but also sterilizes. It also helps in color reduction, iron and manganese oxidation, taste and odor control.

Despite its impressive known qualities, ozone is yet to achieve universal acceptance.

Ultra-violet radiation is another method of disinfection applicable to small water systems. It involves high-cost equipment and requires considerable amount of power. Moreover, this type of treatment requires high quality water, otherwise the ultra-violet rays may be absorbed by substances present in the water supply.

The use of metal ions with bactericidal properties such as copper, silver and mercury is limited by their cost, availability and potential adverse health effects if not properly dosed.

Bromine as a water disinfectant is costly and scarece. Liquid bromine produces irritating fumes and causes severe burns.

Oxidizing agents such as potassium permanganate and hydrogen peroxide have weak purifying qualities that require long contact time and high dosage.

The economics of disinfection serve as an important, if not sole, basis for the selection of a treatment method suitabl to a particular water system. Selection is not necessarily based upon the cheapest method available but on its dependability, effectiveness, suitability and reasonableness in cost. From this viewpoint, ozone and chlorine merit further consideration.

Ozone, as earlier indicated, lacks extensive practical application but its versatility makes it advantageous over chlorine under certain conditions. It can be more effective and economical when used for two or more stages of water purification. When taste and odor in water are organic, ozone may be as effective as chlorine. When disinfection only is required or water supply is clear, however, chlorine will be much more economical.

Plant-scale studies on ozonation show that it entails bigger capital investment than chlorination by the ratio of 3 or 4 to 1.

While ozone appears to be an efficient disinfectant, its practical application is supported with scarce data. This leaves chlorine, a proven disinfectant, as a more dependable method. Although considered a less rapid agent than ozone, chlorine fits well in large water supply systems.

Since the early 1990's, chlorine has been widely used in water treatment but recently in the United States, it has developed into a critical issue. Studies done by regulatory agencies revealed the presence of cancer-producing chlorine compounds in the drinking water of several cities in the eastern part of the United States as a result of treating river waters contaminated by certain organic and chemical wastes. The studies indicated that through chlorination, the hazard levels of man-made chemicals and pesticides that pollute the river sources are increased.

However, the critical aspect of chlorination does not apply to the study area at this period of its development. Rivers are not generally contaminated by agro-industrial chemicals, a condition foreseen to remain for quite some time.

APPENDIX VIII-D

DISTRIBUTION SYSTEM ALTERNATIVES

General

The distribution system, in general, is composed of a network of distribution mains, internal distribution networks, storage facilities, booster pump stations, booster chlorination stations, and appurtenances such as valves, fire hydrants, meters, and service connections. The distribution mains are the larger pipelines which take the water from the transmission lines to the demand areas. The internal network system consists of the smaller street mains which distribute the water to consumers along smaller streets of the city and subdivisions. Booster pump stations are required to raise water from lower pressure zones to higher pressure zones where consumers are usually at higher ground elevations. The booster chlorination stations are required at the fringe areas of the water district to keep the chlorine residual at the desired concentration. The distribution storage facilities provide supplementary flows during the peak-demand periods. The transmission lines convey the water to and from the storage facility depending on whether it is filling or emptying.

The valves are placed throughout the distribution system to keep small service areas isolated by closing the valves at times when maintenance is required. The fire hydrants are connected to the distribution system at regular intervals depending upon the type of area served. The service connections convey the water from the internal distribution system to the consumer. Meters are placed on the service connection line to measure the amount of water consumed by the customer. The components of the distribution system described above are illustrated in Appendix Figure VIII-D-1.

The major alternatives for the components of the distribution system can be grouped into two categories:

(1) Size and Staging. For most components of the distribution system it is possible either to install the capacity required for the design year or to stage the construction of the component by installing part of the required capacity in an early construction phase and the remaining capacity in a subsequent phase. Examples would be: a 10,000 cum storage tank built in 1980 for the design year 2000; or a 6,000 cum storage tank built in 1980 and a 4,000 cum storage tank built in 1990. Installing a smaller size component initially has the advantage of reducing capital cost in the initial construction period. Also, staging provides

flexibility as more data will be available at a later date and the assessment of population and economic growth may indicate a new location is preferable to that originally planned. In any case, studies should be made to indicate the economic feasibility of staging.

(2) Location. Sometimes, more than one location exist for the construction of the distribution system component. In some cases, economic studies will aid in the selection of the most desirable site, and in other cases, practical consideration and engineering judgement will be of primary importance.

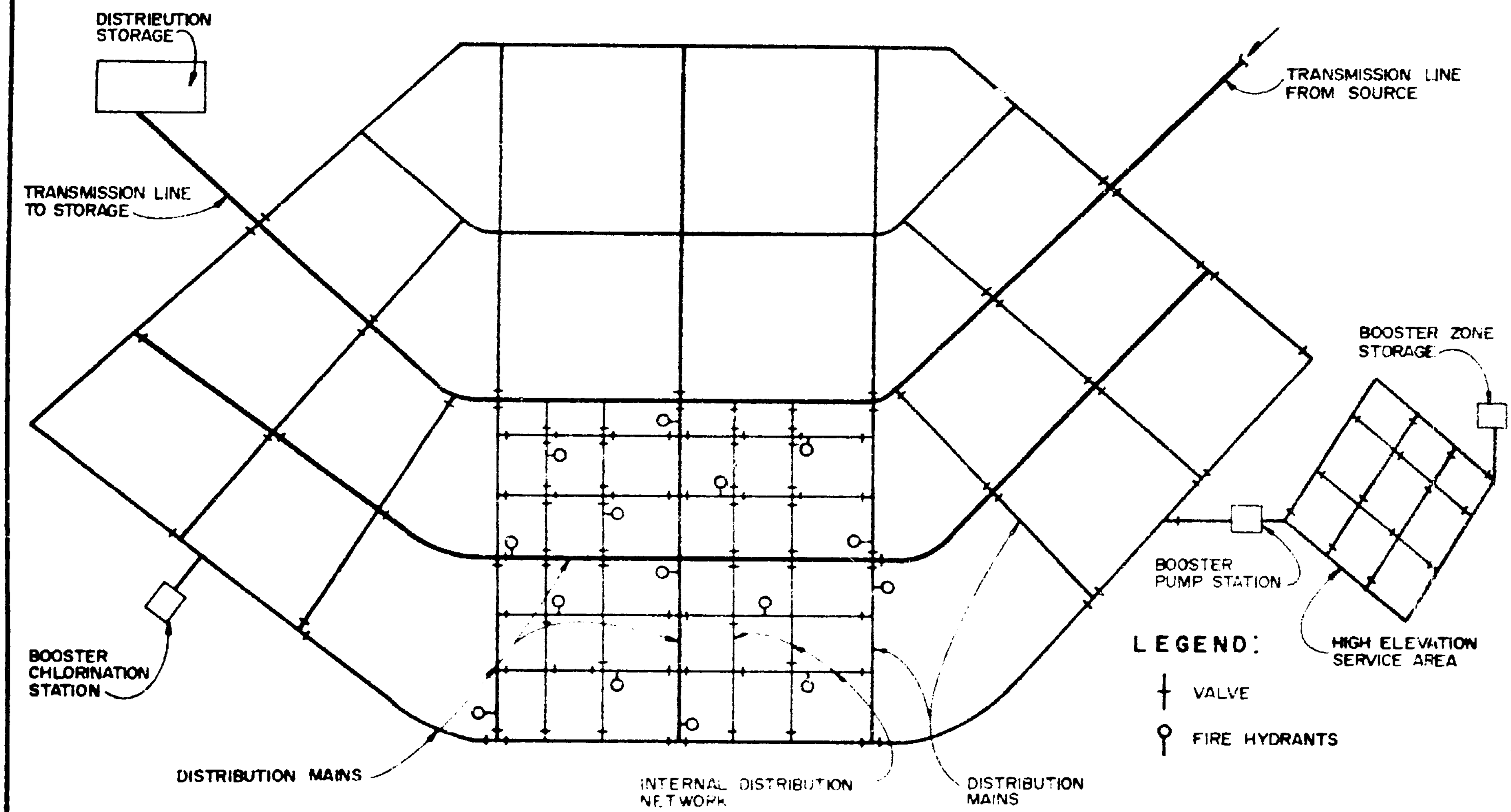
Each component of the distribution system and its respective alternatives are discussed in subsequent sections.

Distribution Mains

The alternatives for distribution mains are location, size, staging and the spacing in the network. To avoid land costs and also to place the mains as close to the demand center as possible, the alignments for future distribution mains should be chosen along existing and planned road and street rights-of-way. Where the service area will extend to areas without planned or existing roadways and streets, the location of the distribution mains is determined by topographic features. As much as possible, the distribution mains should be looped to avoid dead-end service areas; to minimize the number of concessionaires affected when valves are closed for maintenance; and to provide adequate pressure at times of maximum demand as the demand can be supplied from more than one direction.

The distribution main network system is designed to provide a minimum pressure of 14 m during peak-hour conditions. The minimum size of distribution mains has been taken as 200 mm. In general, this size is large enough to provide adequate pressure during peak-hour and fire-flow conditions. In some residential areas, alternating 150 mm and 200 mm pipe sizes is adequate. Staging of distribution mains is economical at 10-year intervals in areas having wide streets and low population densities. However, in high-density areas having small streets, it is usually preferable to avoid two-stage construction. There is limited space for utilities in these areas and considerable disruption occurs when the street is excavated for the new water main. It is better to install the pipe size required for ultimate design in these congested areas so that these problems can be avoided.

It is desirable to maintain the maximum spacing for distribution mains at 1,000 meters. This will provide uniform size and spacing



APPENDIX FIGURE VIII-D-I

COMPONENTS OF THE DISTRIBUTION SYSTEM

NOT TO SCALE

for the internal mains as well as better pressure distribution throughout the system. A wider spacing of the distribution mains would require larger pipe sizes in the internal distribution network to maintain sufficient pressures during fire-flow and peak-hour periods.

It is not recommended that the minimum distribution main sizes be staged. However, larger mains can be staged in some instances. A required pipe size of 250 mm for year 2000 demands can be conveniently staged with one 200 mm line in Stage I and another 200 mm parallel line in Stage II. However, in Stage II an extra cost of 15 per cent may be included in the construction of the parallel line because of the problem encountered with interties to the Stage I line and safeguarding service connections and sometimes transferring the connections with the internal network. The economic evaluation of a two-stage versus one-stage construction of a 250 mm line is shown below:

EVALUATION OF DISTRIBUTION MAIN STAGING

						1976 Present Worth[2]			
Alternative	Construction Period	Pipe Size (mm)	Construction Cost[1] (P/m)	Project Cost (P/m)	Annual Cost (P/m)	Capital Cost (P/m)	Annual Cost (P/m)	Salvage Value (P/m)	Net Cost (P/m)
Single-Stage	1980	250	475	648	3	412	14	19	407
						Total			407
Two-Stage	1980	200	360	491	2	312	9	14	307
	1990	200	414	565	2	116	2	19	99
						Total			406

Comparison of the two alternatives shows that constructing two parallel 200 mm lines in each stage costs almost the same as a single 250 mm line in Stage I. In this case, it would probably be better to install a 200 mm line initially because of the lower capital cost and added flexibility. Similar calculations for staging a 300 mm line with parallel 250 mm and 200 mm lines indicate only slight savings with two-stage construction. Selection of distribution mains which should be staged must follow an analysis of the peak-hour and fire

[1]1990 construction cost includes 15 per cent penalty.

[2]Discount rate is 12 per cent.

flow conditions to be sure that the smaller line constructed in Stage I will be hydraulically adequate until the second line is installed.

The timing of the construction of the distribution main systems should be such that an attainable level of growth in the distribution system is maintained. Areas having higher densities of potential customers should be connected in the early construction periods because the cost per connection will be lower and more revenues will be generated. Also, extension of service to large demand customers such as industries and commercial areas would be desirable when a reliable water supply is available. Service to this type of customer would have a positive impact on the economy of the study area.

Distribution Storage Tanks

Distribution storage tanks provide supplementary supply during peak-hour demand periods, during fire-flow demand periods and during emergency periods when source supply is reduced. The recommended distribution storage volume is 15-20 per cent of maximum daily supply requirements. The storage facility is designed to empty during peak-hour demand periods and to fill overnight during minimum demand periods. The storage tanks should be located as close to the demand center as possible and on the opposite side of the service area from the source. By locating the storage in this manner, the peak-hour pressures will be higher as the supply can be provided from two directions.

It is recommended that storage facilities be constructed on-grade with an operational level fluctuation of 3 to 7 m. The storage tanks should be of reinforced concrete and covered to prevent contamination. Initially, adequate land area should be purchased so that the ultimate storage capacity of the site can easily be accommodated. The storage facility is designed and constructed in increments so that the desired capacity is available when needed. It has been observed that staging at 10-year intervals is an economically appropriate time increment based on the discount rate used in this study.

For operational purposes the storage overflow elevation should be the same elevation as the HGL control at the source. Locating the storage at the same elevation as the source is sound engineering practice. The range of operating pressures within the distribution system is reduced. This keeps the pumping heads at booster stations and wells at more constant levels, simplifying operation of the pump station. No maintenance of double-acting altitude valves at the storage facility is required unlike when the storage is at a lower elevation than the source.

Tank filling will take place during the minimum demand periods. Amount and duration of minimum demand can be determined by 24-hour consumption records. Since these data are not available, it is assumed that the minimum demand is about 30 per cent of the average demand for a period of 8 hours. Assuming a tank with 7 m water depth, the differential head between the source HGL elevation and the storage tank is a maximum of 7 m when the tank is empty and 3.5 m when the tank is half full. Because of this small head differential, care must be taken in choosing location and size of the supply lines.

Placing the storage HGL at an elevation lower than 70 m is not recommended because this will mean that areas at the extreme ends of the distribution system will have insufficient pressures unless inordinately large distribution mains are provided. If locating the distribution storage tank at a lower elevation than the source is considered, a double-acting altitude valve must be placed on the supply line to the tank. The valve closes when the water elevation in the tank reaches the overflow level and opens when the pressure drops in the distribution system, permitting water from the storage to enter. If the valve is not maintained at all times, it could fail to operate properly and cause lower pressures in the distribution system than required. Because the storage is at an elevation less than the source, it is difficult to obtain the required flow from the storage during peak flow demands as most of the supply will come from the direction of the source, the location of the highest HGL.

When suitable ground storage sites are not available, it is possible to utilize elevated storage tanks or standpipes. If possible, the overflow elevation should be the same as the HGL control on the source transmission line. The operational range of elevated storage may be reduced to 5 m. In the case of standpipe storage the volume lower than the top 7 m should not be considered as part of the operational volume. Economic studies can aid in the selection of the best location. The present worth cost of the storage tank and the storage transmission line for several alternative sites should be evaluated to determine the least-cost alternative.

In some cases it is more economical to locate a portion of the distribution storage volume at the source HGL control. This reduces the pipe diameter required to fill the distribution storage tank located at the other end of the system. However, locating storage at the source will mean that more supply must come from the source during peak-hour demand periods. Several alternative distribution and source storage schemes should be evaluated to determine the best apportionment of the required storage volume.

Internal Network System

The internal network system is the network of pipes within the 1,000 m grid spacing of the distribution main network. The internal network consists of pipe sizes usually of 150 mm or smaller diameter, valves, fire hydrants and service connections. The alternatives in the internal distribution network are dependent on the level of water service provided. A system designed for fire flow demands may require larger internal distribution pipes than a system designed only for peak-hour demands. The fire-flow requirements are:

Type of Area	Fire-Flow Demand
Commercial, Industrial and High-Value Residential	20 lps at each of two adjacent fire hydrants
Single Family Residential	10 lps at each of two adjacent fire hydrants

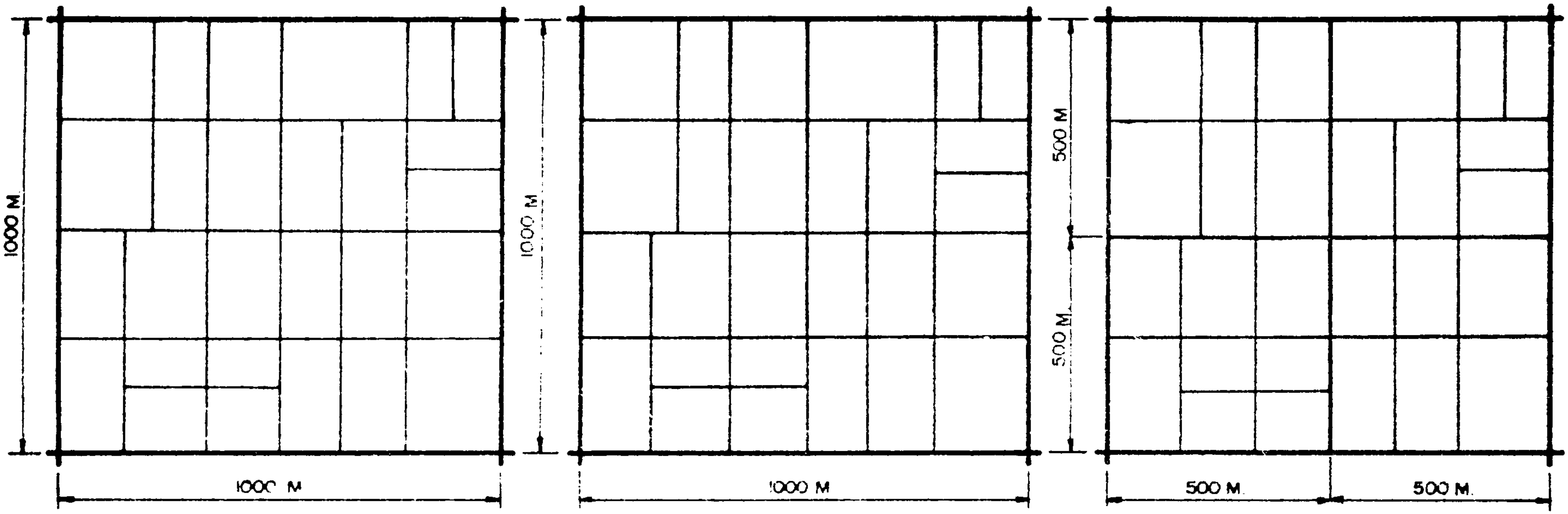
The internal network design is controlled by either of two conditions: peak-hour demands with minimum main pressure of 10 m or fire-flow demand coincident with maximum-day demands with a minimum hydrant pressure of 7 meters.

In order to determine the response of the internal network to several flow conditions, a detailed study of the internal network was made. Commercial/residential areas in several cities in the Philippines were analyzed to determine a composite 100 ha area. A typical 100 ha area in the core city or fringe of the core city has 8-12 km of roads and streets. Since the internal network is installed along street rights-of-way, the total length of internal network pipe will also be 8-12 km for a 100 ha area. An average of 8-12 km per 100 ha area corresponds to 80-120 m of pipe per hectare served. The 80 m per hectare would be in less densely populated, high-value residential areas, and the 120 m per hectare would be in densely populated, mixed residential and commercial areas. The 80 m of pipe per hectare was used in the design study in order to evaluate the internal network under the most stressing conditions - less pipe per hectare will cause higher flows for the same areal demand. Three alternative internal network designs were studied. The three alternatives are shown in Appendix Figure VIII-D-2 and listed below:

Alternative 1. All internal network pipe is 100 mm in diameter.

Alternative 2. All internal network pipe is 150 mm in diameter.

Alternative 3. The ratio of 100 mm pipe to 150 mm pipe is 3:1, i.e., 6,000 m of 100 mm pipe and 2,000 m of 150 mm pipe.



ALTERNATIVE ONE   ALTERNATIVE TWO   ALTERNATIVE THREE

SIZE OF AREA = 100 HA
TOTAL LENGTH OF INTERNAL NET WORK PIPE = 80 M/HA
DISTRIBUTION MAIN SPACING = 1000 M

INTERNAL NETWORK CHARACTERISTICS

	LENGTH OF 150 MM PIPE	LENGTH OF 100 MM PIPE
ALTERNATIVE ONE		8000 M
ALTERNATIVE TWO	8000 M	
ALTERNATIVE THREE	2000 M	6000 M

LEGEND:

DISTRIBUTION MAIN
150 MM PIPE
100 MM PIPE

APPENDIX FIGURE VIII-D-2
ALTERNATIVE INTERNAL NETWORK SYSTEMS

The three alternative systems were evaluated by using the computer to solve for pressures and flows for varying population densities. The 100 ha area was assumed to have a mixed residential and commercial land use. The domestic flow requirement was assumed to be 175 lpcd, and the commercial and institutional demand was assumed to be 10 per cent of the domestic demand. The unaccounted-for-water was assumed to be 25 per cent of the area's total demand. The demand was applied uniformly over the entire area. The alternative networks were analyzed under peak-hour condition (peaking factor of 2.0) and maximum day plus fire-flow condition. The minimum pressure in the internal network is listed in Appendix Table VIII-D-1.

APPENDIX TABLE VIII-D-1

MINIMUM PRESSURE IN ALTERNATIVE INTERNAL NETWORK SYSTEM

Alternative System	Population Density	Minimum Pressure (m)[3/] Peak Hour	Commercial Fire Flow	Residential Fire Flow
1 - All	100/ha	11	7	11
100 mm Pipe	200/ha	10	6[4/]	10
	300/ha	8[4/]	4[4/]	8
2[5/] - All	100/ha	11	12	
150 mm Pipe	200/ha	11	11	
	300/ha	11	11	
3[5/] - Ratio	100/ha	11	8	
of 100 mm to	200/ha	11	8	
150 mm is 3.0	300/ha	10	7	

[3/] Average pressure in distribution mains is 14 m.

[4/] Less pressure than the criteria: Peak-hour minimum is 10 m; fire-flow minimum is 7.0 meters.

[5/] No residential fire test was analyzed because the minimum pressure criteria were satisfied in the commercial fire test.

The data indicate that Alternative 1 can meet minimum pressure requirements for all conditions for a population density of 100 people per hectare. However, minimum criteria cannot be maintained for higher densities. Alternative 2 meets the pressure criteria for all population densities studied. Alternative 3 also satisfies the minimum criteria for all population densities studied.

The only difference between Alternatives 1 and 3 is the two 150 mm lines which are placed in the middle of the 100 ha area. The two 150 mm lines add considerable carrying capacity to the internal network as indicated by the data in Appendix Table VIII-D-1. Alternative 3 can meet minimum pressures during commercial fire-flow test. Even though a 150 mm fire hydrant might be connected to a 100 mm pipe, it is possible to support the commercial fire-flow because water can be supplied from at least two directions and the larger, supporting 150 mm main or distribution main is no more than 250 m away.

Other computer studies were carried out on Alternatives 1 and 3 by increasing the total length of internal network pipe to 120 m/ha. The peak hour pressures were increased approximately 2 m as the peak hour flow was spread among more pipes. However, the fire-flow pressures increased only slightly as the fire-flow was applied to a much smaller area of the system.

The construction costs for installing the internal network piping for each alternative, and several sub-alternatives of total internal network piping are listed below:

Alternative	Length of Pipe m/ha	Ratio of 100 mm : 150 mm	Construction Cost (P/ha)[6]
1 - All	80	80:0	6,400
100 mm Pipe	100	100:0	8,000
	120	120:0	9,600
2 - All	80	0:80	12,000
150 mm Pipe	100	0:100	15,000
	120	0:120	18,000
3 - Mixed	80	60:20	7,800
100-150 mm Pipe	100	80:20	9,400
	120	100:20	11,000

[6] Costs do not include valves or fire hydrants.

Alternative 3 is 15–22 per cent more than Alternative 1 and 35–39 per cent less than Alternative 2. Though Alternative 1 provides sufficient service for residential areas up to 200 people/ha, Alternative 3 is the recommended internal network system. Alternative 3 can meet minimum pressure requirements for higher density levels and during commercial fire-flow conditions. This alternative can serve an area which is initially residential but gradually becomes commercial without requiring reinforcement. Also, further tests indicate that Alternative 3 can meet minimum pressures up to 400 people/ha when 120 m/ha of internal network pipe is required.

The minimum recommended pipe size in the internal network is 100 mm. Smaller pipe would reduce peak-hour pressures and would severely limit fire-fighting potential. Though pipes less than 100 mm are cheaper, the installation costs are not significantly less than that for 100 mm pipe. Also, the capacity of smaller pipe is considerably less. The installation price per unit of capacity is shown in Appendix Figure VIII-D-3 for pipe sizes ranging from 50 to 350 mm. The curve turns upward very sharply for pipes smaller than 100 mm. On a capacity basis, the 75 mm pipe costs 80% more than the 100 mm pipe and is thus not recommended for internal network pipe.

Staging of internal network pipes is not usually economical or practical. Streets and utilities should be provided in accordance with development and zoning plans. The internal network should be designed with the ultimate plan of the area, fixing the required demands and fire flows. Thus, the period for staging to be economical would have to be 10 years or more.

The valves in the distribution main network and the internal distribution system should be spaced so that interruption in service due to maintenance would affect as few customers as possible. Each internal network connection to a distribution main should have a valve so that repair to an internal main would not require closing of valves on distribution mains. High-demand areas should have more frequent valve spacing than low-demand areas. Valves should normally be placed at street intersections with a minimum spacing of 300 m to 500 m depending on the character of the area.

Fire hydrants will be placed at street intersections to permit quick location and enable fire-fighting in several directions. In high-value areas, hydrants will be spaced a maximum of 150 m apart with two outlets – 1 x 60 mm hose outlet and a 1 x 100 mm pump outlet. The hydrant will be connected to a main with a minimum diameter of 100 mm. In single-family residential areas, hydrants will be connected to 100 mm mains and spaced a maximum of 250 m apart. The outlet will be connected to a 1 x 100 mm pump or hose.

Customer service connections consist of a connection to the internal network main and a service line to the customer. The service line will be provided with a "gooseneck" for connection to the main and a service meter will be provided for continuous measurement of water provided to the customer. A valve should be inserted in the service ahead of the meter to enable the Water District to terminate service when it becomes necessary.

Booster Zone

Portions of the service area at elevations which are too high to be served from the same HGL control as the service area at lower elevations, must be served by booster pumpage. The booster pump station raises the HGL to sufficient levels to serve the concessionaires in the booster zone. A storage tank should be located in the booster zone to supply peak-hour demand. The storage tank should be located on the opposite side of the boostor zone from the booster pump station. The booster zone storage should have an overflow elevation no more than 70 m above the lowest ground elevation in the booster zone. The booster pump station should be designed to pump maximum day supply requirements.

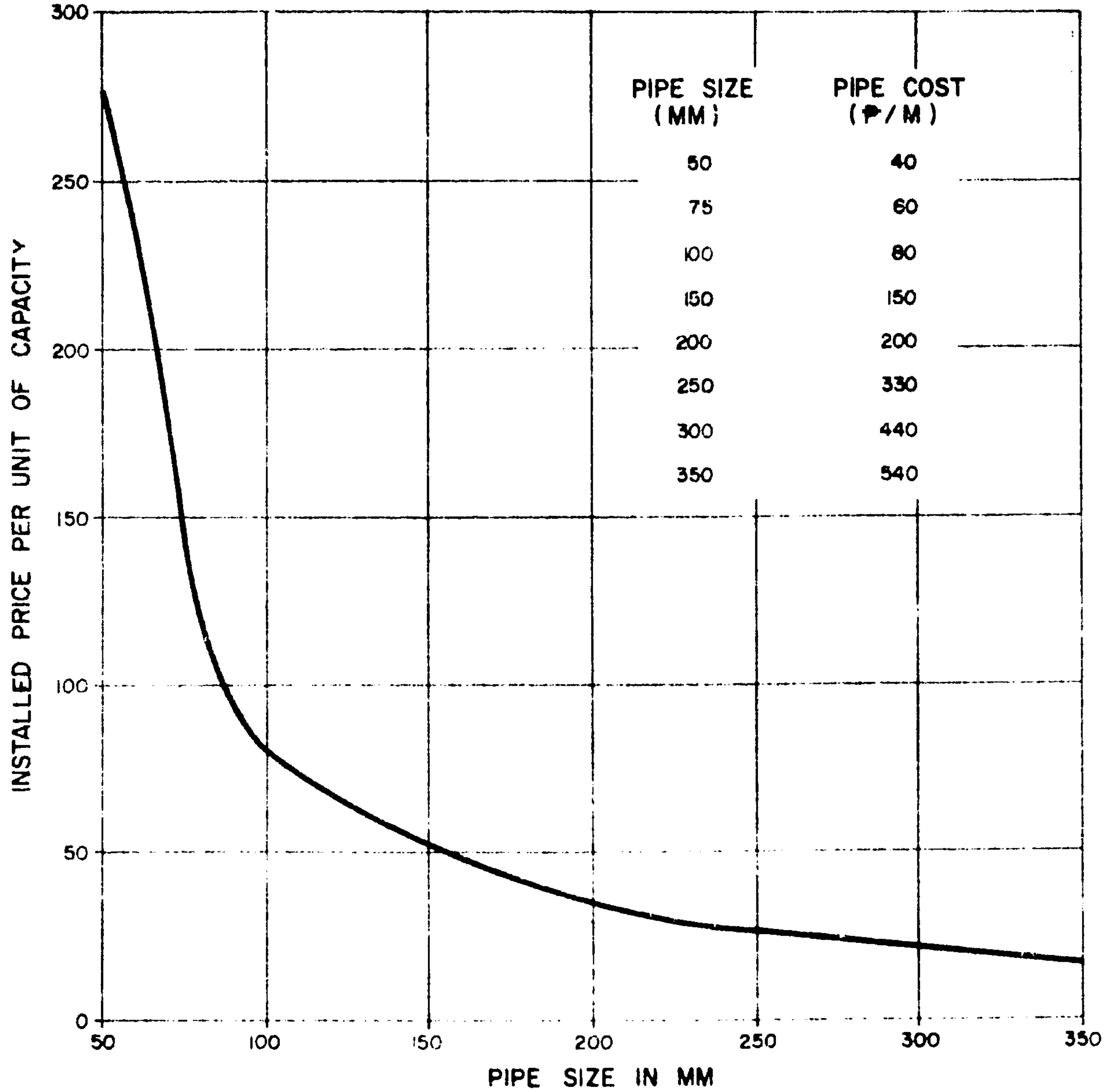
Booster pump station should have at least two pumps to permit maintenance without interrupting service. One of the units should be diesel-powered to permit a minimum level of service during power shortages.

Hand Pump Wells for Urban Areas

An alternative to a piped water system in urban areas is hand pump wells (HPW). Under existing conditions in the Philippines, HPW will probably provide, at the pump, drinking water not significantly less safe than a piped water system. Water from the HPW may be contaminated while being carried from the pump to the point of use. In this respect, safety of the piped water is not guaranteed if also carried.

Benefits from personal and domestic hygiene occur from any water system in proportion to the amount used. This amount depends primarily upon the convenience with which water is provided. A HPW in the yard immediately adjacent to the house, or in the house, will ordinarily be found reasonably convenient to use.

Water from a HPW is ordinarily not used in adequate quantity to support a sanitary sewer system and would not otherwise be very helpful to public or neighborhood cleanliness. HPW is, in this respect, inferior to a piped water system. This specific advantage



NOTE:
CAPACITY OF 100 MM PIPE TAKEN
AS UNITY. CAPACITY CALCULATED USING
SAME HEADLOSS FOR ALL PIPE SIZES.

FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM-LWUA

APPENDIX FIGURE VIII-D-3
INSTALLED PIPE COSTS PER UNIT OF CAPACITY

of a piped water system over HPW is less important if there is no sanitary sewer system, or if the urban area in question does not have the funds to provide private water-borne waste system as substitute for the public sanitary sewer system.

Similarly, water from a good piped water system is ordinarily much more convenient and useful for non-essential uses than water from a HPW. A hand pump well is much inferior to a good piped water system for fire prevention.

In summary, water supply from HPW does not have the advantages of a piped system installed in the house. Water from a convenient HPW (which means a HPW adjacent to the house) is usually preferable to water from a piped system located away from the house. A piped water supply not available continually during at least the day-time hours or available only at a distance from the house is generally inferior to a HPW located in the premises.

Therefore, the HPW may provide a valid alternative to the piped water system in certain urban areas if funds are not sufficient for a modern upgraded piped water service.

<u>Types and Costs of HPW</u>

Hand pump wells may be classified in two categories:

1. Water level high enough for suction pump use (within 6 to 8 m below ground surface);

2. Water level too deep for suction pump use (below 8 m).

The depth of well required to reach a good aquifer at any particular site is a critical factor which influences costs.

<u>High Water Level HPW</u>. A high water level HPW includes the following components:

1. A simple pitcher pump, which should be brass lined and connected to a 32 mm pipe. It may have a piston diameter between 60 to 100 mm ($2\frac{1}{2}$ to 4 in) and a stroke of 25 to 125 mm (3 to 5 in). Whatever the stroke and piston diameter, replacement cup leathers must be readily available to the people using the pump. A PVC lining is acceptable instead of the brass lining.

2. A well screen, commonly called a "well point" when used with a hand pump, as it commonly has a point on the end.

3. Galvanized steel pipe, of 32 mm (1¼ in) nominal diameter to connect the well screen in the aquifer to the pump.

A concrete or masonry platform may be provided around the pipe at the ground surface. It is not essential for sanitary purposes if the connecting pipe is 10 m long or more. When not essential, platforms are commonly left to the pump user to provide, and are therefore not an expense to the public program. If a platform is provided, a short drain is usually also provided to carry waste-water away from the immediate vicinity of the well.

The cost of a hand pump well is the cost of the materials at site, plus the cost of labor of installation and the cost of supervision. The average cost of such a HPW will vary from ₱1,000 (at 20 m depth) to ₱3,000 (at 75 m depth).

Deep Water Level HPW. A deep water level hand pump well should have the pumping element, the pump piston, installed below the pumping water level. If the pumping water level is at 15 m, for instance, the pump piston must be 15 m or more under ground.

The essential materials of such a well would include:

1. A pump, or more properly, a pump cylinder, which should be brass lined steel, of 57 mm (2¼ in) or 54 mm (2 3/16 in) diameter. The cylinder should include the piston, of three-cup type, and the bottom valve assembly. The cylinder should connect on the top to 62 mm (2½ in) diameter pipe and on the bottom to a 62 mm (2½ in) diameter pipe.

2. The well screen, which will be the same as that of the other wells described here.

3. Galvanized steel pipe of 62 mm (2½ in) nominal diameter to connect the well screen to the pump cylinder and serves as well as casing.

4. Galvanized steel pipe of 62 mm (2½ in) nominal diameter to connect the top of the pump cylinder to the discharge head.

5. A pump rod to connect the pump piston through the discharge head to the pump handle. If the rod is not more than about 12 m long it may be of 11 mm (7/16 in) steel. If more than about 12 m long the pump rod should be wood.

The average cost of an HPW will vary from ₱2,500 (at 20 m depth) to ₱8,000 (at 75 m depth).

Potential Application

In the five study areas, there are certain fringe areas[7] that could potentially adopt the HPW as an interim measure for water supply until such time as the permanent conventional system extends toward those areas. The following table shows such areas and probable per capita costs if a HPW were provided for each home:

Water District	Community	Pumping Level (m)	Well Depth (m)	HPW Cost	Per Capita Cost[8]
MOWD	Clarin	near surface	40-50	₱2,000	₱290
CNWD	Basud	3-6	20-60	₱1,800	₱260
MCWD	Liloan	8-15	20-30	₱3,000	₱430
MCWD	Compostela	15	30-50	₱4,400	₱630

The HPW has a per capita cost which is 50-100 per cent of the per capita cost of a piped water system. Because of the better level of service and fire-fighting potential of the piped water system, the HPW alternative is not recommended in areas where a piped water system is a viable alternative. Hence, those areas within the Water District which have a population density of 100 people per hectare should be served by a piped water supply.

7/ Where groundwater conditions are favorable for HPW.

8/ Based on 7 persons per house.

APPENDIX VIII-E

WATER RESOURCES CONSERVATION MEASURES

Alternatives available to counteract future (and present) water shortages consist of the following: reuse of wastewater, desalting, precipitation augmentation, land management, and dual plumbing system . These alternatives are discussed below.

Wastewater Reuse

One of the potential alternatives in meeting future water demand is the reuse or recycling of wastewater (sewage). In Singapore, reclaimed wastewater is used in the cooling process in industries. A full-scale municipal reuse facility in Windhoek in Southwest Africa built in 1969 provides a major source of potable water.

Reuse of wastewater can be accomplished in two ways: by natural self-purification which makes reuse possible for irrigation and recharging of ground and surface waters, and by technological process. The technology of reuse involves treatment of wastewater from the community for domestic, industrial, irrigation and other purposes.

Complex treatment processes are necessary to remove the objectionable characteristics of wastewater and make it suitable for a particular use. There are three basic phases of treatment - primary, secondary and tertiary. One of these or all may be applied depending on the types of use and pollutants present in the wastewater.

Studies on wastewater reclamation deal with enhancing its economic feasibility for large-scale use and technological expansion. Achieving these objectives will depend on several factors.

It is technologically possible to produce water of any desired quality from any source. However, the controlling factor is economics. For instance, because of its lesser solids content, wastewater reclamation is simpler than seawater desalting as an alternative source of supply.

The feasibility of wastewater reclamation as a source of water supply will be limited in the study area since wastewater volumes are currently minimal, and future increases in sewerage are expected to be slow.

Desalting

Desalting is the process of converting seawater into fresh water. It involves removing substantial amounts of the ocean's salts and minerals.

There are three basic methods of desalting: distillation, freezing and electrodialysis, each with several variations.

Over 90 per cent of the present application of desalting is done by distillation. In this process, seawater is evaporated and the vapor is condensed. Salt deposits form on the surfaces of the evaporating equipment and the desalinated water is the resulting distillate. The least costly distillation unit uses solar energy as heat source.

Electrodialysis obtains fresh water by using an electric current to separate the ions of the contaminating salts.

In the process of freezing, ice is formed from a saline solution and is melted to produce fresh water. However, the melted ice sometimes has a salty taste.

In 1970, 33 small desalting plants were put into operation throughout the world, with a combined capacity of 226,000 cumd. Kuwait has the largest plant with a 113,600 cumd capacity which is sufficient to supply a population of 150,000. Other plants are found in the Netherlands, the United States, Venezuela and Aruba.

For the most part, desalting is still experimental. At present it is not technically and economically feasible to convert meaningful amounts of seawater into fresh water. All the processes have inherent defects for general use, including the problem of disposing about 50 per cent of removed salts and minerals of the total treated seawater.

These processes are also costly because they involve significant quantities of energy. Whereas treatment of ordinary water supply costs about five cents per 1,000 gallons of fresh water, desalting costs about $1.00 per 1,000 gallons of desalted water. This cost covers only the plant itself and excludes necessary transmission facilities. (These are pre-energy crisis costs.)

Precipitation Augmentation

Rain can be artificially induced to increase water supply (although it does not diminish the need to collect it). The most common method of stimulating rainfall is cloud seeding.

The theory behind cloud seeding is that under certain conditions, air containing much moisture will not yield precipitation that might possibly occur because of the absence of particles of dust, crystal or chemical droplets. In cloud seeding, such particles are artificially implanted in supersaturated clouds to stimulate rainfall. The particles used in the method are usually silver iodide crystals.

The costs of cloud seeding in 1971 ranged from $0.81 to $1.86 per thousand cubic meters of additional run-off. This cost range, however, was derived from planning reports and as such, might not represent actual operations.

Cloud seeding does not always yield the desired effects of increased rainfall. Experiments show that the method also results in decreases in the amount of expected precipitation. These effects have not been sufficiently explained although a theory advanced is that the amount of rainfall depends on the types of cloud systems being seeded.

Increases in precipitation do not necessarily produce proportional increases in usable water supply. The opportunities to increase precipitation depend on climatic conditions such that during the dry season, water supply increases are less frequent. Precipitation augmentation encourages the growth of vegetation that can reduce usable water yield.

Land Management

Land management can affect the amount and quality of water available for use. It is a method of both increasing and conserving water supply.

Two potential techniques of land management that apply to Philippine conditions are forest management and control of streambank vegetation. Forests impact upon water supply in a number of ways. They intercept rain on the surface of the leaves. They draw moisture from the soil and release it into the atmosphere by transpiration. Through their roots, leaves and other parts, forests facilitate the infiltration of precipitation into the soil. They also tend to shade the soil and slow down wind velocity, thus reducing evaporation from the soil surface. Any alteration on the amount and type of vegetation, such as deforestration, will affect water supply.

Phreatophytes or deep-rooted vegetation along the banks of canals and rivers consume much water in their growth. Especially in cases where precipitation is low, this vegetation may reduce the streamflow and the discharges of springs. Sometimes, it also tends to increase flood stages when it invades stream channels and reduces channel capacity. Phreatophytes are useful in the sense that they provide an important wildlife habitat; otherwise, they do not have food value. Based on these uses and effects on the water supply, they have to be managed carefully as uprooting them is not necessarily the best answer to increasing water supply.

Dual Plumbing System

A relatively small increment of the total public water supply demands highly potable and clean water such as that required for drinking, cooling, bathing and washing clothes. It is possible, for instance, to use seawater for toilet flushing, washing streets, fire-fighting. Where fresh potable water is in short supply, such as in Singapore and Hongkong, a dual system has demonstrated its effectiveness. For example, in Hongkong during the severe drought of the summer of 1963, water service was rationed into the various city sectors 4 hours every 4 days. Extraneous uses of water such as toilet flushing were therefore severely curtailed. In some of the high-rise government housing in Hongkong, dual plumbing system has been used, with sea water for toilet flushing.

There are two main objections that have been identified in the use of dual system - cross-connections and associated cost. Dual water supply system where one system delivers potable water and the other system furnishes untreated water can very well lead to water-borne disease outbreaks. Where proposed, the dual water supply system should have the non-potable supply clearly indicated and separated from the potable supply. Proper plumbing codes and supervision of plumbing installation could minimise this health hazard.

If an existing system is to be replaced entirely by a dual system, the cost may well be unreasonable. However, if the existing water supply piping is retained as a potable (drinking) system and a new non-potable pipe network utilising sea water is added, there is a possibility that the economics may prove the dual system to be worthwhile. Savings result where the non-potable system serves high-rise multi-family dwelling units with high population density. The increased cost resulting from the independent system may be offset by eliminating the need for developing new water resources and above all, retaining premium high quality water for domestic consumption. Accordingly, the feasibility of the dual system has already been proven. Because the potable water system will not be used for fire protection, its distribution system need not be designed for high pressure otherwise required in a system with high fire demand.

APPENDIX TO CHAPTER IX

APPENDIX IX-B

MISCELLANEOUS (EARLY ACTION) IMPROVEMENTS TO THE EXISTING SYSTEM

Item 1. In order to maximize water production from the existing wells, it is recommended that specific capacity tests be performed to determine the possibility of obtaining additional water by installing larger pumps.

In addition, it is recommended that the pumps presently installed in the existing wells be tested to determine the need for repair or replacement. The cost of the pump sets has been included in the Early Action Program.

Item 2. In order to maximize the transmission capacity of the pipeline from the existing spring facilities, it is recommended that air relief valves be purchased and installed at appropriate high points along the pipeline. Three such locations have been noted by the project staff. The repair of leaks along this pipeline will also increase its efficiency.

Item 3. It is recommended that the piping arrangement of the Mataas na Lupa storage tank be modified to permit the flows from both the storage tank and the adjacent wells (No. 1 and 2) to enter the distribution system during periods of peak demand. At present only the storage tank is directly connected to the system, the wells serving only to fill the tank. By connecting the reservoir directly to the adjacent 200 mm diameter distribution pipeline, via the well discharge piping, water from both these sources will be available when required. The following materials are required for this work:

(a) One - 200 x 150 mm diameter Tee
(b) One - 150 x 150 mm diameter Tee
(c) One - 150 mm diameter Valve
(d) 50 m - 150 mm diameter Pipe

Item 4. In order to preserve the quality of water from the existing spring sources, certain improvements should be carried out. These steps are as follows:

(a) Provide surface drainage facilities at the existing spring collection chambers, to divert surface runoff.
(b) Seal the walls and roofs of the collection chambers with a coating of cement mortar.
(c) Provide drain valves and manhole cover locks.

Item 5. Eliminate public faucets where possible. If the municipality or barangay concerned is willing to pay for such services, the existing public faucets should be provided with water meters and retained.

APPENDIX IX-C

DISTRIBUTION SYSTEM GROWTH

General

It is necessary to project the growth of the distribution system in order to estimate the required expenditures for internal network piping, service connections, and requirements for fire hydrants. The projection of distribution system growth is based on (1) an apportionment of the population served into several sectors within the study area; (2) a gradual decrease in the number of people served at each connection; and, (3) the number of connections anticipated per hectare of area served. Each of these items is discussed in detail in subsequent sections.

Population Served

The population served projections given in Chapter VI are divided into present service area, 1990 study area, and year 2000 study area. These projections are tabulated below:

	Population Served Projection			
	1975	1985	1990	2000
A. Present Service Area				
1. Poblacion	4,800	16,600	24,400	30,300
2. Outside Poblacion	4,300	10,100	15,600	25,800
B. 1990 Study Area		4,400	10,200	23,900
C. Year 2000 Study Area			3,200	8,700
Total	9,100	31,100	53,400	88,700

Number of Consumers Served per Connection

The present average number of consumers per connection in the LCWD is estimated to be 12.8. Over the next 25 years, this figure is assumed to decrease gradually because of (1) decreasing population growth which will reduce the number of persons per household; (2) increasing economic growth which will enable more households to own or rent dwelling units; and (3) more reliable water service and supply which will eliminate the practice of non-concessionaires "borrowing" water from concessionaires. The average number of persons per connection is projected as follows:

	Number of Persons per Connection			
	1975	1985	1990	2000
A. Present Service Area	12.8	7	6.5	6
B. 1990 Study Area		6.5	6	5.5
C. Year 2000 Study Area			6	5.5

The number of persons per connection is assumed to be higher in the present service area than in the service area extension. The marked reduction in the number of persons per connection in the present service area between 1975 and 1985 will be achieved by converting the present secondary users to primary users.

Number of Connections per Hectare

Projecting the number of concessionaires to be connected per hectare of area served enables the estimation of the total number of hectares served. At present the LCWD serves approximately 130 ha in and around the core city. There are approximately 570 concessionaires or an average of 4.4 connections per hectare. This is a low figure for connections per hectare and reflects the poor water service and high number of secondary users per connection. Such conditions indicate that as service is improved the water district should expect a signficant increase in the number of concessionaires. The number of connections per hectare in the poblacion is expected to increase to 35 by 1990 and to 47 by year 2000. The number of connections per hectare in the poblacion in year 2000 will increase because of an expected increase in the number of multi-family dwellings. The method used in calculating the number of hectares to be served in the 1990 study area for the year 1985 is illustrated below:

$$\text{Number of Hectares Served} = \frac{\text{Number of People Served}}{\text{Number of Consumers per Connection} \times \text{Number of Connections per Hectare}}$$

$$= \frac{4,400}{6.5 \times 12} = 56 \text{ ha}$$

The 56 ha represents the net area served. This area should be increased by 20 per cent to reflect the land which will be used for schools, churches, and other institutions. Thus the total area served in the 1990 study area will be approximately 70 hectares. The projected service area for the LCWD is listed in Appendix Table IX-C-1.

APPENDIX TABLE IX-C-1

PROJECTION OF AREA SERVED

Study Area	1985 Number of Connections per Hectare	1985 Area Served (ha) Net	1985 Area Served (ha) Gross	1990 Number of Connections per Hectare	1990 Area Served (ha) Net	1990 Area Served (ha) Gross	2000 Number of Connections per Hectare	2000 Area Served (ha) Net	2000 Area Served (ha) Gross
A. Present Service Area									
1. Poblacion	22	108	130	35	108	130	47	108	130
2. Outside Poblacion	15	104	125	17	153	185	20	234	280
B. 1990 Study Area	12	56	70	14	121	145	17	256	310
C. Year 2000 Study Area				15	44	50	14	113	140
T O T A L			325			510			860

Area Served by Internal Network System

The present LCWD service area of 190 ha can be separated into two types of service - service by connection to the internal network system and service by connection to distribution mains and transmission lines. Concessionaires served by mains of 100 mm in diameter or smaller are those served by the internal network system. Those concessionaires connected to pipe 125 mm in diameter and larger are considered to be served by distribution mains and transmission lines. Of the present service area of 190 ha, 140 ha is served by the internal network system and 50 ha is served by distribution mains and transmission lines. It is assumed that the practice of making direct connection to distribution mains and transmission lines will continue, and therefore, the area served by distribution mains and transmission mains is subtracted from the total service area to determine the area which will receive internal network piping. It is estimated that 25 m can be served on each side of transmission lines and distribution mains. No service area is attributed to those transmission lines passing through areas of very low density population or to mains parallel to existing mains. The areas served by transmission lines and distribution mains are listed below:

	1985	1990	2000
Length (m) of transmission and distribution mains likely to support concessionaires	19,900	27,000	36,200
Corresponding area (ha)	100	135	180

The expansion of the service area and the area served by the internal network system is given in Appendix Table IX-C-2. By 1990, the LCWD will have extended internal network service to serve a total of 375 ha and by year 2000 to serve a total of 680 hectares.

In addition to the installation of new internal network system in the recommended program, it will be necessary to reinforce or replace the existing internal network. This will consist of installing new 100-mm and 150-mm pipes to replace old pipe or pipes of too small a diameter to provide adequate service. Dead-end pipes will be looped, valves will be repaired or new valves will be installed, and new fire hydrants will also be installed.

In order to manage the growth of the system and to obtain financing in approximately equal increments, the expansion of the internal network system is divided into five construction periods between 1978

APPENDIX TABLE IX-C-2

PROJECTED AREA SERVED BY INTERNAL NETWORK SYSTEM

	Area (ha) Served			
	1975	1985	1990	2000
A. Area served by transmission lines and distribution mains	50	100	135	180
B. Area served by internal network system				
1. Existing	140	140	140	140
2. New System		85	235	540
3. Total	140	225	375	680
C. Total service area	190	325	510	860

and year 2000. The first three periods are four-year intervals, the last of which ends in 1990. The final two periods are five-year intervals, the last ending in year 2000. The construction program for the internal network is listed below:

	Area (ha) of Internal Network	
Construction Period	Reinforcement	New Service Area
I. First Stage		
A. 1978-82	80	40
B. 1982-86	40	70
C. 1986-90	20	125
Sub-total	140	235
II. Second Stage		
A. 1990-1995		150
B. 1995-2000		155
Sub-total		305
Grand Total	140	540

The cost of reinforcing the existing internal network system is listed in Item A of Table VIII-4. For estimating the cost of internal network extension, a total of 100 m of internal network pipe per hectare served was assumed. The cost of internal network extension is listed in Table VIII-4, Item B. The 80 ha of internal network reinforced in Phase I-A will be in the poblacion area. The internal network reinforced in Phases I-B and I-C will be outside the poblacion. Because the existing internal network systems outside the poblacion are so inadequate, the cost of the reinforcement is assumed to be the same as that for internal network extended to new service areas.

Area Receiving Fire Protection

Because of the financial impact of the overall construction program on the concessionaires in the service area, it is proposed that in Phase A of Stage I (1978-82) only the poblacion area be provided fire hydrants. This will correspond to the 80 ha of the existing internal network which will be reinforced.

The areas outside the poblacion will receive fire protection at later stages. The extension of fire protection will gradually increase, so that by Phase B of Stage II the installation of hydrants will coincide with the construction of the internal network. The construction cost of hydrants is listed in Item C of Table VIII-4. Provision is also made for upgrading 50 ha of residential fire service to high-value fire service. The schedule for fire hydrant installation is listed below:

Construction Period	Area (ha) Having Fire Protection	
	High-Value Area	Residential Area
I. First Stage		
A. 1978-82	40	90
B. 1982-86	30	100
C. 1986-90	30	130
II. Second Stage		
A. 1990-1995	40	220
B. 1995-2000	50 1/	180

Number of Connections

The projection of the number of connections is obtained by dividing the population served in the study area sub-sectors by the average number of people per connection. The number of connections projected for each sub-sector is listed as follows:

1/ Corresponds to upgrading residential fire service to high-value fire service.

	Number of Service Connections			
	1975	1985	1990	2000
A. Present Service Area	714	3,814	6,154	9,350
B. 1990 Study Area		677	1,700	4,345
C. Year 2000 Study Area			533	1,582
Total	714	4,491	8,387	15,277
Rounded		4,500	8,400	15,300

Between 1975 and 1978 the number of connections is projected to increase from 714 to 950. The water supply for the new concessionaires can be obtained by an effective metering program to eliminate waste at flat-rate connections, by reducing leakage in transmission lines and distribution mains, and by increase in supply when the Market Well is connected to the distribution system. The schedule for installation of service connections is listed as follows:

Construction Period	Number of Connections per Construction Period	Total Number of Connections at End of Period
I. Early Action and Immediate Improvements Program (1976–1978)	236	950
II. First Stage		
A. 1978–82	2,050	3,000
B. 1982–86	2,700	5,700
C. 1986–90	2,700	8,400
III. Second Stage		
A. 1990–1995	3,450	11,850
B. 1995–2000	3,450	15,300

During the leakage survey which will be conducted as part of the Early Action Program, it is anticipated that existing service connections will be identified as a major leakage problem. The service connections are made with GS pipe. Experience with the GS pipe (especially Schedule 20) indicates that severe corrosion causes leakage in 10 to 20 years, and in even less time in corrosive soils. Though no data are available it is assumed that all of the existing connections will require replacement by 1990. The replacement schedule is listed below:

Construction Period	Number of Existing Service Connections to be Replaced
I. First Stage	
A. 1978–82	200
B. 1982–86	200
C. 1986–90	314

The cost of service connections will be shared between the water district and the concessionaire. The cost of a 5/8-inch or 3/4-inch service connection is ₱500[2/] based on 1976 cost estimates. The water district will pay one-third of the service connection cost and the concessionaire will pay two-thirds of the cost. The concessionaire will also pay for the water meter (₱190 for 5/8-inch meter). The service connection costs for the replaced connection and new connections are itemized below:

	Replacement Cost (₱)[3/]	New Connection Cost (₱)
A. Service Connection Line		
1. Concessionaire	333	333
2. Water District	167	167
B. Water Meter		
1. Concessionaire	-	190
TOTAL	500	690

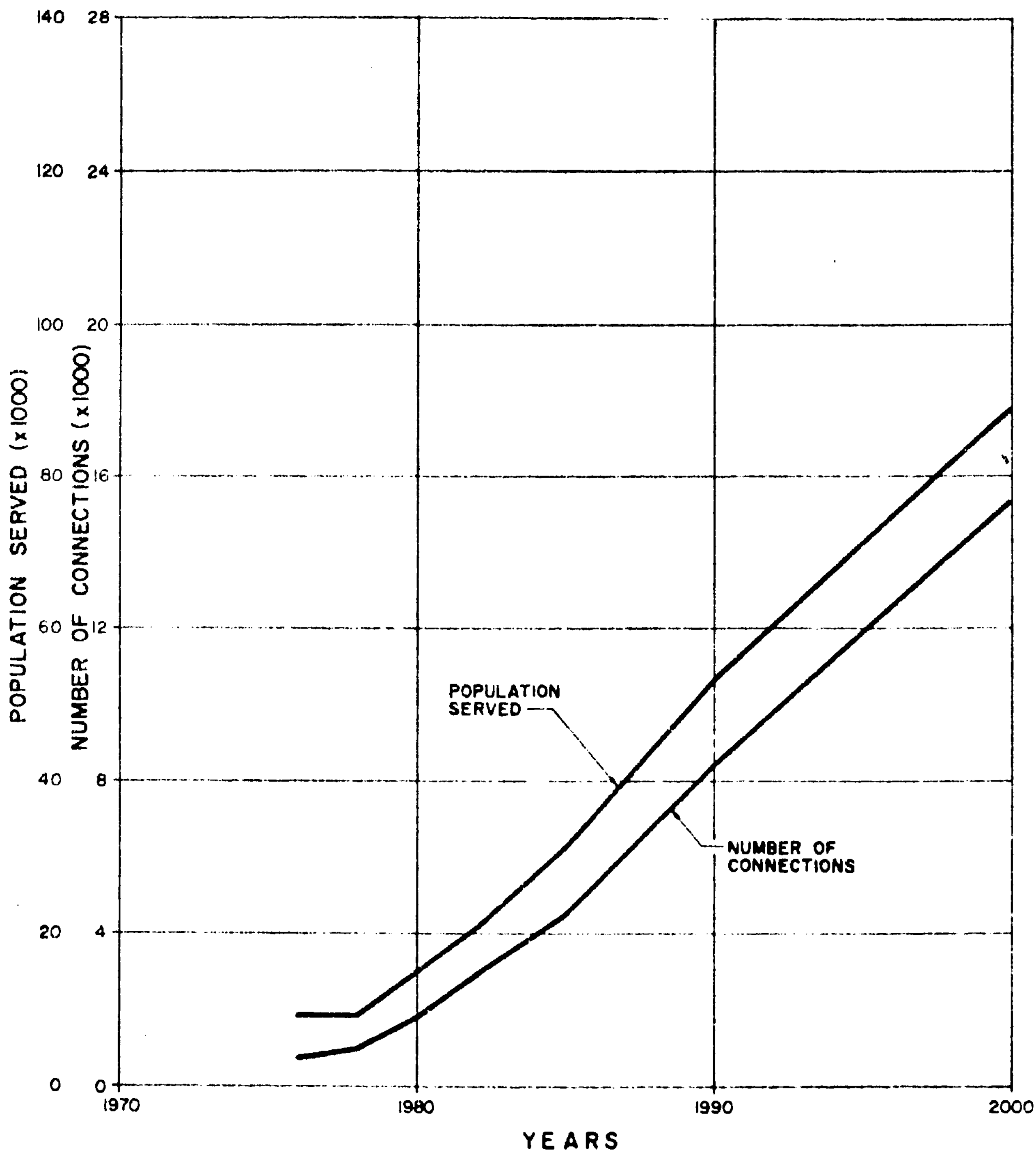
The foreign exchange component of the meter is assumed to be 85 per cent of the total cost of the meter or ₱160.

Summary

The recommended improvement program for each component of the distribution system has been presented in this section. For each component of the distribution system, the recommended schedule of improvements has been described. The projections for population served and number of connections are shown in Appendix Figure IX-C-1.

2/ Connection cost includes ₱100 for pavement replacement.

3/ Meter costs for existing unmetered connections are included in the Early Action Program.



FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM-LWUA

APPENDIX FIGURE IX-C-1
PROJECTIONS FOR POPULATION SERVED AND NUMBER OF CONNECTIONS

The total construction cost by phase is summarized below and in Appendix Table IX-C-3.

Construction Period	Construction Cost (P)
I. First Stage	
A. 1978-1982	2,836,000
B. 1982-1986	3,255,000
C. 1986-1990	3,692,000
Total	9,783,000
II. Second Stage	
A. 1990-1995	4,204,000
B. 1995-2000	4,256,000
Total	8,460,000

In each construction period the service connection costs are at least 50 per cent of the total costs for that period. Since two-thirds of the connection costs are charged to the concessionaires, the portion of the distribution costs which must be financed by the water district will be considerably less than that summarized above.

APPENDIX TABLE IX-C-3

SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period	Item/Description	Quantity	Unit Cost	Total Construction Cost (₱)	FEC (₱)
I. First Stage					
A. 1978–82	Internal Network				
	Reinforcement	80 ha	₱ 9,000/ha	720,000	312,000
	New Service Area	40 ha	₱10,200/ha	408,000	180,000
	Fire Hydrants				
	High-Value Area	40 ha	₱ 3,100/ha	124,000	72,000
	Residential Area	90 ha	₱ 770/ha	69,300	40,500
	Service Connections				
	Replacement	200	₱ 500	100,000	48,000
	New Connections	2,050	₱ 690	1,414,500	820,000
	Sub-total (Rounded)			2,836,000	1,473,000
B. 1982–86	Internal Network				
	Reinforcement	40 ha	₱10,200/ha	408,000	180,000
	New Service Area	70 ha	₱10,200/ha	714,000	315,000
	Fire Hydrants				
	High-Value Area	30 ha	₱ 3,100/ha	93,000	54,000
	Residential Area	100 ha	₱ 770/ha	77,000	45,000
	Service Connections				
	Replacement	200	₱ 500	100,000	48,000
	New Connections	2,700	₱ 690	1,863,000	1,080,000
	Sub-total (Rounded)			3,255,000	1,722,000
C. 1986–90	Internal Network				
	Reinforcement	20 ha	₱10,200/ha	204,000	90,000
	New Service Area	125 ha	₱10,200/ha	1,275,000	562,500
	Fire Hydrants				
	High-Value Area	30 ha	₱ 3,100/ha	93,000	54,000
	Residential Area	130 ha	₱ 770/ha	100,100	58,500
	Service Connections				
	Replacement	314	₱ 500	157,000	75,350
	New Connections	2,700	₱ 690	1,863,000	1,080,000
	Sub-total (Rounded)			₱3,692,000	₱1,920,000
	Grand Total			₱9,783,000	₱5,115,000

APPENDIX TABLE IX-C-3 (Continued)

SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period		Item/Description	Quantity	Unit Cost	Total Construction Cost (₱)	FEC (₱)
II. Second Stage						
A.	1990-95	Internal Network				
		New Service Area	150 ha	₱10,200/ha	1,530,000	675,000
		Fire Hydrants				
		High-Value Area	40 ha	₱ 3,100/ha	124,000	72,000
		Residential Area	220 ha	₱ 770/ha	169,000	99,000
		Service Connections				
		New Connections	3,450	₱ 690	2,381,000	1,380,000
		Sub-total			₱4,204,000	₱2,226,000
B.	1995-2000	Internal Network				
		New Service Area	155 ha	₱10,200/ha	1,581,000	697,500
		Fire Hydrants				
		High-Value Area	50 ha	₱ 3,100/ha	155,000	90,000
		Residential Area	180 ha	₱ 770/ha	139,000	81,000
		Service Connections				
		New Connections	3,450	₱ 690	2,381,000	1,380,000
		Sub-total (Rounded)			₱4,256,000	₱2,249,000
		Grand Total			₱8,460,000	₱4,475,000

APPENDIX TABLE IX-C-4

PRESSURE ZONE I
SYSTEM DATA

```
LIPA CITY 2000 PZ1 PEAK HOUR  CONDITION


INPUT AND OUTPUT IN              LPS
NO OF NODES                       34
NO OF PIPES                       41
MAX NO OF ITERATIONS              20
PEAKING FACTOR               1.50000
ALLOW P-DROP FR/STATIC - PCT    50.0
STATIC HGL FOR P-DROP CALC     363.0
MAX UNBAL - LPS              0.10000
MAX ALLOW VEL -MPS             3.000
MIN ALLOW VEL - MPS            0.400
MAX ALLOW HL -  M/1000 M       10.00
MIN ALLOW HL -  M/1000 M        1.50
MAX ALLOW PRESS - ATM          7.000
MIN ALLOW PRESS - ATM          0.700
NO OF HEADS TO BE READ             1
NO OF UNKNOWN CONSUMPTIONS         1
SUM OF FIXED DEMANDS           35.98
BANDWIDTH                          3
ITER  1 UNBAL     7.53 LPS
ITER  2 UNBAL     2.17 LPS
ITER  3 UNBAL     0.16 LPS
ITER  4 UNBAL     0.00 LPS

SOLUTION NO. 1 REACHED IN    4 ITERATIONS
        0.0009 LPS UNBALANCE
```

APPENDIX TABLE IX-G-4 (Continued)

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS--- MT	MT/1000	CK
20	22	24	296	500.	110	0.915E-03	35.88	0.52		0.69	1.39	LO
22	25	23	195	800.	110	0.112E-01	13.80	0.46		1.44	1.80	
23	28	25	200	540.	100	0.796E-02	23.46	0.75		2.74	5.08	
24	24	28	261	800.	110	0.270E-02	22.56	0.42		0.87	1.08	LO
25	25	26	200	200.	110	0.247E-02	1.30	0.04	LO	0.00	0.02	LO
26	26	27	150	600.	100	0.359E-01	2.49	0.14	LO	0.19	0.32	LO
27	40	26	200	870.	110	0.107E-01	10.90	0.35	LO	0.90	1.03	LO
28	40	41	200	1450.	110	0.179E-01	5.22	0.17	LO	0.38	0.26	LO
29	41	42	150	1300.	100	0.777E-01	11.37	0.64		7.00	5.39	
30	42	43	100	900.	100	0.338E 00	2.58	0.33	LO	2.24	2.49	
31	44	60	125	1900.	100	0.276E 00	10.16	0.83		20.22	10.64	HI
32	29	27	150	950.	100	0.568E-01	6.81	0.39	LO	1.98	2.09	
33	28	29	200	1100.	110	0.136E-01	9.95	0.32	LO	0.96	0.87	LO
34	24	46	200	1150.	110	0.142E-01	4.86	0.15	LO	0.27	0.23	LO
35	32	47	200	660.	110	0.815E-02	7.08	0.23	LO	0.31	0.46	LO
36	32	46	200	300.	110	0.370E-02	6.13	0.19	LO	0.11	0.35	LO
37	35	32	150	1140.	100	0.682E-01	0.0	0.0	LO	0.0	0.0	LO
38	32	33	150	650.	100	0.389E-01	6.34	0.36	LO	1.19	1.83	
39	62	33	150	1100.	100	0.658E-01	7.19	0.41		2.54	2.30	
46	33	29	150	1500.	100	0.397E-01	2.46	0.14	LO	0.47	0.32	LO
47	29	30	150	1000.	100	0.598E-01	21.88	1.24		19.13	19.13	HI
48	30	31	150	700.	100	0.419E-01	14.49	0.82		5.91	8.45	
49	31	45	150	1000.	100	0.593E-01	7.09	0.40		2.25	2.25	
53	70	47	256	550.	110	0.204E-02	7.73	0.15	LO	0.09	0.16	LO
54	47	28	256	400.	110	0.148E-02	20.60	0.40		0.40	1.01	LO
55	51	4[illegible]	200	200.	110	0.247E-02	15.00	0.48		0.37	1.86	
59	48	64	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
60	63	59	125	1167.	100	0.170E 00	6.13	0.50		4.88	4.18	
61	64	63	125	966.	100	0.149E 00	5.68	0.46		3.50	3.62	
62	28	64	125	2007.	100	0.292E 00	1.91	0.16	LO	0.97	0.48	LO
63	64	40	200	250.	110	0.309E-02	21.18	0.67		0.89	3.52	
64	41	63	100	300.	100	0.129E 00	4.66	0.59		2.23	7.44	
65	42	59	100	500.	100	0.215E 00	0.69	0.09	LO	0.11	0.21	LO
66	60	43	100	700.	100	0.302E 00	3.19	0.40		2.57	3.67	
67	53	29	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
68	54	32	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
71	60	59	125	1900.	100	0.276E 00	1.28	0.10	LO	0.44	0.23	LO
73	65	62	200	200.	110	0.247E-02	15.00	0.48		0.37	1.86	
76	68	69	100	185.	90	0.969E-01	13.00	1.66		11.20	60.52	HI
77	69	47	150	140.	90	0.102E-01	13.00	0.74		1.18	8.40	
78	46	70	200	200.	110	0.247E-02	7.73	0.25	LO	0.11	0.54	LO

APPENDIX TABLE IX-C-4 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	-------PRESSURE------- ATM---CK		PCT DROP---CK	
22	333.0	35.88U	338.00	5.00	0.48	LO	83.33	HI
23	320.0	-13.80	332.26U	12.26	1.19		71.50	HI
24	320.0	-8.46	337.31U	17.31	1.68		59.75	HI
25	310.0	-8.35	333.70U	23.70	2.29		55.29	HI
26	315.0	-9.72	333.69U	18.69	1.81		61.06	HI
27	310.0	-9.30	333.50U	23.50	2.27		55.66	HI
28	315.0	-7.84	336.44U	21.44	2.08		55.33	HI
29	310.0	-13.71	335.48U	25.48	2.47		51.92	HI
30	300.0	-7.39	317.35U	17.35	1.68		72.46	HI
31	295.0	-7.39	311.44U	16.44	1.59		75.83	HI
32	320.0	-10.45	337.15U	17.15	1.66		60.12	HI
33	318.0	-11.07	335.96U	17.96	1.74		60.09	HI
35	320.0	0.0	337.15U	17.15	1.66		60.12	HI
40	305.0	-5.05	334.59U	29.59	2.86		48.98	
41	300.0	-4.20	334.21U	34.21	3.31		45.70	
42	310.0	-8.10	327.20U	17.20	1.67		67.54	HI
43	315.0	-5.76	324.96U	9.96	0.96		79.25	HI
44	340.0	10.16	347.75U	7.75	0.75		66.32	HI
45	280.0	-7.09	309.19U	29.19	2.83		54.94	HI
46	318.0	-3.25	337.04U	19.04	1.84		57.68	HI
47	320.0	-7.20	336.84U	16.84	1.63		60.83	HI
48	312.0	30.00	336.81U	24.81	2.40		51.35	HI
51	310.0	15.00	334.58U	24.58	2.38		53.62	HI
53	310.0	30.00	336.83U	26.83	2.60		49.38	
54	320.0	30.00	338.49U	18.49	1.79		57.00	HI
59	313.0	-8.10	327.09U	14.09	1.36		71.81	HI
60	317.0	-5.70	327.53U	10.53	1.02		77.11	HI
62	310.0	-7.81	338.49U	28.49	2.76		46.24	
63	313.0	-4.20	331.97U	18.97	1.84		62.05	HI
64	317.0	-5.05	335.47U	18.47	1.79		59.85	HI
65	310.0	15.00	338.87U	28.87	2.79		45.54	
68	320.0	13.00	349.21U	29.21	2.83		32.06	
69	320.0	0.0	338.02U	18.02	1.74		58.10	HI
70	316.0	0.0	336.93U	20.93	2.03		55.46	HI

APPENDIX TABLE IX-4-5

PRESSURE ZONE 1
SYSTEM DATA

```
LIPA CITY 2000 PZ1 FILLING    CONDITION


INPUT AND OUTPUT IN               LPS
NO OF NODES                        34
NO OF PIPES                        41
MAX NO OF ITERATIONS               20
PEAKING FACTOR                0.30000
ALLOW P-DROP FR/STATIC - PCT 50.0
STATIC HGL FOR P-DROP CALC  363.0
MAX UNBAL - LPS               0.10000
MAX ALLOW VEL -MPS              3.000
MIN ALLOW VEL - MPS             0.400
MAX ALLOW HL -  M/1000 M        10.00
MIN ALLOW HL -  M/1000 M         1.50
MAX ALLOW PRESS - ATM           7.000
MIN ALLOW PRESS - ATM           0.700
NO OF HEADS TO BE READ              1
NO OF UNKNOWN CONSUMPTIONS          1
SUM OF FIXED DEMANDS           -54.19
BANDWIDTH                           3
ITER  1 UNBAL   22.57 LPS
ITER  2 UNBAL    5.94 LPS
ITER  3 UNBAL    1.82 LPS
ITER  4 UNBAL    0.24 LPS
ITER  5 UNBAL    0.00 LPS

SOLUTION NO. 1 REACHED IN   5 ITERATIONS
       0.0045 LPS UNBALANCE
```

APPENDIX TABLE IX-C-5 (Continued)

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
20	24	22	296	500.	110	0.915E-03	54.19	0.79		1.49	2.97	
22	25	23	195	800.	110	0.112E-01	2.76	0.09	LO	0.07	0.09	LO
23	25	28	200	540.	100	0.796E-02	12.13	0.39	LO	0.81	1.50	LO
24	28	24	261	800.	110	0.270E-02	39.44	0.74		2.44	3.05	
25	26	25	200	200.	110	0.2[illegible]E-02	16.56	0.53		0.45	2.23	
26	27	26	150	600.	100	0.359E-01	1.92	0.11	LO	0.12	0.20	LO
27	40	26	200	870.	110	0.107E-01	16.59	0.53		1.95	2.24	
28	40	41	200	1450.	110	0.179E-01	4.08	0.13	LO	0.24	0.17	LO
29	41	42	150	1300.	100	0.777E-01	3.27	0.18	LO	0.70	0.54	LO
30	42	43	100	900.	100	0.360E 00	1.10	0.14	LO	0.46	0.51	LO
31	44	60	125	1900.	100	0.276E 00	0.0	0.0	LO	0.0	0.0	LO
32	29	27	150	950.	100	0.568E-01	3.78	0.21	LO	0.67	0.70	LO
33	25	28	200	1100.	110	0.136E-01	14.99	0.48		2.04	1.86	
34	46	24	200	1150.	110	0.142E-01	16.45	0.52		2.54	2.21	
35	32	47	200	660.	110	0.815E-02	11.16	0.36	LO	0.71	1.08	LO
36	32	46	200	300.	110	0.370E-02	17.08	0.54		0.71	2.37	
37	35	32	150	1140.	100	0.692E-01	0.0	0.0	LO	0.0	0.0	LO
38	33	32	150	650.	100	0.339E-01	0.33	0.02	LO	0.01	0.01	LO
39	33	62	150	1100.	100	0.653E-01	1.56	0.09	LO	0.15	0.14	LO
46	29	33	150	1500.	100	0.397E-01	4.11	0.23	LO	1.23	0.82	LO
47	29	30	150	1000.	100	0.598E-01	4.38	0.25	LO	0.92	0.92	LO
48	30	31	150	700.	100	0.419E-01	2.96	0.16	LO	0.30	0.43	LO
49	31	45	150	1000.	100	0.598E-01	1.42	0.08	LO	0.11	0.11	LO
53	47	70	256	550.	110	0.234E-02	0.02	0.00	LO	0.00	0.00	LO
54	47	28	256	400.	110	0.148E-02	9.70	0.19	LO	0.10	0.25	LO
55	51	41	200	200.	110	0.247E-02	0.0	0.0	LO	0.0	0.0	LO
59	48	64	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
60	63	59	125	1167.	100	0.170E 00	2.26	0.18	LO	0.77	0.66	LO
61	64	63	125	966.	100	0.140E 00	3.13	0.26	LO	1.16	1.20	LO
62	64	28	125	2007.	100	0.292E 00	4.18	0.34	LO	4.12	2.05	
63	64	40	200	250.	110	0.309E-02	21.68	0.69		0.92	3.68	
64	63	41	100	300.	100	0.129E 00	0.03	0.00	LO	0.00	0.00	LO
65	42	59	100	500.	100	0.215E 00	0.55	0.07	LO	0.07	0.14	LO
66	60	43	100	700.	100	0.302E 00	0.06	0.01	LO	0.00	0.00	LO
67	53	29	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
68	54	32	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
71	59	60	125	1900.	100	0.276E 00	1.20	0.10	LO	0.39	0.20	LO
73	65	62	200	200.	110	0.247E-02	0.0	0.0	LO	0.0	0.0	LO
76	68	69	100	185.	90	0.969E-01	0.0	0.0	LO	0.0	0.0	LO
77	69	47	150	140.	90	0.102E-01	0.0	0.0	LO	0.0	0.0	LO
78	70	46	200	200.	110	0.247E-02	0.02	0.00	LO	0.00	0.00	LO

APPENDIX TABLE IX-0-5 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	-------PRESSURE------- ATM	CK	PCT DPGP	CK
22	333.0	-54.190	336.50	3.50	0.34	LO	88.33	HI
23	320.0	-2.76	341.160	21.16	2.05		50.79	HI
24	320.0	-1.69	337.990	17.99	1.74		58.17	HI
25	310.0	-1.67	341.230	31.23	3.02		41.07	
26	315.0	-1.94	341.680	26.68	2.58		44.42	
27	310.0	-1.86	341.800	31.80	3.08		40.00	
28	315.0	-1.57	340.420	25.42	2.46		47.03	
29	310.0	-2.74	342.470	32.47	3.14		36.74	
30	300.0	-1.48	341.550	41.55	4.02		34.05	
31	295.0	-1.48	341.250	46.25	4.48		31.99	
32	320.0	-2.09	341.230	21.23	2.06		50.62	HI
33	318.0	-2.21	341.240	23.24	2.25		46.36	
35	320.0	0.0	341.230	21.23	2.06		50.62	HI
40	305.0	-1.01	343.630	38.63	3.74		35.40	
41	300.0	-0.84	343.390	43.39	4.20		31.13	
42	310.0	-1.62	342.690	32.69	3.16		38.32	
43	315.0	-1.15	342.230	27.23	2.64		43.27	
44	340.0	0.0	342.230	2.23	0.22	LO	90.30	HI
45	280.0	-1.42	341.130	61.13	5.92		26.35	
46	318.0	-0.65	340.520	22.52	2.18		49.95	
47	320.0	-1.44	340.520	20.52	1.99		52.27	HI
48	312.0	30.00	345.890	33.89	3.28		33.55	
51	310.0	0.0	343.390	33.39	3.23		37.01	
53	310.0	30.00	343.310	33.31	3.27		36.21	
54	320.0	30.00	342.580	22.58	2.19		47.50	
59	313.0	-1.62	342.620	29.62	2.87		40.77	
60	317.0	-1.14	342.240	25.24	2.44		45.15	
62	310.0	-1.56	341.090	31.09	3.01		41.34	
63	313.0	-0.84	343.390	30.39	2.94		39.23	
64	317.0	-1.01	344.550	27.55	2.67		40.11	
65	310.0	0.0	341.090	31.09	3.01		41.34	
68	320.0	0.0	340.520	20.52	1.99		52.27	HI
69	320.0	0.0	340.520	20.52	1.99		52.27	HI
70	316.0	0.0	340.520	24.52	2.37		47.82	

LIPA CITY 2000 PZ2 PEAK HOUR CONDITION

```
INPUT AND OUTPUT IN                LPS
NO OF NODES                         19
NO OF PIPES                         19
MAX NO OF ITERATIONS                20
PEAKING FACTOR                 1.50000
ALLOW P-DROP FR/STATIC - PCT    50.0
STATIC HGL FOR P-DROP CALC     363.0
MAX UNBAL - LPS                0.10000
MAX ALLOW VEL -MPS               3.000
MIN ALLOW VEL - MPS              0.400
MAX ALLOW HL -  M/1000 M        10.00
MIN ALLOW HL -  M/1000 M         1.50
MAX ALLOW PRESS - ATM            7.000
MIN ALLOW PRESS - ATM            0.700
NO OF HEADS TO BE READ               2
NO OF UNKNOWN CONSUMPTIONS           2
SUM OF FIXED DEMANDS             15.96
BANDWIDTH                            1
ITER  1 UNBAL     8.25 LPS
ITER  2 UNBAL     1.19 LPS
ITER  3 UNBAL     0.07 LPS

SOLUTION NO. 1 REACHED IN    3 ITERATIONS
      0.0720 LPS UNBALANCE
```

APPENDIX TABLE IX-C-6 (Continued)

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS--- MT	MT/1000	CK
9	10	15	250	1000.	110	0.417E-02	6.57	0.13	LO	0.14	0.14	LO
11	12	13	200	500.	110	0.617E-02	20.85	0.66		1.71	3.42	
12	13	35	150	870.	100	0.520E-01	9.05	0.51		3.07	3.53	
13	14	13	200	1300.	110	0.161E-01	2.76	0.09	LO	0.11	0.08	LO
14	15	16	250	800.	110	0.333E-02	0.76	0.02	LO	0.00	0.00	LO
16	19	16	200	1040.	110	0.128E-01	8.10	0.26	LO	0.62	0.59	LO
17	18	19	150	600.	100	0.359E-01	2.13	0.12	LO	0.15	0.24	LO
18	21	19	200	800.	110	0.988E-02	12.68	0.40		1.09	1.36	LO
19	20	21	200	470.	110	0.580E-02	18.14	0.58		1.24	2.64	
41	35	36	150	300.	100	0.179E-01	1.97	0.11	LO	0.06	0.21	LO
42	37	36	150	1200.	110	0.612E-01	8.35	0.47		3.07	2.55	
44	39	37	200	400.	110	0.494E-02	9.39	0.30	LO	0.31	0.78	LO
45	36	37	200	1400.	110	0.173E-01	9.80	0.31	LO	1.19	0.84	LO
50	20	13	150	1000.	100	0.578E-01	6.99	0.40	LO	2.19	2.19	
56	52	19	200	300.	100	0.442E-02	0.0	0.0	LO	0.0	0.0	LO
58	59	20	200	250.	110	0.339E-02	30.00	0.95		1.68	6.71	
60	56	17	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
70	55	38	200	200.	110	0.247E-02	15.00	0.48		0.37	1.86	
71	13	53	200	300.	110	0.370E-02	3.64	0.12	LO	0.04	0.14	LO

APPENDIX TABLE IX-C-6 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM	CK	PRESSURE PCT DROP	CK
10	360.0	6.570	363.00	3.00	0.29	LC	0.02	
12	345.0	-9.15	364.470	19.47	1.88		-8.15	
13	335.0	-10.92	362.760	27.76	2.69		0.87	
15	340.0	-5.80	362.860	22.86	2.21		0.59	
16	325.0	-6.10	362.860	37.86	3.67		0.36	
18	340.0	-4.86	363.620	23.62	2.29		-2.71	
19	320.0	-6.70	363.480	43.48	4.21		-1.11	
20	335.0	-4.38	365.810	30.81	2.98		-10.04	
21	330.0	-5.46	364.570	34.57	3.35		-4.75	
35	323.0	-7.06	359.680	36.68	3.55		8.29	
36	325.0	-10.32	359.620	34.62	3.35		8.89	
37	320.0	-10.83	362.690	42.69	4.13		0.73	
38	310.0	-5.20	363.870	53.87	5.21		-1.64	
39	340.0	9.390	363.00	23.00	2.23		0.0	
50	340.0	30.00	367.490	27.49	2.66		-19.52	
52	320.0	0.0	363.480	43.48	4.21		-1.11	
55	310.0	15.00	364.240	54.24	5.25		-2.34	
56	340.0	30.00	365.310	25.31	2.50		-17.22	
58	335.0	-3.64	362.720	27.72	2.68		1.02	

APPENDIX TABLE IX-6-7

PRESSURE ZONE 2
SYSTEM DATA

```
LIPA CITY 2000 PZ2 FILLING   CONDITION


INPUT ANC OUTPUT IN              LPS
NO OF NODES                       19
NO OF PIPES                       19
MAX NC OF ITERATIONS              20
PEAKING FACTOR               0.30000
ALLOW P-DRCP FR/STATIC - PCT 50.0
STATIC HGL FOR P-DRCP CALC   363.0
MAX UNBAL - LPS              0.10000
MAX ALLOW VEL -MPS             3.COC
MIN ALLOW VEL - MPS            C.400
MAX ALLCW HL -  M/1000 M       10.00
MIN ALLOW HL -  M/1000 M        1.50
MAX ALLCW PRESS - ATM          7.000
MIN ALLOW PRESS - ATM          0.700
NO UF HEADS TO BE READ             2
NC OF UNKNOWN CONSUMPTIONS         2
SUM OF FIXED DEMANDS          -26.81
BANDWIDTH                          1
ITER  1 UNBAL    C.70 LPS
ITER  2 UNBAL    0.04 LPS

SOLUTICN NO. 1 REACHED IN    2 ITERATIONS
       0.0434 LPS UNBALANCE
```

APPENDIX TABLE IX-C-7 (Continued)

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS-- MT	MT/1000	CK
9	15	10	250	1000.	110	0.417E-02	14.14	0.29	LO	0.56	0.56	LO
11	12	13	200	500.	110	0.617E-02	13.17	0.42		0.73	1.46	LO
12	13	35	150	870.	100	0.520E-01	4.35	0.25	LO	0.79	0.91	LO
13	13	16	200	1300.	110	0.161E-01	5.91	0.19	LO	0.43	0.33	LO
14	16	15	250	800.	110	0.333E-02	15.31	0.31	LO	0.52	0.65	LO
16	19	16	200	1040.	110	0.128E-01	10.62	0.34	LO	1.02	0.98	LO
17	19	18	150	600.	100	0.359E-01	1.05	0.06	LO	0.04	0.07	LO
18	19	21	200	800.	110	0.988E-02	1.99	0.06	LO	0.04	0.04	LO
19	21	20	200	470.	110	0.580E-02	0.90	0.03	LO	0.00	0.01	LO
41	35	36	150	300.	100	0.179E-01	2.93	0.17	LO	0.13	0.44	LO
43	36	37	150	1200.	110	0.602E-01	0.87	0.05	LO	0.05	0.04	LO
44	37	39	200	400.	110	0.494E-02	12.66	0.40		0.54	1.36	LO
45	38	37	200	1400.	110	0.173E-01	13.96	0.44		2.28	1.63	
50	18	20	150	1000.	100	0.598E-01	0.08	0.00	LO	0.00	0.00	LO
56	52	19	200	300.	100	0.442E-02	15.00	0.48		0.67	2.22	
58	20	50	200	250.	110	0.309E-02	0.0	0.0	LO	0.0	0.0	LO
69	56	12	200	200.	110	0.247E-02	15.00	0.48		0.37	1.86	
70	55	38	200	200.	110	0.247E-02	15.00	0.48		0.37	1.86	
71	13	58	200	300.	110	0.370E-02	0.73	0.02	LO	0.00	0.01	LO

APPENDIX TABLE IX-G-7 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM	CK	PCT DROP	CK
10	360.0	-14.14U	361.50	1.50	0.15	LC	50.00	
12	345.0	-1.83	363.74U	18.74	1.81		-4.13	
13	335.0	-2.18	363.01U	28.01	2.71		-0.05	
15	340.0	-1.16	362.06U	22.06	2.14		4.08	
16	325.0	-1.22	362.58U	37.58	3.64		1.10	
18	340.0	-0.97	363.56U	23.56	2.28		-2.45	
19	320.0	-1.34	363.60U	43.60	4.22		-1.40	
20	335.0	-0.97	363.56U	28.56	2.77		-2.01	
21	330.0	-1.09	363.57U	33.57	3.25		-1.72	
35	323.0	-1.42	362.22U	39.22	3.80		1.95	
36	325.0	-2.06	362.09U	37.09	3.59		2.40	
37	320.0	-2.17	362.04U	42.04	4.07		2.23	
38	310.0	-1.04	364.32U	54.32	5.26		-2.49	
39	340.0	-12.66U	361.50	21.50	2.03		6.53	
50	340.0	0.0	363.56U	23.56	2.28		-2.45	
52	320.0	15.00	364.27U	44.27	4.29		-2.95	
55	310.0	15.00	364.69U	54.69	5.29		-3.20	
56	340.0	15.00	364.12U	24.12	2.33		-4.85	
58	335.0	-0.73	363.01U	28.01	2.71		-0.04	

APPENDIX TABLE IX-6-8

PRESSURE ZONE 3
SYSTEM DATA

```
LIPA CITY 2000 PZ3 PEAK HOUR CONDITION


INPUT AND OUTPUT IN              LPS
NO OF NODES                       14
NO OF PIPES                       13
MAX NO OF ITERATIONS              20
PEAKING FACTOR               1.50000
ALLOW P-DROP FR/STATIC - PCT 50.0
STATIC HGL FOR P-DROP CALC  383.0
MAX UNBAL - LPS              0.10000
MAX ALLOW VEL -MPS             3.000
MIN ALLOW VEL - MPS            0.400
MAX ALLOW HL -  M/1000 M       10.00
MIN ALLOW HL -  M/1000 M        1.50
MAX ALLOW PRESS - ATM          7.000
MIN ALLOW PRESS - ATM          0.700
NO OF HEADS TO BE READ             2
NO OF UNKNOWN CONSUMPTIONS         2
SUM OF FIXED DEMANDS            9.51
BANDWIDTH                          1
ITER  1 UNBAL     9.34 LPS
ITER  2 UNBAL     0.64 LPS
ITER  3 UNBAL     0.00 LPS

SOLUTION NO. 1 REACHED IN    3 ITERATIONS
       0.0007  LPS UNBALANCE
```

APPENDIX TABLE IX-C-8 (Continued)

PIPE DATA

PIPE NO	NODES FROM	-TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
1	2	1	150	750.	100	0.449E-01	3.67	0.21	LO	0.50	0.67	LO
2	2	9	200	400.	110	0.494E-02	5.22	0.17	LO	0.11	0.26	LO
3	3	2	200	1500.	110	0.185E-01	15.35	0.49		2.91	1.94	
4	4	3	200	900.	110	0.111E-01	6.80	0.22	LO	0.39	0.43	LO
5	5	6	150	1470.	100	0.879E-01	4.45	0.25	LO	1.40	0.95	LO
6	4	5	200	470.	110	0.580E-02	18.46	0.59		1.26	2.73	
7	5	7	200	1340.	110	0.165E-01	4.62	0.15	LO	0.28	0.21	LO
8	7	11	200	1000.	110	0.123E-01	17.37	0.55		2.44	2.44	
10	11	12	200	1000.	110	0.123E-01	0.0	0.0	LO	0.0	0.0	LO
72	61	7	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
74	66	4	200	200.	110	0.247E-02	30.00	0.95		1.34	6.71	
75	67	3	150	200.	100	0.120E-01	15.00	0.85		1.80	9.01	
79	68	11	200	280.	100	0.412E-02	14.73	0.47		0.60	2.15	

APPENDIX TABLE IX-C-8 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	--------PRESSURE-------- ATM---CK	PCT DROP---CK
1	365.0	-3.67	382.61U	17.61	1.70	2.19
2	360.0	-6.45	383.11U	23.11	2.24	-0.46
3	355.0	-6.45	386.02U	31.02	3.00	-10.77
4	355.0	-4.74	386.40U	31.40	3.04	-12.15
5	358.0	-9.39	385.12U	27.12	2.63	-8.48
6	350.0	-4.45	383.72U	33.72	3.26	-2.18
7	370.0	-17.25	384.84U	14.84	1.44	-14.14
9	360.0	-5.22U	383.00	23.00	2.23	0.0
11	360.0	-32.10	382.40U	22.40	2.17	2.62
12	345.0	0.0	382.40U	37.40	3.62	1.58
61	370.0	30.00	386.18U	16.18	1.57	-24.46
66	355.0	30.00	387.75U	32.75	3.17	-16.95
67	355.0	15.00	387.82U	32.62	3.18	-17.21
68	365.0	14.73U	383.00	18.00	1.74	0.01

[illegible]

[illegible]

[illegible]

```
LIPA CITY 2000 PZ3 FILLING   CONDITION


INPUT AND OUTPUT IN                LPS
NO OF NODES                         14
NO OF PIPES                         13
MAX NO OF ITERATIONS                20
PEAKING FACTOR                 0.30000
ALLOW P-DROP FR/STATIC - PCT 50.0
STATIC HGL FOR P-DROP CALC   383.0
MAX UNBAL - LPS                0.10000
MAX ALLOW VEL -MPS               3.000
MIN ALLOW VEL - MPS              0.400
MAX ALLOW HL -   M/1000 M       10.00
MIN ALLOW HL -   M/1000 M        1.50
MAX ALLOW PRESS - ATM            7.000
MIN ALLOW PRESS - ATM            0.700
NO OF HEADS TO BE READ               2
NO OF UNKNOWN CONSUMPTIONS           2
SUM OF FIXED DEMANDS            -28.10
BANDWIDTH                            1
ITER   1 UNBAL     0.53 LPS
ITER   2 UNBAL     0.01 LPS

SOLUTION NO. 1 REACHED IN    2 ITERATIONS
       0.0070 LPS UNBALANCE
```

APPENDIX TABLE IX-6-9 (Continued)

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
1	2	1	150	750.	100	C.449E-01	0.73	0.04	LO	0.03	0.03	LO
2	2	9	200	400.	110	0.494E-02	13.51	0.43		0.61	1.53	
3	3	2	200	1500.	110	C.185E-01	15.54	0.49		2.98	1.99	
4	4	3	200	900.	110	0.111E-01	1.83	0.06	LO	0.03	0.04	LO
5	5	6	150	1470.	100	C.879E-01	0.89	0.05	LO	0.07	0.05	LO
6	5	4	200	470.	110	0.580E-02	2.78	0.09	LO	0.04	0.08	LO
7	7	5	200	1340.	110	0.165E-01	5.55	0.18	LO	0.39	0.29	LO
8	7	11	200	1000.	110	0.123E-01	21.00	0.67		3.47	3.47	
10	11	12	200	1000.	110	0.123E-01	0.0	0.0	LO	0.0	0.0	LO
72	61	7	200	200.	11C	0.247E-02	30.00	0.95		1.34	6.71	
74	66	4	200	200.	110	C.247E-02	0.0	0.0	LO	0.0	0.0	LO
75	67	3	150	200.	100	0.120E-01	15.00	0.85		1.80	9.01	
79	11	68	200	280.	100	C.412E-02	14.58	0.46		0.59	2.11	

APPENDIX TABLE IX-C-9 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM---CK	PRESSURE PCT DROP---CK
1	365.0	-0.73	382.09U	17.09	1.65	5.07
2	360.0	-1.29	382.11U	22.11	2.14	3.86
3	355.0	-1.29	385.09U	30.09	2.91	-7.47
4	355.0	-0.95	385.13U	30.13	2.92	-7.59
5	358.0	-1.88	385.16U	27.16	2.63	-8.66
6	350.0	-0.89	385.09U	35.09	3.40	-6.34
7	370.0	-3.45	385.56U	15.56	1.51	-19.68
9	360.0	-13.51U	381.50	21.50	2.08	6.52
11	360.0	-6.42	382.09U	22.09	2.14	3.96
12	345.0	0.0	382.09U	37.09	3.59	2.40
61	370.0	30.00	386.90U	16.90	1.64	-30.01
66	355.0	0.0	385.13U	30.13	2.92	-7.59
67	355.0	15.00	386.89U	31.89	3.09	-13.90
68	365.0	-14.58U	381.50	16.50	1.60	8.34

APPENDIX IX-H

MANAGEMENT OF GROUNDWATER RESOURCES

The basic problem related specifically to groundwater resources management in LCWD concerns preserving the primary water sources for permanent use. The wells of the Lipa City area are by far the most important sources, thus various measures must be adopted to preserve their usefulness.

All LCWD wells constructed in the area should be equipped with flow measuring devices. A continuous program of flow and water level measurement, as well as water quality determination, should be implemented. This will determine any variations in well and aquifer performance or water quality, which may indicate the need for maintenance or other corrective measures. The maintenance of associated rainfall records may assist in determination of the long-term prospects of the respective well fields as water supply sources.

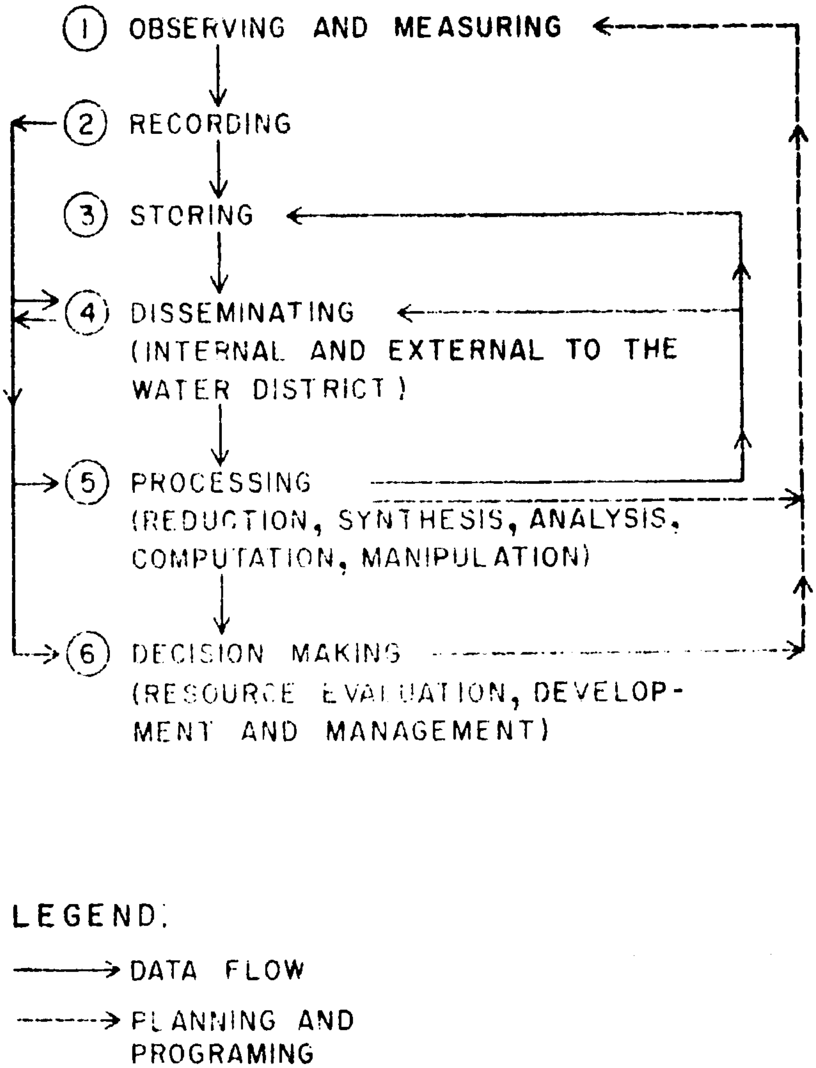
The management considerations with regard to wells should include a monitoring program of flow rates, pumping time, pumping water levels, static water levels and water quality. These records should be maintained on a daily basis. Water quality analyses, consisting of the parameters indicated in Chapter IV, should be performed at least once a month.

The node-path network shown in Appendix Figure IX-H-1 illustrates the development and transmission of water quality monitoring and flow gaging data from the stream to the level of management decisions.

Nodes 1 and 2 represent functions to be performed by the water district. These two functions involve the actual field sampling and laboratory analysis and the recording of data in field and laboratory notebooks and eventually computer input file forms. Node 3 represents the storage function. Data may be stored directly as field and laboratory notes, then published in monthly reports and copies sent to LWUA and the National Water Resources Council (NWRC). One of the future plans of NWRC is to computerize its data system.

The disseminating node, represented as 4, involves the retrieval from the NWRC computer file, or copies of field and laboratory notes, annual, monthly, or other periodic reports and summaries. The processing node, 5, represents data summary by technical personnel and consultants for derivation of water quality/quantity relationships, for the definition of long-term trends, problem areas, and derivation of alternative solutions to water quality/quantity problems. This leads to the decision-making step, 6, wherein planning decisions are made, based on sound water quality/quantity knowledge.

Updating and review of the sampling program should be performed by the water district and their consultants as the goals and needs of the area change. These agencies should be responsible for maintaining communication among all the involved agencies. All data and information should be routed through LWUA and NWRC.



APPENDIX FIGURE IX-H-1
WATER DATA COLLECTION STORAGE AND RETRIEVAL

APPENDIX IX-I

UPDATING THE WATER SUPPLY MASTER PLAN

After the water supply master plan has been adopted and initially implemented, it will be necessary to undertake a program for continuously updating and keeping the plan current. Plan updating should take place at least once every five years, or sooner if significant changes occur. Updating is required to assess the effectiveness of the current plan, the benefits gained, the actual costs, the problems encountered, and to provide overall review, refinement, and direction for the future.

In time, certain aspects of the plan may change. These aspects directly concern or are related to the following areas: technological changes, social goal changes, land use concept changes, and population projection changes.

One of the first steps in the updating procedure is to determine to what degree the previous plan has been implemented and the direction of implementation. This determination has a dual purpose. First, it will reflect the basic suitability of the original plan and second, it will serve as the basis on which to update the plan. Technological changes in the water supply field may result in improved design criteria and contemporary construction methods enough to alter the alternative systems analysis results. Social goals will undoubtedly change and there may be more or less emphasis on environmental and ecological control.

The use of land may change in some areas thereby altering population distribution and the need for previously unplanned-for water services. Population projections may also be altered as refined information becomes available. Transportation networks and employment opportunities will be major determinants in the patterns of population distribution.

An updated report should contain sections or chapters similar to the current plan. The first chapter should be a summary of the updated findings, conclusions, and recommendations.

The second chapter should include the objectives of the updating exercise and the major events that lead into the need for updating. The third chapter should contain an updated description of the study area.

Chapter IV should include the implemented facilities of the master plan, deviations thereof and reasons for the deviation from the master plan. It should include present updated water use and source flow date, and should describe water quality problem areas.

Data concerning former water quality problem areas should be assessed to find what improvements have been made and to document any need for additional improvement. These data should have become available through the recommended monitoring and surveillance program.

The fifth chapter should present new planning the design criteria that might have evolved after the master plan was adopted.

Chapter VI should contain the projections and future conditions. In the areas of economic and demographic change, a review of all updated projections should be made to compare them with the previous plan. Where significant changes have occurred, the data should be used for an updated alternative analysis.

Chapters VII and VIII should contain a re-evaluation of water resources and alternative systems. Where significant changes in projections of future conditions have taken place, it will be necessary to re-evaluate the economic comparison of the alternatives.

Chapter IX should explain in detail the updated plan. In the area of economic and financial feasibility analysis, re-evaluation of the internal rate of return as well as the rate structure should be made.

APPENDIX IX-J

ENVIRONMENTAL ASSESSMENT

A. GENERAL

The consideration of certain environmental factors has been a necessary part of the decision-making process in analyzing the project. On a macroscale, the project is a means of controlling and providing one environmental asset - water - for the needs of people in a developing community.

To provide basic water supply, the project will involve the installation of deep and/or shallow wells; diversion or intake structure; treatment facilities; distribution system storage facilities; pumps, valves and other machinery; customer meters and fire hydrants; and a network of pipelines along streets, roads, highways and other rights-of-way (generally following normal routes of transport) specifically acquired for these purposes. Each component of the project, as it is constructed, will have a local environmental effect in terms of land use, construction activities, and final aesthetics. In the sense of a treatment plant's use of chemicals and power, and in the use of power for pumping water, there will be the wider environmental effect of depleting natural resources.

B. PROBABLE ENVIRONMENTAL EFFECTS

Soil Erosion

A short-term adverse impact will result from soil erosion and dust during construction of transmission and distribution pipelines.

Activities such as urban, roadway and pipeline construction, and agricultural development increase the sediments carried into the streams. Agricultural development may increase erosion four to nine times while urban construction may increase erosion 100 times. When surface water has excess quantities of sediment, the following adverse conditions are experienced:

1. Impairment of recreational values

2. Reduction in fish propagation

3. Increased cost of water treatment

4. Reduction of sunlight penetration

5. Clogging of stream channels

6. Loss of storage capacity in reservoirs

Increased attention has been given recently to the effects of urbanization and construction activities on soil erosion. Concern centers on the resulting sedimentation, i.e., the transport and deposition of soil sediment in receiving waters. Urbanization and construction activities increase sedimentation in two major ways. First, the general increase in peak storm runoff increases the erosion potential. Second, the clearing, leveling, and bulldozing of land for construction expose soil to erosive forces. Certain construction practices tend to increase erosion much more than is necessary.

Good planning cannot eliminate all potential sedimentation problems and certain physical controls may be desirable. Permanent physical controls should be installed as quickly as possible in construction areas. These may include sodded diversion terraces, sod on steep cut or fill banks, and ponds that can be drained and cleaned as necessary during and after construction. Recognition should be given to the limited ability of small ponds or detention basins to remove the clay-sized particles which increase turbidity. More effective methods may be necessary. Where permanent controls cannot be used because of heavy traffic, delays in installing utilities, etc., then mulch, temporary seeding, straw bales, and temporary detention dams, or some combination thereof, may be appropriate.

Dust problems during construction may be minimized by routine sprinkling of the construction area and returning the excavated area to its original state as soon as possible.

The erosion and dust problem can be minimised by the inclusion of strict erosion and dust control criteria in the contract specifications.

Noise

During the construction period, noise associated with heavy equipment and traffic will occur near the construction sites. This temporary disruption will have only a moderate short-term impact. Wells with electric motors or engines and treatment plants will be moderately noisy. Such equipment must be housed within concrete (or block) structures so that noise transmitted is minimal.

Aesthetics

The transmission and distribution lines will be installed below ground level, the surfaces of which will then be returned to original condition. The treatment, storage, pumping, and other supporting facilities and equipment are relatively inconspicuous and will have little or no adverse effect on the existing environment. To minimise the adverse effect on the existing environment, the design of physical facilities must be done unobtrusively and in harmony with the surrounding areas.

Increase in Wastewater

A long-term adverse effect of the water supply project is the unavoidable increase in wastewater. As the available water supply increases, so will wastewater increase. Water-flush toilets are expected to increase in use as local economy progresses.

The additional volumes of wastewater that improved waterworks will generate are anticipated to be disposed of through the same means being used throughout the Philippines, i.e., septic tanks, cesspools, and through surface drains in ditches or g[illegible]rs. In terms of being a burden to the existing surface drai[illegible]ilities, or causing flooding, wastewater is insignifi[illegible]y comparison with run-off from even a minor rainstorm, al[illegible] minor revisions to surface drainage facilities may be necessary to prevent unsightly or undesirable accumulations.

There are no current provisions for wastewater collection, treatment or disposal on a nationwide basis. While knowledgeable officials recognise this problem must be addressed in the future, there are no formal plans to meet the requirement at present. If left unattended and unresolved, unsatisfactory disposal of wastewater could present an additional hazard to public health, and could conceivably produce an adverse visual effect on the environment. The potable water system will not be in danger of contamination from the wastewater since, assuming proper installation and operation of the new improved distribution systems, the water supply will be under sufficient constant pressure to prevent infiltration. In fact, the incidence of waterborne diseases should decline since many present water systems are subject to contamination by infiltration owing to occasional negative pressures on distribution systems.

In the Philippines, wastewater has not been given the significance or priority it enjoys in more developed countries. In the contemplation and order of priorities, local decision-makers consider basic water supply and distribution to be far more important than sewage disposal and at this point are simply unwilling to consider investing an equal amount, and possibly more, of capital funds in sewerage as in waterworks improvements. In the highly urbanized coastal areas[1/] such as Manila, Cebu and Zamboanga, the order of priority appears to be water supply, drainage/flood control and then wastewater disposal.

Before planning the implementation of drainage and/or wastewater facilities, a policy decision must be made on whether the works should be designed on the basis of separate piping for surface run-off (storm) and (sanitary) wastewater, or of a combined system. This can only be accomplished through a feasibility study. It is, therefore, essential that sewerage feasibility studies be conducted as expeditiously as possible after the initial phase of water supply implementation is underway.

[1/] In the Philippines, only Manila and Zamboanga have some form of wastewater disposal system. At present Manila is basically served by an antiquated sewage disposal system designed to serve 220,000 (Metro Manila is now about 4.9 million in population). Practically all other liquid wastewater is transported to natural drainage systems through open ditches, gutters, canals, etc. Yet there is no evidence of intolerable or unacceptable public health conditions as a result. Zamboanga has a system which was built in 1913 and has had no significant improvements since then. It serves about 20 per cent of the core city area.

Environmental Effects of an Impoundment

The construction of a water supply impoundment will have positive and negative impacts on the environment. Evaluation of these impacts on the environment indicates a net environmental benefit.

Positive Impacts. A water supply impoundment will provide safe, adequate and economical water supply to an urban area. The alternative to this impoundment/transmission/treatment scheme is brackish water treatment of pumped groundwater in the specific case of Metropolitan Cebu. Besides being over four times more expensive, the brackish water treatment scheme involves very high energy use.

The lake or impoundment created behind the dam will provide a scenic and aesthetic asset to the community. This lake will attract tourists to view the waterscape which would have then replaced the currently denuded and eroding hillsides.

Reservoir storage, if properly operated, will also reduce the destructive effect of flash floods in areas downstream of the dam.

Negative Impacts. The impoundment will:

1. increase the loss of water due to evaporation;
2. change the habitat of any wildlife and other fauna;
3. serve as a nutrient trap, holding nutrients which otherwise would have moved downstream.
4. need to relocate people/homes from the watershed.

Loss of water from evaporation is a relatively minor impact since water in the uncontrolled rivers eventually gets lost to the seas.

While inundation means a loss of some non-aquatic species, the reservoir will provide a new habitat for waterfowl and other lake-oriented species.

Nutrients "trapped" in the impoundment may accelerate entrophication within the lake, stimulating the growth of algae and aquatic weeds.

With careful consideration during final design, these negative impacts of the project can be significantly reduced.

Increase in Migration to Urban Areas

In the Philippines, migration from rural to urban areas is inescapable. Rural migrants seeking new economic opportunities usually have two distinct choices of destination: the Metropolitan Manila and the other urban areas. Being the most favored migration point, Metropolitan Manila, however, has reached a level when present government policies incline towards migration restraint. In the meanwhile, other urban areas of high growth potential await further development.

Metropolitan Manila is troubled with "people" congestion, "traffic" congestion, lack of housing, environmental problems (water and air pollution), unemployment, slums, poor quality of life, etc. For these reasons, plans for regional development in selective urban areas and growth centers such as Batangas, Dagupan, Baguio, Tuguegarao, Legaspi, Iloilo, Cebu, Butuan and Davao (in the order of distance from Metropolitan Manila) are currently being discussed.

The regional development plan offers an opportunity to minimize population density in already congested areas. It helps avoid overcrowding that clearly has been detrimental to the health, safety and welfare of the residents. It intends to maintain a balance between quality of life and city living. Such development plan intends to intercept rural migration to Metro Manila and redirect this to the various regional development centers.

Infrastructure projects including water supply projects are strong stimuli to urban growth. It is usually admitted that a potable water supply is a necessary condition for economic growth and development of an urban area. Infrastructure projects such as public housing, roads, communications, markets, etc., become more beneficial when an adequate water supply project accompanies such investments.

Therefore, the provision of water supply projects to selected and dispersed urban areas in the Philippines will assist:

(1) in the current policy of discouraging migration into Metro Manila;

(2) in enhancing in a meaningful way the various regional development centers throughout the Philippines.

C. IRREVERSIBLE COMMITMENT OF RESOURCES

The primary impact of the recommended plan on natural resources is the use of chlorine, power and fuel during the operating phase, and the use of materials, foreign exchange, etc., during the construction phase. The labor time for construction and operation is also a natural resource. However, in view of high unemployment in the study area, the use of this resource has a beneficial impact.

Secondary resource commitment occurs as a consequence of new development encouraged by expansion of the water supply system. As the population continues to increase, areas of land will be irreversibly committed to residential, commercial, and industrial uses. Well-developed and successful land use planning will minimise the loss of open space and related natural biota.

D. BENEFITS OF THE PROPOSED ACTION

Health Benefits

The establishment of a water supply system in a community will necessarily bring about health benefits to the population. Undoubtedly, the provision of safe, potable water to the population is a prerequisite for the maintenance of minimum health standards. These health benefits are ordinarily manifested in the following:

1. A significant reduction in the incidence of waterborne diseases such as cholera, dysentery, gastro-enteritis, and typhoid/paratyphoid. As a result, there will be a decrease in the amount of time lost by income earners who are afflicted with such diseases.

2. A subsequent reduction in premature deaths due to the lower incidence of waterborne diseases.

3. A corresponding reduction in medical expenses due to lower incidence of waterborne diseases.

Other Benefits

The water supply project will generate other benefits as shown in the following table. This table indicates the implication of having (with) or not having (without) the water supply project.

Hypothesis	"Without" Project	"With" Project
1. Water Adequacy	will continue to become in short supply; service will be intermittent and unreliable.	supply will be adequate at continuous pressure.
2. Water Quality	will continue to provide unsafe water and water-borne diseases will continuously be a threat.	supply will be safe, wholesome and healthful.
3. Personal Hygiene of Served Population	because of current water shortage, personal cleanliness is expected to range from marginal to lacking.	will enhance personal hygiene and overall appearance and cleanliness of the population.
4. Personal Satisfaction	will be minimal; significant time spent in fetching water.	release time for other productive activities; provides "modernization" benefits; enhances self-reliance.
5. Employment Benefits	no improvement.	will provide short and long-term employment benefits.
6. Fire Protection and Fire Insurance	no improvement; area vulnerable to extensive fire damage because of water shortage; no reduction in insurance because the level of fire risks will remain essentially the same.	will improve the fire-fighting capabilities of the area; reduction in fire insurance cost since availability of water with adequate pressure will reduce fire risks.
7. Water-Using Industries in Area	no inducement to industries which use water as a primary or secondary input to locate in the area.	water-using industries will be encouraged to expand facilities, or relocate in the area.
8. Local Tourism	non-availability of piped potable water and poor sanitation facilities will be a deterrent to local tourism.	availability of water, if accompanied by sanitation program, will help boost local tourism.
9. Development of Areas Adjacent to Core City Area into Housing Subdivisions	no impetus to the development of areas adjacent to core city since not much economic activity can occur without adequate water supply.	will help spur the development of areas adjacent to core city into housing subdivisions because water supply availability somehow enhances standard of living.
10. Wastage of Water - a Valuable Natural Resource	wasteful consumption of water will continue because of the absence of safeguards to check its use.	undertaking of metering program and adoption of new realistic water rates will definitely minimize water wastage.
11. Land Values	market value of land will remain at present levels except for effect of inflation.	will increase land values by at least 10 per cent since water availability is a major consideration in market values of land.

E. ALTERNATIVES TO THE PROPOSED ACTION

Alternatives to the proposed (conventional) water supply project consist of the following: desalting of sea or brackish water, wastewater reuse and dual plumbing. These are discussed below.

<u>Desalting</u>

Desalting is the process of converting seawater into fresh water. It involves removing substantial amounts of the ocean's salts and minerals.

There are three basic methods of desalting: distillation, freezing and electrodialysis, each with several variations.

Over 90 per cent of the present application of desalting is done by distillation. In this process, seawater is evaporated and the vapor is condensed. Salt deposits form on the surfaces of the evaporating equipment and the desalinated water is the resulting distillate. The least costly distillation unit uses solar energy as heat source.

Electrodialysis obtains fresh water by using an electric current to separate the ions of the contaminating salts. In the process of freezing, ice is formed from a saline solution and is melted to produce fresh water. The melted ice, however, sometimes has a salty taste.

In 1970, 33 small-sized desalting plants were put to operation throughout the world, with a combined capacity of 226,000 cumd. Kuwait has the largest plant with a 113,600 cumd capacity sufficient to supply a population of 150,000. Other plants are found in Netherlands, United States, Venezuela and Aruba.

For the most part, desalting is still experimental. At present it is not technically and economically feasible to convert meaningful amounts of seawater into fresh water. All the processes have inherent defects for general use, including the problem of disposing about 50 per cent of removed salts and minerals of the total treated seawater.

These processes are also costly because they involve significant quantities of energy. Whereas treatment of ordinary water supply costs about five cents per 1,000 gallons of fresh water, desalting costs about $1.00 per 1,000 gallons of desalted water. This cost covers only the plant itself and excludes necessary transmission facilities. (Also these costs are pre-energy crisis costs.)

Economic factors cannot be ignored if desalting is to be considered for application in the Philippines. Existing conditions, especially the continual inflationary effect of the worldwide oil crisis and technological limitations, do not allow the immediate use of desalting to augment water supply in the Philippines. Until a technological breakthrough occurs which will require the minimal use of energy, desalting appears to be economically impractical at present.

Wastewater Reuse

One of the potential alternatives in meeting future water demand is the reuse or recycling of wastewater (sewage). In Singapore, reclaimed wastewater is used in the industrial cooling process. A full-scale municipal reuse facility in Windhoek in Southwest Africa built in 1969 provides a major source of potable water.

Reuse of wastewater can be accomplished in two ways: by natural self-purification which makes reuse possible for irrigation and recharging of ground and surface waters, and by technological process. The technology of reuse involves treatment of used water supplies from the community for domestic, industrial, irrigation and other purposes.

Complex treatment processes are necessary to remove the objectionable characteristics of wastewater and make it suitable for a particular use. There are three basic phases of treatment - primary, secondary and advanced. One of these or all may be applied depending on the types of use and pollutants present in the wastewater.

Studies on wastewater reclamation deal with enhancing its economic feasibility for large-scale use and technological expansion. Achieving these objectives will depend on several factors.

It is technologically possible to produce water of any desired quality from any source. However, the controlling factor is economics. Because of its lesser solids content, wastewater reclamation, for instance, is simpler than seawater desalting as an alternative source of supply.

The feasibility of wastewater reclamation as a source of water supply will be limited since wastewater volumes are currently minimal, with future increases in sewage expected to be slow.

Existing wastewater treatment technology is currently applicable for purposes other than potable drinking water use. Drinking water standards have not been designed to apply to recycled wastewater and for this liquid, no potable water standards have been established yet. Even by advanced treatment, certain toxic elements in the wastewater remain. The techniques are not yet fully developed to treat adequately certain objectionable characteristics of wastewater.

The advanced wastewater treatment plant is not simple to operate. Moreover, the problem of treatment is accompanied by another technological problem - disposing the significant quantities of solid matter removed from the treated wastewater.

The wide application of wastewater reuse in the future will depend heavily on technological progress and on public acceptance, in the case of using recycled wastewater for drinking.

Any future consideration of wastewater reuse for municipal water supply will require thorough studies. The present and near future condition of minimal sewer collection facilities (and therefore minimal wastewater) in the Philippines precludes the possibility of harnessing wastewater as a major source of water supply.

Dual Plumbing System

A relatively small increment of the total public water supply demands highly potable and clean water such as that required for drinking, cooling, bathing and washing clothes. It is possible for instance to use sea water for toilet flushing, washing streets, and fire-fighting. Where fresh potable water is in short supply, such as in Singapore and Hongkong, a dual system has demonstrated its efficacy. For example, in Hongkong during the severe drought of summer 1963, water service was rationed into the various city sectors four hours every four days. Extraneous uses of water such as toilet flushing were therefore severely curtailed. In some of the high-rise government housing in Hongkong, dual plumbing system has been used, with sea water for toilet flushing.

There are two main objections that have been identified in the use of dual system - cross-connections and associated cost. The dual water supply system where one system delivers potable water and the other system furnishes untreated water can very well lead to serious waterborne disease outbreaks. Where proposed, the dual water supply system should have the non-potable supply clearly indicated and separated from the potable supply. Proper plumbing codes and supervision of plumbing installation could minimise this health hazard.

If an existing system is to be replaced entirely by a dual system the cost may well be unreasonable. However, if the existing water supply piping is retained for potable (drinking) system and a new non-potable pipe network utilizing sea water is added, there is a possibility that the economics may prove the dual system to be worthwhile. Savings accrue where the non-potable system serves high-rise multi-family dwelling units with high density. The increased cost resulting from the independent system may be offset by eliminating the need for developing new water resources and above all retaining high quality premium water for domestic consumption. Accordingly, the feasibility of the dual system has already been proven. Because the potable water system will not be used for fire protection, its distribution system need not be designed for high pressures that otherwise are required in a system which has a high fire demand. However, because most existing pipe systems have very marginal useful life remaining, the economics will most likely mitigate against a dual system.

Conclusion

The above stated alternatives are likely to be economically less favorable than the proposed conventional water supply project.

Wastewater reuse and dual plumbing have significant health hazards when compared with the proposed project.

Desalting may eliminate impoundments and long transmission lines thereby lessening the adverse effects such as erosion and noise, and on aesthetics. However, desalting will consume unusually large quantities of power and energy.

In all cases, wastewater will be generated and handling facilities will still be required (except for the wastewater reuse alternative). Resources will still be committed such as land, power, fuel and human resources.

From the overall (economics, health, and technology) viewpoint, the proposed water supply project is still the most reliable and economical solution to the water supply needs of these communities.

F. SUMMARY

The probable environmental effects are summarised in table form below:

SUMMARY TABLE
PROBABLE ENVIRONMENTAL EFFECTS

Item	Term	Positive	Negative	Solution
Soil Erosion	Short		✓	tight construction space
Dust	Short		✓	"do"
Noise - Construction	Short		✓	"do"
Noise - Operational	Long		✓	proper design
Aesthetics	Long	✓		"do"
Increase in Wastewater	Long		Unavoidable	solve sewage problem
Impoundment	Long	✓✓	✓	careful design
Migration	Long	✓	✓	careful planning
Resource Use	Long/Short		Unavoidable	

1. Careful design and construction will minimise environmental disturbances while these will also create aesthetic and culturally pleasing conditions under which man can develop his most desirable potentialities.

2. The recommended plan will enhance public health, improve the quality of life in the community, and guide its long-term growth and productivity.

3. The peso costs and the short-term adverse effects are offset by the long-term benefits. When compared to the general benefits, particularly those of the health aspects and social uplift, the amount of upset is relatively minor.

4. The commitment of resources is small compared with the anticipated benefits. Resource use is necessary in the construction and operation of a water supply system, but the overall benefits show the overwhelming advantage of carrying on with the project.

APPENDIX TO CHAPTER X

APPENDIX TABLE X-B-1

PROJECT COST OF RECOMMENDED PROGRAM
LIPA CITY WATER DISTRICT
(WITHOUT ESCALATION)
(P x 1000)

Item		1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	TOTAL
Source Development	a) Structures	-	133	819	1,508	753	-	82	507	932	466	-	-	-	-	-	5,200
	b) Equipment	-	67	414	762	381	-	42	262	481	240	-	-	-	-	-	2,649
Transmission Distribution Mains and Valves		-	163	541	923	923	1,000	826	579	579	643	593	480	480	400	240	8,450
Storage Tanks and Appurtenances	a) Structures	-	4	24	43	22	-	30	256	453	226	-	-	-	-	-	1,058
	b) Equipment	-	3	18	33	17	-	7	56	99	50	-	-	-	-	-	283
Internal Network		-	71	237	404	404	457	452	395	395	462	520	511	511	511	255	5,585
Service Connections	a) Pipes	-	63	209	355	355	418	477	474	474	530	498	413	413	414	207	5,300
	b) Meters	-	19	64	107	107	124	135	125	125	152	193	204	204	204	101	1,864
Water District Buildings		-	58	359	660	330	-	-	-	-	-	-	-	-	-	-	1,407
Early Action Works	a) Service Connections																
	1) Pipes	78	77	-	-	-	-	-	-	-	-	-	-	-	-	-	155
	2) Meters	18	18	-	-	-	-	-	-	-	-	-	-	-	-	-	36
	b) Vehicles	61	60	-	-	-	-	-	-	-	-	-	-	-	-	-	121
	c) Other Equipment	278	277	-	-	-	-	-	-	-	-	-	-	-	-	-	555
	d) Miscellaneous System Improvements	73	72	-	-	-	-	-	-	-	-	-	-	-	-	-	145
Sub-total 1/		508	1,085	2,685	4,795	3,292	1,999	2,051	2,654	3,538	2,769	1,804	1,608	1,608	1,609	803	32,808
Land		175	-	-	-	-	-	70	-	-	-	-	-	-	-	-	245
TOTAL PROJECT COST 2/		683	1,085	2,685	4,795	3,292	1,999	2,121	2,654	3,538	2,769	1,804	1,608	1,608	1,609	803	33,053

1/ Includes design (first year of each major segment of development), supervision of construction, contingencies, legal and administrative costs spread uniformly during the period of construction.

2/ Does not include interest during construction. For calculated interest see Table X-G-1.

APPENDIX TABLE X-B-2

PROJECT COST OF RECOMMENDED PROGRAM
LIPA CITY WATER DISTRICT
(WITH ESCALATION)
(P x 1000)

Item	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	TOTAL
Escalation Factor	1.00	1.10	1.21	1.33	1.46	1.58	1.71	1.85	1.99	2.15	2.28	2.42	2.56	2.71	2.88	
Source Development a) Structures	-	146	991	2,006	1,099	-	140	938	1,855	1,002	-	-	-	-	-	8,177
b) Equipment	-	74	501	1,013	556	-	72	485	957	516	-	-	-	-	-	4,174
Transmission Distribution Mains and Valves	-	179	655	1,228	1,348	1,580	1,412	1,071	1,152	1,382	1,352	1,162	1,229	1,301	691	15,742
Storage Tanks and Appurtenances																
a) Structures	..	4	29	57	32	-	51	474	901	486	-	-	-	-	-	2,034
b) Equipment	-	3	22	44	25	-	12	104	197	108	-	-	-	-	-	515
Internal Network	-	78	287	537	590	722	773	731	786	993	1,186	1,237	1,308	1,385	734	11,347
Service Connections a) Pipes	-	69	253	472	518	660	816	877	943	1,140	1,135	999	1,057	1,122	596	10,657
b) Meters	-	21	77	142	156	196	231	231	249	327	440	494	522	553	291	3,930
Water District Buildings	-	64	434	878	482	-	-	-	-	-	-	-	-	-	-	1,858
Early Action Works a) Service Connections																
1) Pipes	78	85	-	-	-	-	-	-	-	-	-	-	-	-	-	163
2) Meters	18	20	-	-	-	-	-	-	-	-	-	-	-	-	-	38
b) Vehicles	61	66	-	-	-	-	-	-	-	-	-	-	-	-	-	127
c) Other Equipment	278	305	-	-	-	-	-	-	-	-	-	-	-	-	-	583
d) Miscellaneous Improvements	73	79	-	-	-	-	-	-	-	-	-	-	-	-	-	152
Sub-total	508	1,193	3,249	6,377	4,806	3,158	3,507	4,911	7,040	5,954	4,113	3,892	4,116	4,361	2,312	59,497
Land	175	-	-	-	-	-	120	-	-	-	-	-	-	-	-	295
TOTAL PROJECT COST	683	1,193	3,249	6,377	4,806	3,158	3,627	4,911	7,040	5,954	4,113	3,892	4,116	4,361	2,312	59,792

APPENDIX TABLE L-8-1

ASSETS AND DEPRECIABLE VALUE FORECAST
LIPA CITY WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
I. WORK-IN-PROCESS																									
Source Development																									
a) Structures	-	146	1137	3143	4242	-	140	1078	2933	3935	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Equipment	-	74	575	1588	2144	-	72	557	1514	2030	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Storage Tanks and Appurtenances																									
a) Structures	-	4	33	90	122	-	51	525	1426	1912	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Equipment	-	3	25	69	94	-	12	116	313	421	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Water District Buildings	-	64	498	1376	1850	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total Work-in-Process	-	291	2268	6266	8460	-	275	2276	6186	8298	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
II. ASSETS ADDED BY YEAR END																									
Transmission Distribution																									
Mains and Valves	-	179	695	1228	1348	1580	1412	1071	1152	1382	1352	1162	1229	1308	691	-	-	-	-	-	-	-	-	-	-
External Network	-	78	207	537	590	722	773	731	766	993	1186	1237	1308	1385	734	-	-	-	-	-	-	-	-	-	-
Service Connections																									
a) Pipes	-	69	253	472	538	660	816	877	943	1140	1135	999	1057	1122	596	-	-	-	-	-	-	-	-	-	-
b) Meters	-	21	77	142	156	196	231	231	269	327	400	494	522	553	298	-	-	-	-	-	-	-	-	-	-
Early Action Works																									
a) Service Connections																									
1) Pipes	78	85	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
2) Meters	18	20	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Vehicles	61	66	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
c) Other Equipment	278	305	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
d) Miscellaneous Improvements	73	79	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
III. REPLACEMENTS																									
Existing Facilities	-	-	-	-	1070	-	1927	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1140	-	-
Meters	-	-	-	-	-	-	-	20	-	-	-	-	-	-	-	55	132	264	535	601	890	1000	1152	1230	2086
Vehicles	-	-	-	-	-	-	-	113	119	-	-	-	-	-	376	183	-	-	-	-	-	264	275	-	-
Total Assets Added by Year End	508	902	1232	2379	3702	3158	5159	3043	3249	3842	4113	3892	4116	4361	2695	238	132	264	535	601	890	1264	2575	1230	2086
IV. DEPRECIABLE VALUE																									
A. 50 Years Service Life																									
Existing Facilities	[illegible]	3373	3373	3373	8600	3720	3583	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4510	4080	5080	5080
Structures																									
a) Source Development	-	-	-	-	-	4062	4062	4062	4062	4062	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177	8177
b) Storage Tanks	-	-	-	-	-	122	122	122	122	122	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034	2034
Transmission Distribution Mains and Valves	-	-	179	834	2082	3430	4990	6402	7473	8825	10007	11359	12521	13750	15051	15702	15702	15702	15702	15702	15702	15702	15702	15702	15702
External Network	-	-	78	305	902	1492	2214	3007	3738	4504	5497	6683	7900	9208	10623	11347	11347	11347	11347	11347	11347	11347	11347	11347	11347

APPENDIX TABLE X-B-1 (Continued)

ASSETS AND DEPRECIABLE VALUE FORECAST
LIPA CITY WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
Service Connections (Meters)	-	-	69	322	794	1312	1972	2788	3665	4608	5748	6883	7882
Water District Buildings	-	-	-	-	-	1858	1858	1858	1858	1858	1858	1858	1858
Early Action Works													
a) Miscellaneous Improvements	-	73	152	152	152	152	152	152	152	152	152	152	152
b) Service Connections (Pipes)	-	78	161	161	161	161	161	161	161	161	161	161	161
Total 50 Years Service Life	3373	3524	4014	5209	6713	16461	18296	23224	25903	28784	38146	41819	45217
B. 25 Years Service Life													
Equipment													
a) Source Development	-	-	-	-	-	2144	2144	2144	2144	2144	4174	4174	4174
b) Storage Tank	-	-	-	-	-	94	94	94	94	94	515	515	515
Early Action Works (Other Equipment)	-	278	583	583	583	583	583	583	583	583	583	583	583
Total 25 Years Service Life	-	278	583	583	583	2821	2821	2821	2821	2821	5272	5272	5272
C. 15 Years Service Life													
Existing and Replacements	11	11	11	11	11	11	11	-	20	20	20	20	20
Service Connections (Meters)	-	18	59	136	278	434	630	861	1092	1341	1668	2108	2602
Total 15 Years Service Life	11	29	70	147	289	445	641	861	1112	1361	1688	2128	2622
D. 7 Years Service Life													
Vehicles	-	61	127	127	127	127	127	66	113	232	232	232	232
Total 7 Years Service Life	-	61	127	127	127	127	127	66	113	232	232	232	232
Total Depreciable Value	3384	[illegible]	4794	6066	7712	1985	21885	26972	29949	33198	45338	49451	53343
Book Value of Assets Other Than Land	3892	5085	8334	14711	19854	23012	27319	32291	39384	45338	49451	53343	57459
Land	175	175	175	175	175	175	295	295	295	295	295	295	295
TOTAL BOOK VALUE OF ALL CAPITAL ASSETS	4067	5260	8509	14886	20029	23187	27614	32586	39679	45633	49746	53638	57754

	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Service Connections (Meters)	8939	10061	10657	10657	10657	10657	10657	10657	10657	10657	10657	10657
Water District Buildings	1858	1858	1858	1858	1858	1858	1858	1858	1858	1858	1858	1858
Early Action Works												
a) Miscellaneous Improvements	152	152	152	152	152	152	152	152	152	152	152	152
b) Service Connections (Pipes)	161	161	161	161	161	161	161	161	161	161	161	161
Total 50 Years Service Life	48811	52619	54640	54640	54640	54640	54640	54640	54640	54390	55538	55538
B. 25 Years Service Life												
Equipment												
a) Source Development	4174	4174	4174	4174	4174	4174	4174	4174	4174	4174	4174	4174
b) Storage Tank	515	515	515	515	515	515	515	515	515	515	515	515
Early Action Works (Other Equipment)	583	583	583	583	583	583	583	583	583	583	583	583
Total 25 Years Service Life	5272	5272	5272	5272	5272	5272	5272	5272	5272	5272	5272	5272
C. 15 Years Service Life												
Existing and Replacements	20	20	20	75	207	471	986	1587	2307	3367	4519	5789
Service Connections (Meters)	3124	3677	3950	3909	3832	3690	3534	3338	3107	2876	2627	2380
Total 15 Years Service Life	3144	3697	3970	3984	4039	4161	4520	4905	5494	6243	7146	[illegible]
D. 7 Years Service Life												
Vehicles	232	119	176	359	359	359	359	359	183	264	539	539
Total 7 Years Service Life	232	119	176	359	359	359	359	359	183	264	539	539
Total Depreciable Value	57459	61707	64058	64255	64310	64432	64791	65196	[illegible]	66109	[illegible]	[illegible]
Book Value of Assets Other Than Land	61820	64195	64296	64387	64574	64967	65398	[illegible]	66853	67744	[illegible]	[illegible]
Land	295	295	295	295	295	295	295	295	295	295	295	295
TOTAL BOOK VALUE OF ALL CAPITAL ASSETS	62115	64490	64591	[illegible]	64869	65262	65687	[illegible]	67148	[illegible]	[illegible]	71357

APPENDIX TABLE X-E-2

SCHEDULE OF DEPRECIATION EXPENSES
LIPA CITY WATER DISTRICT
(P x 1000)

Year	Service Life Category: 50 Years	25 Years	15 Years	7 Years	Total Annual Deprec. Expenses	Accum. Deprec. Prior Year	Book Value of Assets Retired During The Year: 50 Years	25 Years	15 Years	7 Years	Total	Net Accum. Deprec. Year End
1976	67	-	-	-	67	2,143	-	-	-	-	-	2,210
1977	70	11	2	9	92	2,210	-	-	-	-	-	2,302
1978	80	23	5	18	126	2,302	-	-	-	-	-	2,428
1979	104	23	10	18	155	2,428	-	-	-	-	-	2,583
1980	134	23	19	18	194	2,583	733	-	-	-	733	2,044
1981	329	113	30	18	490	2,044	-	-	-	-	-	2,534
1982	366	113	43	18	540	2,534	1,127	-	-	-	1,127	1,947
1983	464	113	57	9	643	1,947	-	-	11	61	72	2,518
1984	518	113	74	16	721	2,518	-	-	-	66	66	3,173
1985	576	113	91	33	813	3,173	-	-	-	-	-	3,986
1986	763	211	112	33	1,119	3,986	-	-	-	-	-	5,105
1987	836	211	142	33	1,222	5,105	-	-	-	-	-	6,327
1988	904	211	175	33	1,323	6,327	-	-	-	-	-	7,650
1989	976	211	210	33	1,430	7,650	-	-	-	-	-	9,080
1990	1,052	211	246	17	1,526	9,080	-	-	-	113	113	10,493
1991	1,093	211	265	25	1,594	10,493	-	-	18	119	137	11,950
1992	1,093	211	266	51	1,621	11,950	-	-	41	-	41	13,530
1993	1,093	211	269	51	1,434	13,530	-	-	47	-	77	14,887
1994	1,093	211	277	51	1,632	14,887	-	-	142	-	142	16,377
1995	1,093	211	301	51	1,656	16,377	-	-	156	-	156	17,877
1996	1,093	211	328	51	1,683	17,877	-	-	196	-	196	19,364
1997	1,093	211	366	26	1,696	19,364	-	-	231	176	407	20,653
1998	1,088	211	416	38	1,753	20,653	250	-	251	183	684	21,722
1999	1,111	211	476	77	1,875	21,722	-	-	249	-	249	23,348
2000	1,111	211	535	77	1,934	23,348	-	-	327	-	327	24,955

APPENDIX TABLE X-E-3

WORKING CAPITAL REQUIREMENTS
FOR REVOLVING FUND FOR NEW CONNECTIONS
LIPA CITY WATER DISTRICT

Year	Number of New Connections	Number of Installment Plan Added	Number of Installment Plan Paid	Total Paying Monthly Installment (Cumulative)	Monthly Installment Plan (Escalated)	Increment Added[1]	Increment Deducted[2]	Cash Receipts (P x 1000): Lump Sum[3] Payments (Escalated)	Cash Receipts (P x 1000): Installment Payments (Cumulative)	Cash Receipts (P x 1000): Total Payments	Annual Construction Cost[4] (P x 1000)	Working Capital Required (P x 1000)	Cumulative Capital Requirements (P x 1000)
1976	78	47	∅	47	5.65	3	∅	17	2	19	43	24	24
1977	79	47		94	6.22	4		19	5	24	48	24	48
1978	79	47		141	6.84	4		21	9	30	53	23	71
1979	475	285		426	7.51	26		140	24	164	350	186	257
1980	475	285		711	8.25	28		154	51	205	384	179	436
1981	550	330		1,041	8.93	35		193	83	276	482	206	642
1982	550	330		1,371	9.66	38		209	119	328	522	194	836
1983	500	300		1,671	10.45	38		205	157	362	512	150	986
1984	500	300		1,971	11.24	40		220	196	416	551	135	1,121
1985	500	300	∅	2,271	12.15	44	∅	238	238	476	596	120	1,241
1986	780	468	24	2,715	12.88	72	2	394	294	688	985	297	1,538
1987	780	468	47	3,136	13.65	77	4	418	365	783	1,045	262	1,800
1988	780	468	47	3,557	14.46	81	4	442	440	882	1,106	224	2,024
1989	780	468	166	3,859	15.31	86	15	468	508	976	1,171	195	2,219
1990	780	468	285	4,042	16.27	91	27	498	570	1,068	1,244	176	2,395
1991	∅	∅	308	3,734	17.23	∅	32	∅	583	583	∅	(583)	1,812
1992			330	3,404	18.24		37		546	546		(546)	1,266
1993			315	3,089	19.38		38		508	508		(508)	758
1994			300	2,789	20.51		39		469	469		(469)	289
1995			300	2,489	21.75		42		427	427		(427)	(138)
1996			384	2,105	23.05		58		369	369		(369)	(507)
1997			468	1,637	24.46		75		294	294		(294)	(801)
1998			468	1,169	25.93		79		215	215		(215)	(1,016)
1999			468	701	27.46		84		131	131		(131)	(1,147)
2000	∅	∅	468	233	29.10	∅	89	∅	42	42	∅	(42)	(1,189)

1/ Accumulated installment payments are calculated on the basis of 100 per cent incremental addition during previous years and 50 per cent of the last year.
2/ Based on the assumption that installment plan will be paid back in 10 years.
3/ Assumed to be 40 per cent of construction cost.
4/ Amount to be shouldered by the customers which is 2/3 the average cost of pipes plus meters.

APPENDIX TABLE X-B-4

REVENUE UNIT FORECAST
LIPA CITY WATER DISTRICT

Type of Connection By Meter Size	1976					1980					1985			1990		
	Number of Connections 1/	Prop. of Consumption 2/	Estimated Consumption	Use Factor	Total RUs 3/	Number of Connections	Prop. of Consumption	Estimated Consumption	Use Factor	Total RUs	Number of Connections	Estimated Consumption	Total RUs	Number of Connections	Estimated Consumption	Total RUs
Domestic																
1/2-inch	708	100	570	1	570	1,698	100	1,840	1	1,840	4,021	3,900	3,900	7,506	7,200	7,200
Sub-total	708	100	570		570	1,698		1,840		1,840	4,021	3,900	3,900	7,506	7,200	7,200
Commercial and Industrial																
1/2-inch	81	100	112	2	224	164	67	214	2	428	390	322	644	729	590	1,180
3/4-inch						19	72	38	2	76	46	58	116	86	106	212
1-inch						10	13	42	2	84	23	62	124	42	114	228
2-inch (wholesale)						1	8	26	3	78	1	38	114	2	70	210
Sub-total	81	100	112		224	194	100	320		666	460	480	998	859	880	1,830
Institutional																
1/2-inch	3	100	28	2	56	6	56	45	2	90	16	67	134	29	123	246
3/4-inch						1	15	12	2	24	2	18	36	4	33	66
1-inch						1	29	23	2	46	1	35	70	2	64	128
Sub-total	3	100	28		56	8	100	80		160	19	120	240	35	220	440
TOTAL	792		710		850	1,900		2,240		2,666	4,500	4,500	5,138	8,400	8,300	9,470

1/ 1975 figures are actual; 1980, 1985 and 1990 are estimated with the proportion of connections in each size remaining constant.

2/ Proportion of consumption based on flow relationship.

3/ Includes both "service RUs" and "commodity RUs"; The effect of minimum monthly charges will be to increase total RUs since there will always be some customers not using the basic quantity of water allowed within the minimum price.

APPENDIX TABLE X-F-1

REVENUE FORECASTS
LIPA CITY WATER DISTRICT

Year	Rate/RU P	Estimated Number of RUs (Yearly in 000s)	Income from Sales	(Bad Debt)	Other Income[10]	Total Net Income
1976	1.00	310	310	6	6	310
1977	1.00	476	476	5	10	481
1978	1.00	641	641	6	13	648
1979	1.90	807	1,533	31	31	1,533
1980	1.90	973	1,849	18	37	1,868
1981	1.90	1,153	2,191	22	44	2,213
1982	2.45	1,334	3,268	65	65	3,268
1983	2.45	1,514	3,709	37	74	3,746
1984	2.45	1,695	4,153	42	83	4,194
1985	2.80	1,875	5,250	105	105	5,250
1986	2.80	2,191	6,135	61	123	6,197
1987	2.80	2,508	7,022	70	140	7,092
1988	2.95	2,824	8,331	167	167	8,331
1989	2.95	3,141	9,266	93	185	9,358
1990	2.95	3,457	10,198	102	204	10,300
1991	3.00		10,371	207	207	10,371
1992	3.00		10,371	104	207	10,474
1993	3.00		10,371	104	207	10,474
1994	3.30		11,408	228	228	11,408
1995	3.30		11,408	114	228	11,522
1996	3.30		11,408	114	228	11,522
1997	3.70		12,791	256	256	12,791
1998	3.70		12,791	128	256	12,919
1999	3.70		12,791	128	256	12,919
2000	3.70	3,457	12,791	128	256	12,919

[10] Other income (derived from meter replacement charges, contingency fees of new connections, service fees, etc.) is about two per cent of sales.

APPENDIX TABLE X-G-1

FINANCING PLAN AND DEBT SERVICE
LIPA CITY WATER DISTRICT
(₱ x 1000)

Fiscal Year	Total Capital Expenditure	Cash Sources: Revolving Fund Revenues	Cash Sources: Amount Disbursed	Loan Disbursements and Debt Service: Outstanding Debt Start of Year	Loan Disbursements and Debt Service: Amortized During Year	Loan Disbursements and Debt Service: Outstanding Debt End of Year	Interest	Total Debt Service
1976	683	19	664	-	-	664	-	-
1977	1,193	24	1,169	664	-	1,833	60	60
1978	3,249	30	3,219	1,833	-	5,052	165	165
1979	6,377	164	6,213	5,052	-	11,265	454	454
1980	4,806	205	4,601	11,265	-	15,866	1,014	1,014
1981	3,158	276	2,882	15,866	-	18,748	1,428	1,428
1982	3,627	328	3,299	18,748	-	22,047	1,687	1,687
1983	4,911	362	4,549	22,047	165	26,431	1,984	2,149
1984	7,040	416	6,624	26,431	165	32,890	2,379	2,544
1985	5,954	476	5,478	32,890	165	38,203	2,960	3,125
1986	4,113	688	3,425	38,203	248	41,380	3,438	3,686
1987	3,892	783	3,109	41,380	248	44,241	3,724	3,972
1988	4,116	882	3,234	44,241	248	47,227	3,982	4,230
1989	4,361	976	3,385	47,227	414	50,198	4,250	4,664
1990	2,312	1,068	1,244	50,198	414 11/	51,028	4,518	4,932
1991	ø	ø	ø	51,028	812	50,216	4,592	5,404
1992				50,216	812	49,404	4,519	5,331
1993				49,404	976	48,428	4,446	5,422
1994				48,428	1,092	47,336	4,358	5,450
1995				47,336	1,092	46,244	4,260	5,352
1996				46,244	1,092	45,152	4,162	5,254
1997				45,152	1,493	43,659	4,064	5,557
1998				53,659	1,493	52,166	3,929	5,422
1999				42,166	1,726	40,440	3,795	5,521
2000	ø	ø	ø	40,440	1,726	38,714	3,640	5,366

11/ Includes payments for second loan (1983–1990).

APPENDIX TABLE I-G-2

PROJECTED INCOME STATEMENT
LIPA CITY WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
Water Production per Year (cum x 1,000)	770 —		934 —		1358 —		1570 —			2190 —			
Water Sales per Year (cum x 1,000)	259 —		467 —		818 —		1102 —			1643 —			
Unaccounted-for-water (%)	66 12/ —		50 —		40 —		30 —			25 —			
Connections: Metered	356	653	950	1425	1900	2450	3000	3500	4000	4500	5280	6060	6840
Unmetered	434	218	–	–	–	–	–	–	–	–	–	–	–
Consumption (lpcd)	116	117	118	120	121	122	123.3	124.5	125.7	127	128.3	129.5	130.8
OPERATING REVENUE													
Water Sales	310	476	641	1533	1849	2191	3268	3709	4153	5250	6135	7022	8331
Less: Uncollectibles	6	5	6	31	18	22	65	37	42	105	61	70	157
Other Revenue	6	10	13	11	17	44	65	74	83	105	123	140	167
Total Revenue	310	481	648	1533	1868	2213	3268	3746	4194	5250	6197	7092	8331
OPERATING EXPENSES													
Administration and Personnel	86	97	122	142	194	216	238	257	278	452	518	566	617
Source Facilities	30	35	37	50	165	214	271	334	402	490	602	688	793
Transmission and Distribution	17	21	24	29	35	46	59	72	89	106	127	152	176
Water Treatment Facilities	14	17	21	25	30	35	41	48	56	66	76	89	101
Miscellaneous	13	14	16	18	20	24	29	33	39	44	52	58	65
Depreciation	67	92	126	155	194	490	540	643	721	811	1119	1222	1323
Total Operating Expenses	227	276	346	419	638	1025	1178	1387	1585	1971	2494	2775	3075
Operating Income	83	205	302	1114	1230	1188	2090	2359	2609	3279	3703	4317	5256
Plus: Interest on Reserves	1	2	4	8	15	28	51	80	113	152	217	309	416
Net Income Before Interest	84	207	306	1122	1245	1216	2141	2439	2722	3431	3920	4626	5672
Interest on Debt	–	60	165	454	1014	1428	1687	1984	2379	2960	3438	3724	3982
Net Income (Loss)	84	147	141	668	231	(212)	454	455	343	471	482	902	1690
Cumulative Net Income (Loss)	84	231	372	1040	1271	1059	1513	1968	2311	2782	3264	4166	5856
Appropriation to Reserves	9	14	19	46	55	131	196	223	249	315	614	702	833
Average Net Fixed Assets in Operation	1549	2762	3240	4925	7781	15089	23023	26592	29056	31835	38995	45976	48708
Rate of Return	5.3	9.0	9.3	22.6	15.8	7.9	9.1	8.9	9.0	10.3	9.5	9.4	10.8

	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Water Production per Year (cum x 1,000)	—	3906	3906	3906	3906	3906	3906	3906	3906	3906	3906	3906
Water Sales per Year (cum x 1,000)	—	3030	3030	3030	3030	3030	3030	3030	3030	3030	3030	3030
Unaccounted-for-water (%)	—	22	22	22	22	22	22	22	22	22	22	22
Connections: Metered	7620	8400	8400	8400	8400	8400	8400	8400	8400	8400	8400	8400
Unmetered	–	–	–	–	–	–	–	–	–	–	–	–
Consumption (lpcd)	132	135	135	135	135	135	135	135	135	135	135	135
OPERATING REVENUE												
Water Sales	9266	10198	10371	10371	10371	11408	11408	11408	12791	12791	12791	12791
Less: Uncollectibles	93	102	207	104	104	228	114	114	256	128	128	128
Other Revenue	185	204	207	207	207	228	228	228	256	256	256	256
Total Revenue	9358	10300	10371	10474	10474	11408	11522	11522	12791	12919	12919	12919
OPERATING EXPENSES												
Administration and Personnel	666	802	866	935	1010	1091	1176	1272	1374	1484	1603	1731
Source Facilities	911	1084	1170	1264	1365	1474	1593	1720	1858	2006	2166	2340
Transmission and Distribution	206	235	254	274	296	320	345	373	403	435	470	507
Water Treatment Facilities	114	129	140	151	163	176	190	205	221	229	258	279
Miscellaneous	76	88	95	103	111	120	219	140	251	163	176	190
Depreciation	1430	1526	1594	1621	1434	1632	1656	1681	1696	1751	1375	1934
Total Operating Expenses	3401	3864	4119	4348	4379	4813	5091	5393	5703	6080	6548	6981
Operating Income	5957	6436	6252	6126	6095	6595	6431	6129	7088	6839	6371	5938
Plus: Interest on Reserves	539	675	819	964	1109	1261	1421	1581	1750	1929	2108	2287
Net Income Before Interest	6496	7111	7071	7090	7204	7856	7852	7710	8838	8768	8479	8225
Interest on Debt	4250	4518	4592	4519	4446	4358	4260	4162	4064	3929	3795	3640
Net Income (Loss)	2246	2593	2479	2571	2758	3500	3592	3548	4774	4839	4684	4585
Cumulative Net Income (Loss)	8102	10695	13174	15745	18503	22003	25595	29143	33917	38756	[illegible]	[illegible]
Appropriation to Reserves	927	1020	1037	1037	1037	1141	1141	1141	1279	1279	1279	1279
Average Net Fixed Assets in Operation	51570	53516	53319	51897	50567	49424	48338	47369	46711	46906	46985	46527
Rate of Return	11.2	12.0	11.7	11.8	12.1	13.3	13.3	12.9	15.2	14.6	13.4	12.8

12/ Based on October 1975 Billing.

APPENDIX TABLE E-G-3

PROJECTED SOURCES AND APPLICATION OF FUNDS
LIPA CITY WATER DISTRICT
₱ x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
SOURCES OF FUNDS													
Net Income Before Interest	84	207	306	1122	1245	1216	2141	2439	2722	3431	3920	4626	5672
Add: Depreciation	67	92	126	155	194	490	540	643	721	813	1119	1222	1323
Total Internal Cash Generation	151	299	432	1277	1439	1706	2681	3082	3443	4244	5039	5848	6995
Long-Term Borrowing	664	1169	3219	6213	4601	2882	3299	4549	6624	5478	3425	3109	3234
Capital Contributions	19	24	30	164	205	276	328	1162	416	476	688	783	882
Total External Cash Generation	683	1193	3249	6377	4806	3158	3627	4911	7040	5954	4113	3892	4116
TOTAL SOURCES OF FUNDS	834	1492	3681	7654	6245	4864	6308	7993	10483	10198	9152	9740	11111
APPLICATIONS OF FUNDS													
Capital Expenditures	683	1193	3249	6377	4806	3158	3627	4911	7040	5954	4113	3892	4116
Debt Service: Interest	–	60	165	454	1014	1428	1687	1984	2379	2960	3438	3724	3982
Principal	–	–	–	–	–	–	–	165	165	165	248	248	248
Sub-total	–	60	165	454	1014	1428	1687	2149	2544	3125	3686	3972	4230
Replacements	–	–	–	–	1070	–	1727	133	–	–	–	–	–
Decrease in Working Capital	35	42	51	321	75	91	105	172	114	146	232	222	302
TOTAL APPLICATIONS OF FUNDS	752	1295	3508	7152	[illegible]	4677	7545	7315	9717	9225	8031	8086	8648
Increase (Decrease) in Cash Balance	65	197	173	502	(722)	187	(1235)	678	656	873	1121	1647	2463
Cash Balance Beginning of Year	0	65	262	435	937	217	404	(834)	(156)	460	1333	2454	4101
Cash Balance End of Year	65	262	435	937	217	404	(834)	(156)	470	1333	2454	4101	6564
Debt-Service Ratio	–	5.00	2.61	2.8	1.42	1.19	1.59	1.43	1.35	1.36	1.37	1.47	1.65

	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
SOURCES OF FUNDS												
Net Income Before Interest	4696	7111	7072	7090	7204	7658	7652	7710	8838	8768	8479	8225
Add: Depreciation	1430	1526	1594	1621	1434	1632	1656	1683	1696	1751	1875	1934
Total Internal Cash Generation	7926	8637	8665	8711	8638	9490	9508	9393	10534	10521	10354	10159
Long-Term Borrowing	3385	1244	∅ →									∅
Capital Contributions	876	1068	583	546	508	469	427	369	294	215	131	42
Total External Cash Generation	4361	2312	583	546	508	469	427	369	294	215	131	42
TOTAL SOURCES OF FUNDS	12287	10949	9248	9257	9146	9959	9935	9762	10828	10736	10485	10201
APPLICATIONS OF FUNDS												
Capital Expenditures	4361	2312	∅ →									∅
Debt Service: Interest	4250	4518	4592	4519	4446	4358	4260	4162	4064	3929	3795	3640
Principal	414	414	812	812	976	1092	1092	1092	1493	1493	1726	1726
Sub-total	4664	4932	5404	5331	5422	5450	5352	5254	5557	5422	5521	5366
Replacements	–	176	238	132	264	515	601	800	1264	2575	1210	1684
Decrease in Working Capital	252	(88)	(258)	21	12	278	17	27	332	36	(111)	103
TOTAL APPLICATIONS OF FUNDS	9277	7332	5254	5484	5698	6243	5970	6081	7153	8033	[illegible]	7153
Increase (Decrease) in Cash Balance	3010	3617	3894	3773	3448	3716	3965	3681	3675	2703	3787	3048
Cash Balance Beginning of Year	6564	9574	13191	17085	20858	24306	28022	31987	35668	39343	43046	[illegible]
Cash Balance End of Year	9574	13191	17085	20858	24306	28022	31987	35668	39343	43046	[illegible]	[illegible]
Debt-Service Ratio	1.70	1.75	1.60	1.63	1.59	1.74	1.78	1.79	1.90	1.94	1.88	1.89

APPENDIX TABLE X-G-4

PROJECTED BALANCE SHEET
LIPA CITY WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
ASSETS												
FIXED ASSETS:												
Gross Value of Fixed Assets	4067	4959	6241	8620	11569	23187	27339	30310	33493	37335	49746	53638
Less: Accumulated Depreciation	2210	2302	2428	2583	2044	2534	1947	2518	3173	3986	5105	6327
Net Value of Fixed Assets	1857	2657	3813	6037	9525	20653	25392	27792	30320	33349	44641	47311
Work in Process	–	291	2268	6266	8460	–	275	2276	6186	8298	–	–
Total Fixed Assets	1857	2958	6081	12303	17985	20653	25667	30068	36506	41647	44641	47311
CURRENT ASSETS:												
Cash	65	262	435	937	217	406	(834)	(206)	460	1333	2454	4101
Accounts Receivables	77	119	160	383	462	548	817	927	1038	1313	1534	1756
Provision for Bad Debts	(1)	(1)	(2)	(8)	(5)	(5)	(16)	(9)	(10)	(26)	(15)	(18)
Inventories	37	41	101	212	235	255	319	392	416	452	488	528
Total Current Assets	178	421	694	1524	909	1202	286	1104	1904	3072	4461	6367
TOTAL ASSETS	2035	3379	6775	13827	18894	21855	25953	31172	38410	44719	49102	53678
EQUITY AND LIABILITIES												
CURRENT LIABILITIES:												
Accounts Payable	27	31	37	44	74	89	106	124	144	193	229	259
Current Maturities of Long-term Debt	–	–	–	–	–	–	165	165	165	248	248	248
Total Current Liabilities	27	31	37	44	74	89	271	289	309	441	477	507
Long-term Debt (Less: Current Maturities)	664	1833	5052	11265	15866	18748	21882	26266	32725	37955	41132	43993
EQUITY:												
Government Contribution	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241
Capital Contribution	19	43	73	237	442	718	1044	1408	1824	2300	2988	3771
Reserves	9	23	42	88	143	276	470	693	942	1257	1871	2573
Unappropriated Retained Earnings	75	208	130	952	1128	765	1043	1275	1369	1525	1393	1393
Total Equity	1344	1515	1686	2518	2954	3018	3800	4617	5376	6323	7493	9178
TOTAL EQUITY AND LIABILITIES	2035	3379	6775	13827	18894	21855	25953	31172	38410	44719	49102	53678

	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
ASSETS													
FIXED ASSETS:													
Gross Value of Fixed Assets	57754	62115	64490	64591	64632	64869	65242	65687	66291	67148	69039	70080	71357
Less: Accumulated Depreciation	7650	9080	10491	11750	13530	14887	16377	17877	19364	20651	21722	[illegible]	[illegible]
Net Value of Fixed Assets	50104	53035	53997	52641	51152	49982	48865	47810	46927	46495	47317	46652	46402
Work in Process	–	–	–	–	–	–	–	–	–	–	–	–	–
Total Fixed Assets	50104	53035	53997	52641	51152	49982	48865	47810	46927	46495	47317	46652	46402
CURRENT ASSETS:													
Cash	6564	9574	13191	17085	20858	24306	28022	31987	35668	39343	42066	45883	[illegible]
Accounts Receivables	2083	2317	2550	2593	2593	2593	2852	2852	2852	3197	3197	3197	3197
Provision for Bad Debts	(42)	(23)	(26)	(52)	(26)	(26)	(57)	(29)	(29)	(64)	(32)	(32)	(32)
Inventories	560	596	339	65	94	142	231	263	335	407	[illegible]	[illegible]	[illegible]
Total Current Assets	9165	12464	16054	19691	23519	27015	31048	35973	38626	42883	45675	49487	52700
TOTAL ASSETS	59269	65499	70051	72332	74671	76997	79913	[illegible]	85753	89378	92992	96139	99102
EQUITY AND LIABILITIES													
CURRENT LIABILITIES:													
Accounts Payable	292	320	390	421	455	491	530	573	618	668	721	779	841
Current Maturities of Long-term Debt	414	414	812	812	976	1092	1092	1092	1493	1493	1726	1726	[illegible]
Total Current Liabilities	706	743	1202	1233	1431	1583	1622	1665	2111	2161	2447	2505	3046
Long-term Debt (Less: Current Maturities)	46813	49784	50216	40404	49429	47336	47244	45152	43659	42166	40449	38714	36909
EQUITY:													
Government Contribution	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241	1241
Capital Contribution	4653	[illegible]	6697	7280	7286	8334	8803	9230	9599	9893	10109	10239	[illegible]
Reserves	3406	4333	5353	6390	7427	8464	9605	10746	11887	13166	14441	15734	17003
Unappropriated Retained Earnings	2450	3769	5342	6784	8318	[illegible]	[illegible]	14849	17256	20751	[illegible]	[illegible]	[illegible]
Total Equity	11750	14972	18633	21695	24812	[illegible]	32047	36066	39983	[illegible]	[illegible]	[illegible]	[illegible]
TOTAL EQUITY AND LIABILITIES	59269	65499	70051	72332	74671	76997	79913	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

APPENDIX TABLE X-C-5

RATE OF RETURN ON TOTAL INVESTMENT
(DISCOUNTED CASH FLOW METHOD)
LIPA CITY WATER DISTRICT
(₱ x 1000)

Year	Debt Service	Net Increase in Cash	Total Cash Inflow	Investments	Net Cash Inflow	1st Trial Present Value: 8% Factor	1st Trial Present Value: 8% Value	2nd Trial Present Value: 10% Factor	2nd Trial Present Value: 10% Value
1976	-	(65)	(65)	683	(618)	1.000	(618)	1.000	(618)
1977	60	197	257	1,193	(936)	0.926	(867)	0.909	(851)
1978	165	173	338	3,249	(2,911)	0.857	(2,495)	0.826	(2,404)
1979	454	502	956	6,377	(5,421)	0.794	(4,304)	0.751	(4,071)
1980	1,014	(732)	282	5,876	(5,594)	0.735	(4,112)	0.683	(3,821)
1981	1,428	187	1,615	3,158	(1,543)	0.681	(1,051)	0.621	(958)
1982	1,687	(1,238)	449	5,554	(5,105)	0.630	(3,216)	0.564	(2,879)
1983	2,149	628	3,777	5,044	(1,267)	0.583	(739)	0.513	(650)
1984	2,544	666	3,210	7,159	(3,949)	0.540	2,132	0.467	(1,844)
1985	3,125	873	3,998	5,954	(1,956)	0.500	(978)	0.424	(829)
1986	3,686	1,121	4,807	4,113	694	0.463	321	0.386	268
1987	3,972	1,647	7,619	3,892	1,727	0.429	741	0.350	604
1988	4,230	2,463	1,767	4,116	(2,349)	0.397	(933)	0.319	(749)
1989	4,664	3,010	7,674	4,361	3,313	0.368	1,219	0.290	961
1990	4,932	3,617	8,549	2,488	6,061	0.340	2,061	0.263	1,594
1991	5,404	3,894	9,298	238	9,060	0.315	2,854	0.239	2,165
1992	5,331	3,773	9,104	132	8,972	0.292	2,620	0.218	1,956
1993	5,422	3,448	8,870	264	8,606	0.270	2,324	0.198	1,704
1994	5,450	3,716	9,166	515	8,651	0.250	2,163	0.180	1,557
1995	5,352	3,965	9,317	601	8,716	0.232	2,022	0.164	1,429
1996	5,254	3,681	8,935	800	8,135	0.215	1,749	0.149	1,212
1997	5,557	3,675	9,232	1,264	7,968	0.199	1,586	0.135	1,075
1998	5,422	2,703	8,125	2,575	5,550	0.184	1,021	0.123	683
1999	5,521	3,787	9,308	1,210	8,098	0.170	1,377	0.112	907
2000	5,366	3,048	8,414	1,684	17,396 12/	0.158	2,749	0.102	1,774
							+ 7,626		- 1,731

RATE OF RETURN = 9.6%

12/ Includes net asset value of ₱10,666

Total Assets	₱ 99,102
Total Liabilities	(39,555)
Cash	(48,881)
Net Asset Value	₱ 10,666

APPENDIX TO CHAPTER XI

APPENDIX XI-C

QUANTIFIABLE BENEFITS

Increase in Land Values

Appendix Table XI-C-1 shows the present worth of benefits associated with increase in land values, based on the following assumptions:

1. In accordance with the staging program of the construction of facilities, the service area was projected to increase annually by 19.3 ha from 1978 to 1985, by 37 ha from 1985 to 1990 and by 37 ha from 1990 to 2000.

2. The land use distribution of 20 per cent commercial, industrial and institutional; and 80 per cent residential, was used from 1979 to 1985. From 1985 to 2000, the land use was assumed to be 13 per cent commercial, industrial and institutional, and 87 per cent residential. This classification was based on the water demand projections in 1985 and 1990 by consumer category, as shown in Table VI-8, Chapter VI.

3. The 1975 costs of land are:

Residential and Industrial	: ₱ 36 per sqm
Commercial	: ₱ 78 " "

   These costs were assumed to be constant over the projection period.

4. The portion of the total cost of land specifically attributable to the provision of water supply was assumed to be 20 per cent of the cost of land.

5. A discount factor of 12 per cent was used to obtain the present values of the benefits. This is believed to be the opportunity cost of capital and is commonly used for public investment projects like water supply development.

APPENDIX TABLE XI-C-1

PORTION OF LAND VALUES ATTRIBUTABLE TO WATER SUPPLY PROJECT

Year	Land Use (sqm) Comm./Inst./Ind.	Res.	Cost of Land Comm./Inst./Ind.	Res.	Cost of Served Land	20% Benefit Due to To Project	Discount Factor*	Present Value of Benefit
1979	38,600	154,400	₱ 2,991,500	₱ 5,558,400	₱8,549,900	₱ 1,709,980	0.712	₱ 1,217,506
1980	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.636	1,087,547
1981	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.567	969,559
1982	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.507	866,960
1983	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.452	772,911
1984	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.404	690,832
1985	38,600	154,400	2,991,500	5,558,400	8,549,900	1,709,980	0.361	617,303
1986	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.322	986,360
1987	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.288	882,210
1988	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.257	787,250
1989	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.229	701,480
1990	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.205	627,962
1991	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.183	560,571
1992	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.163	499,306
1993	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.146	447,232
1994	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.130	398,220
1995	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.116	355,335
1996	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.104	318,576
1997	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.093	284,880
1998	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.083	254,248
1999	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.074	226,679
2000	48,100	321,900	3,727,750	11,588,400	15,316,150	3,063,230	0.066	202,173
							TOTAL	₱13,755,100

*Discounted at 12 per cent.

Health Benefits

To determine the amount of benefit arising from the reduction of income lost of those afflicted with water-borne diseases, pertinent statistics on morbidity rate were gathered from the Department of Health. From 1963 to 1973, an average of 155.5 out of every 100,000 population in Lipa City were afflicted with primary water-borne diseases every year, regardless of age, sex and income class. The morbidity rate in the study area was assumed to remain constant during the 23-year projection period.

Since not all of those afflicted with said diseases are wage-earners, an adjustment was made accordingly. Based on the 1970 Census on Population and Housing of the National Census and Statistics Office, 36 per cent of the city's population was economically active.[1] It was assumed, therefore, that only 36 per cent of 155.5 per 100,000, who were afflicted with primary water-borne diseases were economically active. Hence, this is the only segment of the population who would suffer a reduction in income due to said diseases. Furthermore, these afflicted wage-earners were assumed to be earning ₱8 a day and unable to work for 15 days on the average because of their illness. The final figure corresponding to the economic cost of time lost due to water-borne diseases was thereby arrived at by multiplying the number of people afflicted with water-borne diseases by 36 per cent, by ₱8 a day and then by 15 days.

Another health benefit that could be associated with the establishment of a safe public water supply system is the reduction of the economic cost of the premature death of those afflicted with water-borne diseases in the study area. Obviously, the reduction of the life span of the population caused by said diseases is an economic loss to the community.

This economic loss due to premature death was determined by multiplying the number of people who die because of water-borne diseases (assuming that a water supply improvement program were not undertaken) by 36 per cent and then by ₱11,629. The projected number of such deaths was based on the average of the 11-year mortality rate for primary water-borne diseases in Lipa City, as gathered from the Department of Health. These figures indicated that 24 persons died of the 155.5 per 100,000 who were afflicted with water-borne diseases. This mortality rate was assumed to be constant over the projection period. The 36 per cent corresponds to the portion of the study area population who are income-earners. The ₱11,629,

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[1] Economically active population includes those who are 10 years old and over, whether employed or unemployed, excluding retired persons, students and housewives.

on the other hand, represents the monetary value of each death. This was derived from the estimated income to be earned by the average wage-earner over a period of five years discounted at 12 per cent plus 20 per cent associated economic costs such as funeral expenses and burial plot (summation of ₱200 a month x 12 months x discount factor + 20 per cent associated costs).

The third health benefit that can be derived from the improvement of the water supply in the study area is the reduction of the medical expenses of persons afflicted with water-borne diseases. According to the Lipa City pilot survey on "Ability to Pay",2/ an afflicted person spends ₱113.00 on the average for medical expenses, which include hospitalization, medicine and doctors' fees. Based on this finding, the total medical expenses incurred due to water-borne diseases were arrived at by multiplying ₱113.00 by the number of people afflicted with such diseases in the study area.

The sum of all three economic costs related to health benefits had to undergo two final adjustments to arrive at more meaningful figures. First, 40 per cent of the total economic loss due to water-borne diseases was taken as the health benefit directly resulting from the water supply improvement program. This reduction was made to account for the fact that not all water-borne diseases are caused by a poor water system and may also be due to less than ideal personal hygiene or lack of sewerage facilities. Second, the 40 per cent health benefit was discounted to its present worth at 12 per cent. Appendix Table XI-C-2 shows the calculations associated with the health benefits for Lipa City. The total present value of said benefits after the adjustments amounts to ₱275,900.

2/Refer to Methodology Manual, Chapter 20 for "Ability to Pay" studies.

APPENDIX TABLE XI-C-2

HEALTH BENEFITS

Year	Study Area Population	Cost of Time Lost Due to Illness	Economic Loss Due to Premature Death	Cost of Medical Expenses	Total Losses w/o Project	40% Reduction 1976 Prices	Discount Factor*	Present Value
1978	67,254	₱3,356	₱ 67,573	₱ 8,778	₱ 79,707	₱31,883	0.797	₱ 25,411
1979	69,375	3,462	69,704	9,054	82,220	32,888	0.712	23,416
1980	71,565	3,571	71,905	9,340	84,816	33,926	0.636	21,577
1981	73,825	3,684	74,175	9,635	87,494	34,998	0.567	49,844
1982	76,155	3,800	76,516	9,939	90,255	36,102	0.507	18,304
1983	78,560	3,920	78,933	10,253	93,106	37,242	0.452	16,833
1984	81,040	4,044	81,425	10,577	96,046	38,418	0.404	15,521
1985	83,600	4,171	83,997	10,911	99,079	39,630	0.361	14,306
1986	85,945	4,288	86,353	11,217	101,858	40,743	0.322	13,119
1987	88,355	4,409	88,774	11,532	104,715	41,886	0.288	12,063
1988	90,835	4,532	91,266	11,855	107,653	43,061	0.257	11,067
1989	93,380	4,659	93,823	12,187	110,669	44,268	0.229	10,737
1990	96,000	4,790	96,456	12,529	113,775	45,510	0.205	9,330
1991							0.183	8,328
1992							0.163	7,418
1993							0.146	6,644
1994							0.130	5,916
1995							0.116	5,279
1996							0.104	4,733
1997							0.093	4,232
1998							0.083	3,777
1999							0.074	3,368
2000	96,000	4,790	96,456	12,529	113,775	45,510	0.066	3,004
							TOTAL	₱294,227

*Discounted at 12 per cent.

Reduction in Fire Insurance Costs

Because of the unavailability of specific information, certain assumptions had to be made in order to quantify the benefit due to the reduction in fire insurance costs:

1. According to the 1970 Census on Housing, there were 15,301 dwelling units in Lipa City of which 13,212 units were made of concrete and galvanized iron.

2. Since not all of the 15,301 units are within the service area which will be provided with fire hydrants, it was assumed that 90 per cent of the 13,212 units made of concrete and galvanized iron (equivalent to 11,890 units) are located in the service area. This 90 per cent assumption was based on the fact that the service area covers almost the entire poblacion of the city where there is usually a big concentration of dwelling units.

3. These 11,890 dwelling units are all made of concrete and galvanized iron and may be considered insurable. However, only 20 per cent (2,378 units) were assumed to be actually insured. These were classified into 20 per cent commercial, industrial and institutional and 80 per cent residential from 1978 to 1985. For the rest of the projection period, they were classified into 13 per cent commercial, industrial and 87 per cent residential. This classification was based on the projected water demand by consumer category in 1985, 1990 and 2000, as shown in Table VI-8, Chapter VI.

4. Based on the study area population projections in Chapter VI, it was assumed that the number of insured commercial, industrial and residential units would increase by 3.2 per cent from 1970 to 1985 and by 2.8 per cent from 1985 to 2000. These are the rates by which the number of households in LCWD service area have been projected to increase, on the assumption that each household consists of 6.5 members.

5. The assumed standard value per unit and the corresponding premium rate for buildings in provincial areas (based on the general tariff rates set by the Philippine Insurance Rating Association) are as follows:

	Value/Unit	Premium/Year
Residential	₱ 75,000	₱ 422.25
Commercial	100,000	1,250.00
Industrial	100,000	1,250.00
Institutional	100,000	500.00

6. The level of fire insurance cost was derived by multiplying the number of insured dwelling units in the core city by their corresponding premiums and summing their products.

7. It is probable that the level of fire insurance costs may be expected to be reduced by 20 per cent because of an improved and plentiful water supply system and increased fire-fighting capabilities.

8. With the development of the area, specifically its urbanization, additional dwelling units made of stronger materials are expected to be constructed. Accompanying this activity, other fire protection techniques in building construction would be considered. While premium rates in general remain constant over a period of years, the quantification of the reduction of fire insurance costs from 1979 to 2000 is nevertheless presented in Appendix Table XI-C-3 to illustrate the impact of an improved water supply system. This benefit for LCWD amounts to ₱2.6 million.

APPENDIX TABLE XI-C-3

REDUCTION IN FIRE INSURANCE COSTS

Year	Insured Units	Com./Ind./ Inst.	Domestic	Insurance Costs Com./ Ind./ Inst.	Insurance Costs Domestic	Total Insurance Cost	20% Reduction Due to Project (1976 Prices)	Discount Factor*	PV of 20% Reduction (1976 Prices)
1979	3,146	630	2,516	₱787,500	₱1,062,381	₱1,849,881	₱ 369,977	0.712	₱ 263,424
1980	3,245	649	2,596	811,250	1,096,161	1,907,411	381,482	0.636	242,623
1981	3,348	670	2,678	837,500	1,130,786	1,968,286	393,657	0.567	223,204
1982	3,454	691	2,763	863,750	1,166,677	2,030,427	406,085	0.507	205,885
1983	3,563	713	2,850	891,250	1,203,413	2,094,663	418,933	0.452	189,358
1984	3,675	735	2,940	918,750	1,241,415	2,160,165	432,033	0.404	174,541
1985	3,792	758	3,034	947,500	1,281,107	2,228,607	445,721	0.361	160,905
1986	3,898	507	3,391	633,750	1,431,850	2,065,600	413,120	0.322	133,025
1987	4,007	521	3,486	651,250	1,471,964	2,123,214	424,643	0.288	122,297
1988	4,120	536	3,584	670,000	1,513,344	2,183,344	436,669	0.257	112,224
1989	4,235	551	3,684	688,750	1,555,569	2,244,319	448,864	0.229	102,790
1990	4,353	566	3,787	707,500	1,599,061	2,306,561	461,312	0.205	94,569
1991								0.183	84,420
1992								0.163	75,194
1993								0.146	67,352
1994								0.130	59,971
1995								0.116	53,512
1996								0.104	47,976
1997								0.093	42,902
1998								0.083	38,289
1999								0.074	34,137
2000	4,353	566	3,787	707,500	1,599,061	2,306,561	461,312	0.066	30,447
								TOTAL	₱2,559,045

*Discounted at 12 per cent.

Reduction in Fire Damage

Based on the records of the Lipa City fire department, the average annual loss due to fire in the city, was estimated to be P440,000. It is reasonable to expect that in time, as urbanization of the study area develops, further increases in fire incidence will be experienced. However, the amount of damages per year will decrease considerably due to a combination of factors, including the presence of water supply. Other factors are fire prevention consciousness; adherence of households, commercial, industrial and institutional establishments to fire prevention regulations; and intelligent urban planning within the study area.

In the computation of the reduction in fire damage benefit, the following factors were considered: proposed fire hydrant schedule, average annual fire loss in the study area and assumed reduction in fire loss due to the project.

Since installation of fire hydrants will be undertaken on a staggered basis over the projection period, the extent of fire protection was assumed to be directly related to the portion of the study area with fire hydrants.

Every year from 1978 to 1982, fire hydrants will be installed in 32.5 ha of high-value and residential areas in Lipa City, or a total of 130 ha during the four-year period. From 1982 to 1986, another 32.5 ha every year in both high-value and residential areas will be extended fire protection. From 1986 to 1990, additional 40 ha every year will be covered. Hence, by the end of Stage I Construction, a total of 420 ha of the study area will be provided with fire hydrants.

Stage II Construction which will extend from 1990 to 2000 will involve the provision of more hydrants as well as the reinforcement of a number of existing ones. Over this 10-year period, 480 additional hectares will be extended fire protection. However, this area is not considered in the computation of the benefit because only the construction costs that would be incurred up to 1990 were included in the cost analysis. Hence, the level of fire-protected area in 1990 (420 ha) was maintained up to 2000 for purposes of this study.

According to the records of Lipa City Fire Department, the average annual fire damage from 1972 to 1975 in the city is P440,000. Since P440,000 represents the annual fire damage to the entire study area and not to the portion provided with fire hydrants, an adjustment was necessary using the following formula:

$$\frac{\text{No. of hectares with installed fire hydrants}}{\text{No. of hectares in study area}} \times ₱440{,}000$$

This was done for each year from 1979 to 1990. Thereafter up to 2000, the 1990 level of annual fire damage was maintained inasmuch as project costs considered were up to 1990 only.

After determining the annual fire loss in the portion of the study area with fire hydrants, it was then assumed that this loss would be reduced by 75 per cent because of the proposed project. Obviously, the increased fire-fighting capabilities in the study area in the form of new fire hydrant and rehabilitated old fire hydrants with adequate water pressure and in sufficient quantity will go a long way in controlling fires. The existing 84 fire hydrants in the study area were not taken into consideration because their effectivity is practically nil.

Lastly, the reduction in fire damage was discounted at 12 per cent to its present worth. Appendix Table XI-C-4 shows the fire protection benefit in LCWD. In the first approach where 1976 prices were used and then discounted, reduction in fire damage amounts to ₱225,000. In the second approach where inflation was considered, the same benefit amounts to ₱550,000.

APPENDIX TABLE XI-C-4

REDUCTION IN FIRE DAMAGE

Year	Annual Fire Damage	75% Reduction Due to Project (1976 Prices)	75% Reduction (Escalated)	Discount Factor*	PV of Benefit (1976 Prices)	PV of Escalated Benefit
1979	₱ 7,277	₱ 5,458	₱ 7,265	0.712	₱ 3,886	₱ 5,172
1980	14,554	10,916	15,981	0.636	6,943	10,164
1981	21,832	16,374	25,887	0.567	9,284	14,678
1982	29,109	21,832	37,289	0.507	11,069	18,906
1983	36,387	27,290	50,323	0.452	12,335	22,746
1984	43,664	32,748	65,234	0.404	13,230	26,354
1985	50,941	38,206	82,181	0.361	13,792	29,667
1986	58,219	43,664	99,554	0.322	14,060	32,056
1987	67,176	50,382	121,773	0.288	14,510	35,071
1988	76,132	57,099	146,288	0.257	14,674	37,596
1989	85,089	63,817	173,327	0.229	14,614	39,692
1990	94,046	70,534	203,067	0.205	14,459	41,629
1991				0.183	12,908	37,161
1992				0.163	11,497	33,100
1993				0.146	10,298	29,648
1994				0.130	9,169	26,399
1995				0.116	8,182	23,556
1996				0.104	7,336	21,119
1997				0.093	6,560	18,885
1998				0.083	5,854	16,855
1999				0.074	5,220	15,027
2000	94,046	70,534	203,067	0.066	4,655	13,402
				TOTAL	₱224,535	₱548,883

*Discounted at 12 per cent.

Incremental Revenue

Since water is essential to human life, all members of the served population in the study area presumably would be willing to obtain it in sufficient quantities at some given price. With the present water supply system, the concessionaires of LCWD are paying an average of P0.27 per cubic meter, with a present aggregate consumption of 259,000 cubic meters per annum. With the proposed improvements of the system's facilities, the volume of water consumption is expected to increase considerably to serve the needs of the growing population. This will bring about additional revenue to the water district.

In the first approach adopted for the economic analysis, the following steps were taken in the computation of this benefit:

1. The projected water consumption of the served population of LCWD from 1977 to 2000 was obtained from Table VI-8, Chapter VI. Since these figures are in cubic meters per day, they were first converted to cubic meters per year by multiplying them by 365 days.

2. The present consumption of 259,000 cubic meters of water per annum was deducted from the projected annual water consumption to obtain the incremental volume of water consumption that is directly attributable to the proposed project. The incremental volume was further broken down by type of consumer category: domestic and commercial/industrial.

3. The incremental volume for each consumer category per year was then multiplied by the proposed water charges, as determined in Chapter X, Financial Studies. The water charges used here, however, do not reflect the effect of inflation. Lastly, the benefit was discounted to obtain its present worth.

The second approach involves the concept of "consumer's surplus", as well as, incremental total revenue rather than incremental volume as used in the first approach.

"Consumer surplus" in the case of a commodity like water refers to the excess of what the consumers are willing to pay for water consumed over what the water district is charging them. It is believed that the true value of water is actually higher than the water district rates. This true value (or economic value), therefore, should be considered in the determination of this benefit. It is estimated that the economic value of water is 50 per cent higher than the proposed water rates in the case of domestic water and 20 per cent higher in the case of commercial and industrial water.

The steps followed in the second approach are similar to those for the first approach except for the fact that the revenues of the present system were deducted from the revenues of the aggregate system to arrive at the net benefit. It is believed that the proposed project will bring about not only an increase in the volume of production (and consumption) but also an increase in the water rate. Consequently, this will result in higher revenue for the water district. The proposed rates used in this benefit (plus 50 per cent or 20 per cent consumer's surplus as the case may be) are those contained in Chapter X and reflect the effect of inflation up to 1990.

APPENDIX TABLE XI-C-5

INCREMENTAL REVENUE BENEFIT
FIRST APPROACH

Year	Projected Delivered Water	Incremental Volume of Water*	Domestic	Com./Ind.	Price per cum Domestic	Price per cum Com./Ind.	Total Revenues ₱ x 1,000	Discount Factor**	PV of Total Revenues ₱ x 1,000
1976	259	–	–	–	–	–	–	1.000	–
1977	330	71	58	13	1.00	2.00	84.00	0.893	75.0
1978	420	161	133	28	1.00	2.00	189.00	0.797	150.6
1979	535	276	231	45	1.00	2.00	321.00	0.712	228.6
1980	681	422	358	64	1.30	2.60	631.80	0.636	401.8
1981	867	608	522	86	1.30	2.60	902.20	0.567	511.5
1982	1,105	846	735	111	1.30	2.60	1,244.10	0.507	630.8
1983	1,407	1,148	1,011	137	1.33	2.66	1,709.05	0.452	772.5
1984	1,791	1,532	1,367	165	1.33	2.66	2,257.01	0.404	911.8
1985	2,281	2,022	1,828	194	1.33	2.66	2,947.28	0.361	1,064.0
1986	2,511	2,252	2,030	222	1.23	2.46	3,043.02	0.322	979.9
1987	2,764	2,505	2,251	254	1.23	2.46	3,393.57	0.288	977.3
1988	3,043	2,784	2,495	289	1.23	2.46	3,779.79	0.257	971.4
1989	3,349	3,090	2,762	328	1.09	2.18	3,725.62	0.229	853.2
1990	3,687	3,428	3,054	374	1.09	2.18	4,144.18	0.205	849.6
1991								0.183	758.4
1992								0.163	675.5
1993								0.146	605.1
1994								0.130	538.7
1995								0.116	480.7
1996								0.104	431.0
1997								0.093	385.4
1998								0.083	344.0
1999								0.074	306.7
2000	3,687	3,428	3,054	374	1.09	2.18	4,144.18	0.066	273.5
						TOTAL	69,813.42		14,177.0

*The 1976 volume of delivered water was deducted from the projected delivered water throughout the projection period to obtain the incremental volume of water.

**Discounted at 12 per cent.

APPENDIX TABLE XI-C-6

INCREMENTAL REVENUE BENEFIT
SECOND APPROACH

Year	Projected Delivered Water Total (cum x 1,000)	Domestic (cum x 1,000)	Com./Ind. (cum x 1,000)	Price per cum Domestic	Price per cum Com./Ind.	Total Economic Revenues ₱ x 1,000	Net Revenues* ₱ x 1,000	Discount Factor**	PV of Net Revenues ₱ x 1,000
1976	259	208	51			–	–	1.000	–
1977	330	268	60	₱1.65	₱2.75	607.2	537.3	0.893	479.8
1978	420	346	71	1.65	2.75	766.2	696.3	0.797	555.0
1979	535	447	83	1.65	2.75	965.8	889.9	0.712	637.9
1980	681	577	98	2.85	4.75	2,110.0	2,040.0	0.636	1,297.5
1981	867	744	115	2.85	4.75	2,666.7	2,597.0	0.567	1,472.4
1982	1,105	960	136	2.85	4.75	3,382.0	3,312.1	0.507	1,679.2
1983	1,407	1,239	160	3.68	4.13	5,540.3	5,470.4	0.452	2,472.6
1984	1,791	1,598	188	3.68	6.13	7,033.1	6,963.2	0.404	2,813.1
1985	2,281	2,062	219	3.68	6.13	8,930.6	8,860.7	0.361	3,199.0
1986	2,511	2,263	247	4.20	7.00	9,521.9	9,452.0	0.322	3,043.5
1987	2,764	2,484	279	4.20	7.00	12,385.8	12,315.9	0.288	3,547.0
1988	3,043	2,727	315	4.20	7.00	13,658.4	13,588.5	0.257	3,492.2
1989	3,349	2,993	356	4.43	7.38	15,886.3	15,816.4	0.229	3,621.9
1990	3,687	3,285	402	4.43	7.38	17,519.3	17,449.4	0.205	3,577.1
1991								0.183	3,193.2
1992								0.163	2,844.2
1993								0.146	2,547.6
1994								0.130	2,268.4
1995								0.116	2,024.1
1996								0.104	1,814.7
1997								0.093	1,622.8
1998								0.083	1,448.3
1999								0.074	1,291.3
2000	3,687	3,285	402	4.43	7.38	17,519.3	17,449.4	0.066	1,151.7
						TOTAL	274,483.1		52,094.5

*The present economic revenue amounting to ₱69,900 (259,000 cum x ₱.27) was deducted from the total economic revenues every year throughout the projection period to obtain the net revenue (benefit).

**Discounted at 12 per cent.

APPENDIX TABLE XI-E-1

CONVERSION OF FINANCIAL COST TO ECONOMIC COST
LIPA CITY WATER DISTRICT
₱ x 1,000

	Financial Project Cost	Foreign Component	Domestic Component	Unskilled Labor	Balance of Domestic	Taxes 5%	Others 95%	Shadow Pricing			Economic Project Cost	Economic Construction Cost
								Foreign Component x 1.2	Unskilled Labor x .5	Others x 1.00		
Source Development												
a) Structures	5,200.0	1,462.0	3,738.0	1,597.0	2,141.0	107.1	2,033.9	1,754.4	798.5	2,033.9	4,586.8	3,580.3
b) Equipment	2,649.0	2,378.8	270.2	115.4	154.8	7.7	147.1	2,854.6	57.7	147.1	3,059.4	2,348.1
Transmission Distribution												
Mains & Valves	8,450.0	4,322.5	4,127.5	697.0	3,430.5	171.5	3,259.0	5,187.0	348.5	3,259.0	8,794.5	6,749.7
Storage Tanks & Appurt.												
a) Structure	1,058.0	175.7	882.3	133.0	749.3	37.5	711.8	210.8	66.5	711.8	989.1	759.1
b) Equipment	283.0	218.6	64.4	9.7	54.7	2.7	52.0	262.3	4.8	52.0	319.1	244.9
Internal Network	5,585.0	2,579.2	3,005.8	575.1	2,430.7	121.5	2,309.2	3,095.0	287.6	2,309.2	5,691.8	4,368.4
Service Connections	7,164	4,034.9	3,129.1	843.0	2,286.1	114.3	2,171.8	4,841.9	421.5	2,171.8	7,435.2	5,706.4
a) Pipes	4,255.9	2,410.8	1,845.1	497.1	1,348.0	67.4	1,280.6	2,893.0	248.5	1,280.6	4,422.1	3,393.9
b) Meters	1,844.5	1,563.1	281.4	75.8	205.6	10.3	195.3	1,875.7	37.9	195.3	2,108.9	1,618.6
c) Other	1,063.6	61.0	1,002.6	270.1	732.5	36.6	695.9	73.2	135.1	695.9	904.2	694.0
Water District Building	1,407.0	535.0	872.0	186.3	685.7	34.3	651.4	642.0	93.2	651.4	1,386.6	1,064.2
Early Action Works												
a) Service Connections	190.4	92.3	98.1									
Pipes	160.2	92.3	67.9	6.7	61.2	3.1	58.1	110.8	3.4	58.1	172.3	142.7
Meters	-	-	-	-				-			-	-
Other	30.2	-	30.2	6.0	24.2	1.2	23.0	-	3.0	23.0	26.0	21.5
b) Vehicles	121.0	72.5	48.5	-	48.5	2.4	46.1	87.0		46.1	133.1	110.2
c) Other Equipment	555.0	458.8	96.2	5.2	91.0	4.6	86.4	550.6	2.6	86.4	639.6	529.7
d) Miscellaneous System												
Improvements	145.0	102.7	42.3	1.1	41.2	2.1	39.1	123.2	0.6	39.1	162.9	134.9
SUB-TOTAL	32,807.4	16,433.0	16,374.4	4,175.5	12,198.9	609.9	11,589.0	19,719.6	2,087.9	11,589.0	33,396.4	31,406.6
Land	245.0	-	245.0	-	245.0	12.3	232.7	-	-	232.7	232.7	192.7
TOTAL	33,052.4	16,433.0	16,619.4	4,175.5	12,443.9	622.2	11,821.7	19,719.6	2,087.9	11,821.7	33,629.1	31,599.3

APPENDIX TABLE XI-B-2

REPLACEMENT COST (1976 PRICES)

₱ x 1,000

YEAR	VEHICLES	METERS	TOTAL
1976			
1977			
1978			
1979			
1980			
1981			
1982			
1983			
1984	67.1		67.1
1985	66.0		66.0
1986			
1987			
1988			
1999			
1990			
1991			
1992	67.1	23.2	90.3
1993	66.0	82.2	148.2
1994		135.0	135.0
1995		162.4	162.4
1996		179.3	179.3
1997		177.1	177.1
1998		177.1	177.1
1999		200.3	200.3
2000	67.1	202.5	269.6
	333.3	1,339.1	1,672.4

APPENDIX TABLE XI-E-3

SALVAGE VALUE (1976 PRICES)

₱ x 1,000

Year	50 Yrs		25 Yrs		15 Yrs		7 Yrs	
1976	149.6	26/50	264.8	1/25				
1977	516.9	27/50	325.2	2/25				
1978	1,620.6	28/50	382.2	3/25				
1979	2,863.0	29/50	702.4	4/25				
1980	2,084.7	30/50	352.8	5/25				
1981	1,469.4	31/50	-					
1982	1,440.4	32/50	43.7	7/25				
1983	1,647.9	33/50	281.0	8/25				
1984	2,073.4	34/50	572.8	9/25				
1985	1,743.1	35/50	256.8	10/25				
1986	1,271.0	36/50						
1987	1,133.8	37/50			139.2	1/15		
1988	1,133.8	38/50			139.2	2/15		
1989	1,133.8	39/50			139.2	3/15		
1990	565.5	40/50			69.6	4/15		
1991								
1992					17.8	6/15		
1993					63.1	7/15		
1994					103.6	8/15		
1995					103.6	9/15		
1996					124.6	10/15		
1997					137.6	11/15		
1998					136.0	12/15		
1999					136.0	13/15		
2000							55.5	6/7
TOTAL	20,846.9		3,121.7		1,463.3		55.5	

APPENDIX TABLE XI-B-4

SALVAGE VALUE IN 2001
1976–2001
P x 1,000 in 1976 Prices

	50 Years			25 Years			15 Years			7 Years			Infinite		
	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value
1976	149.6	52	77.8	264.8	4	10.6									
1977	516.9	54	279.1	325.2	8	26.0							137.3	100	137.3
1978	1,620.6	56	907.5	382.2	12	45.9									
1979	2,863.0	58	1,660.5	702.4	16	112.4									
1980	2,084.7	60	1,250.8	352.8	20	70.6									
1981	1,469.4	62	911.0	–	–	–									
1982	1,440.4	64	921.9	43.7	28	12.2									
1983	1,647.9	66	1,087.6	281.0	32	89.9							55.0	100	55.0
1984	2,073.4	68	1,409.9	512.8	36	184.6									
1985	1,743.1	70	1,220.2	356.8	40	102.7									
1986	1,271.0	72	915.1	–											
1987	1,133.8	74	839.0	–			139.2	6.6	9.2						
1988	1,133.8	76	661.7	–			139.2	13.3	18.5						
1989	1,133.8	78	884.4	–			139.2	20.0	27.8						
1990	565.5	80	452.4	–			69.6	26.7	18.6						
1991															
1992							17.8	40	7.1						
1993							63.1	47	29.7						
1994							103.6	53	54.9						
1995							103.6	60	62.2						
1996							124.6	67	83.5						
1997							137.6	73	100.4						
1998							136.0	80	108.8						
1999							136.0	87	118.3						
2000							153.8	93	143.0	55.5	86	47.7			
Total	20,846.9		13,478.9	3,121.7		654.9	1,463.2		782.0	55.5		47.7	192.3		192.3

APPENDIX TABLE XI-E-5

SUMMARY OF TOTAL ECONOMIC COSTS
FIRST APPROACH
₱ x 1,000

Year	Project Cost	Replacement Cost	Salvage Value	O and M Cost	Total Cost 1976 Prices	Discount Factor*	PV of Project Cost	PV of Replacement Cost	PV of Salvage Value	PV of O and M Cost	PV of Total Cost
1976	735.2			10	745.2	1.000	735.2			10.0	745.2
1977	1,147.7			20	1,167.7	0.893	1,024.9			17.9	1,042.8
1978	2,691.6			40	2,731.6	0.797	2,145.2			31.9	2,177.1
1979	4,780.6			60	4,840.6	0.712	3,403.8			42.7	3,446.5
1980	3,311.0			177	3,488.0	0.636	2,105.8			112.6	2,218.4
1981	2,077.0			214	2,921.0	0.567	1,177.7			121.3	1,299.0
1982	2,179.6			252	2,431.6	0.507	1,105.1			127.8	1,232.9
1983	2,690.5			284	2,974.5	0.452	1,216.1			128.4	1,344.5
1984	3,547.0	67.1		316	3,930.1	0.404	1,433.0	27.1		127.7	1,587.8
1985	2,806.3	66.0		429	3,301.3	0.361	1,013.1	23.8		154.9	1,191.8
1986	1,858.7			487	2,345.7	0.322	598.5			156.8	755.3
1987	1,658.7			516	2,174.7	0.288	477.7			148.6	626.4
1988	1,658.7			546	2,024.7	0.257	426.3			140.3	566.6
1989	1,658.7			575	2,233.7	0.229	379.8			131.7	511.6
1990	827.7			646	1,473.7	0.205	169.7	18.6		132.4	302.1
1991				646	646.0	0.183				118.2	118.2
1992		90.3			736.3	0.163		14.7		105.3	120.0
1993		148.2			794.2	0.146		21.6		94.3	115.9
1994		135.0			781.0	0.130		17.6		84.0	101.6
1995		162.4			808.4	0.116		18.8		74.9	93.8
1996		179.3			825.3	0.104		18.6		67.2	85.8
1997		177.1			823.1	0.093		16.5		60.1	76.5
1998		177.1			823.1	0.083		14.7		53.6	68.3
1999		200.3			846.3	0.074		14.8		47.8	62.6
2000		269.6		646	915.6	0.066		17.8		42.6	60.4
2001			15,155.8			0.059			894.2		
	33,629.1	1,672.4		11,032	46,333.4		17,411.9	206.0		2,333.0	19,950.9
					- 15,155.8						- 894.2
	TOTAL				31,177.6						19,056.7

*Discounted at 12 per cent.

APPENDIX TABLE XI-E-6

SUMMARY OF TOTAL ECONOMIC COSTS
SECOND APPROACH
₱ x 1,000

Year	Escalated Project Cost*	Escalated Replacement Cost*	Escalated Salvage Value*	Escalated O and M Cost**	Escalated Total Cost	Discount Factor***	PV of Project Cost	PV of Replacement Cost	PV of Salvage Value	PV of O and M Cost	PV of Total Cost
1976	735.2			10.0	745.2	1.000	735.2			10.0	745.2
1977	1,262.5			21.6	1,284.1	0.893	1,127.4			19.3	1,146.7
1978	3,256.8			46.6	3,303.4	0.797	2,595.7			37.1	2,632.8
1979	6,363.0			75.6	6,438.6	0.712	4,530.5			53.8	4,584.3
1980	4,847.3			240.7	5,088.0	0.636	3,082.9			153.1	3,236.0
1981	3,283.7			314.4	3,598.1	0.567	1,861.9			178.3	2,040.2
1982	3,722.8			400.0	4,122.8	0.507	1,887.5			202.8	2,090.3
1983	4,961.3			486.8	5,448.1	0.452	2,242.5			220.0	2,462.5
1984	7,065.6	133.7		584.9	7,784.2	0.404	2,854.5	54.0		236.3	3,144.8
1985	6,036.4	142.0		857.6	7,036.0	0.361	2,179.1	51.3		309.6	2,540.0
1986	4,237.8			1,051.4	5,289.2	0.322	1,364.6			338.6	1,703.2
1987	4,009.1			1,203.3	5,212.4	0.288	1,154.6			346.6	1,501.2
1988	4,249.6			1,374.8	5,624.4	0.257	1,092.1			353.3	1,445.5
1989	4,505.0			1,564.0	6,069.0	0.229	1,031.6			358.2	1,389.8
1990	2,382.9			1,897.3	4,280.2	0.205	488.5			388.9	877.4
1991				1,897.3	1,897.3	0.183				347.2	347.2
1992		260.0		1,897.3	2,157.3	0.163		42.4		309.2	351.6
1993		426.7		1,897.3	2,324.0	0.146		62.3		277.0	339.3
1994		388.7		1,897.3	2,286.0	0.130		50.5		246.6	297.1
1995		467.5		1,897.3	2,364.8	0.116		54.2		220.1	274.3
1996		516.2		1,897.3	2,413.5	0.104		53.7		197.3	251.0
1997		509.9		1,897.3	2,407.2	0.093		47.4		176.4	223.8
1998		509.9		1,897.3	2,407.2	0.083		42.3		157.5	199.8
1999		576.7		1,897.3	2,474.0	0.074		42.7		140.4	183.1
2000		776.2		1,897.3	2,673.5	0.066		51.2		125.2	176.4
2001			43,633.6			0.059			2,574.4		
	60,919.0	4,707.5		29,102.0	94,728.5		28,228.6	552.0		5,402.8	34,183.4
					- 43,633.6						- 2,574.4
TOTAL					51,094.9						31,609.0

*Escalated annually by 10 per cent from 1976 to 1980, by 8 per cent from 1980 to 1985 and by 6 per cent from 1985 to 1990. Thereafter up to 2000, the escalation factor was maintained at its 1990 level.

**Escalated annually by 8 per cent from 1976 to 1990. Thereafter up to 2000, the escalation factor was maintained at its 1990 level.

***Discounted at 12 per cent.

APPENDIX TABLE XI-E-7

INTERNAL ECONOMIC RATE OF RETURN
LIPA CITY WATER DISTRICT
₱ x 1,000

Year	FIRST APPROACH: Benefits (1976 Prices)	FIRST APPROACH: Costs (1976 Prices)	Present Value at 23%: Discount Factor	Present Value at 23%: Benefits (1976 Prices)	Present Value at 23%: Costs (1976 Prices)	SECOND APPROACH: Escalated Benefits	SECOND APPROACH: Escalated Costs	Present Value at 22%: Discount Factor	Present Value at 22%: Benefits	Present Value at 22%: Costs
1976		745.2	1.000		745.2		745.2	1.000		745.2
1977	84.0	1,167.7	0.813	68.3	949.3	537.3	1,284.1	0.820	440.6	1,053.0
1978	220.9	2,731.6	0.661	146.0	1,805.6	696.3	3,303.4	0.672	467.9	2,219.[illegible]
1979	2,439.3	4,840.6	0.537	1,309.9	2,599.4	903.2	6,438.6	0.551	497.7	3,547.[illegible]
1980	2,768.1	3,488.0	0.437	1,209.6	1,524.2	2,056.1	5,088.0	0.451	927.3	2,294.[illegible]
1981	3,057.2	2,291.0	0.355	1,085.3	813.3	2,622.9	3,598.1	0.370	970.5	1,331.3
1982	3,418.1	2,431.6	0.289	987.8	702.7	3,349.4	4,122.8	0.303	1,014.9	1,249.2
1983	3,902.5	2,974.5	0.235	917.1	699.0	5,520.7	5,448.1	0.248	1,369.1	1,351.1
1984	4,470.2	3,930.1	0.191	853.8	750.6	7,028.4	7,784.2	0.204	1,433.8	1,588.0
1985	5,180.8	3,301.3	0.155	803.0	511.7	8,942.9	7,036.0	0.167	1,493.5	1,175.0
1986	6,603.8	2,345.7	0.126	832.1	295.6	9,551.6	5,289.2	0.137	1,308.6	724.6
1987	6,973.7	2,174.7	0.102	711.3	221.8	12,437.7	5,212.4	0.112	1,393.0	583.8
1988	7,379.8	2,024.7	0.083	612.5	183.0	13,734.8	5,624.4	0.092	1,263.6	517.4
1989	7,345.8	2,233.7	0.068	499.5	151.9	15,989.7	6,069.0	0.075	1,199.2	455.[illegible]
1990	7,784.8	1,473.7	0.055	428.2	81.1	17,652.5	4,280.2	0.062	1,094.4	265.4
1991		646.0	0.045	350.3	29.1		1,897.3	0.051	900.3	96.6
1992		736.3	0.036	280.2	26.5		2,157.3	0.042	741.4	90.6
1993		794.2	0.030	233.5	23.8		2,324.0	0.034	600.2	79.0
1994		781.0	0.024	186.8	18.7		2,286.0	0.028	494.3	64.0
1995		808.4	0.020	155.7	16.2		2,364.8	0.023	406.0	54.[illegible]
1996		825.3	0.016	124.6	13.2		2,413.5	0.019	335.4	45.9
1997		823.1	0.013	101.2	10.7		2,407.2	0.015	264.8	36.1
1998		823.1	0.011	85.6	9.1		2,407.2	0.013	229.5	31.3
1999		864.3	0.008	62.3	6.9		2,474.0	0.010	176.5	24.7
2000	7,784.8	915.6	0.007	54.5	6.4	17,652.5	2,673.5	0.008	141.2	21.4
	139,477.0	46,333.4		12,099.1	12,195.0	277,548.5	94,728.5		19,[illegible]63.7	19,645.7
2001		- 15,155.8	0.006		- 90.9		- 43,633.6	0.007		- 305.4
		31,177.6			12,104.1		51,094.9			19,340.3
	(4.474)	(1.000)		(0.999)	(1.000)	(5.432)	(1.000)		(0.991)	(1.000)