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FEASIBILITY STUDY
TECHNICAL FINAL REPORT

VOLUME II (APPENDICES)

WATER SUPPLY

TARLAC WATER DISTRICT

JULY 1976

CAMP DRESSER & McKEE INTERNATIONAL INC.

CONSULTING ENVIRONMENTAL ENGINEERS

FOREWORD

Volume II (Appendices) of the Technical Final Report on the Tarlac 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.

Appendix VII-B provides 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

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 areas 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 customersand 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 hectarewill 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 unacoounted-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:

	Diameter (mm)				Service	
Pipe Material	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 equipment 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.

<u>Raw Water Pump Stations</u>

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.

Aeration. 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.

Mixing. 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 Chlorammonia-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/1	20-70						O	O			O			
	>70						E	E			O			
Tastes and odors	noticeable		O		O				O		O	E		
Calcium carbonate-mg/1	> 200					O	E	E	E				E	
Iron and manganese-mg/1	< 0.3		O	O				S						
	0.3-1.0				O		E	E	O					
	>1.0		E		E		E	E	O				O	
Chloride-mg/1	0-250													
	250-500													O
	500[5]													E
Phenolic compounds-mg/1	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	Cost in Pesos FEC	Cost in Pesos 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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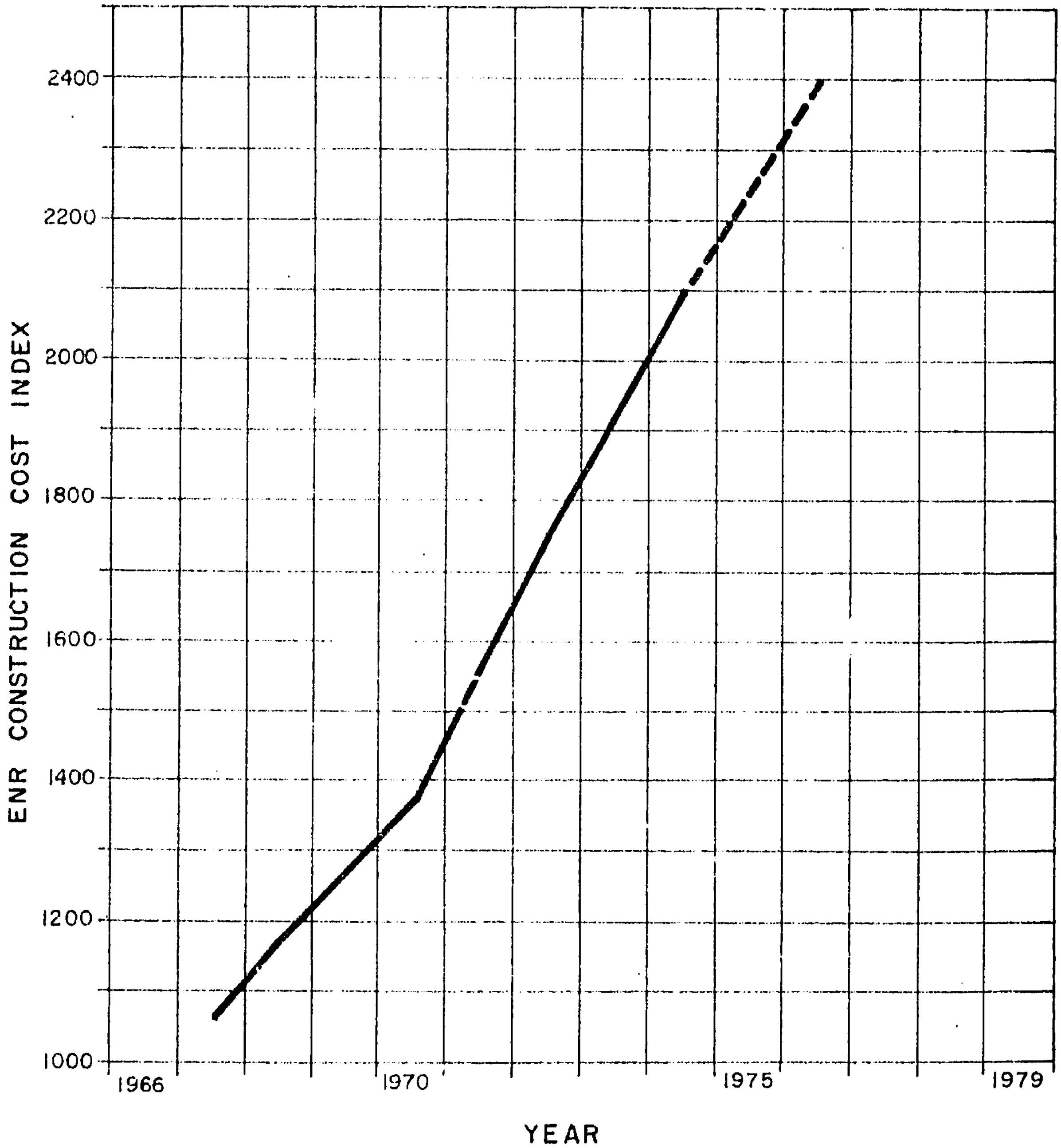
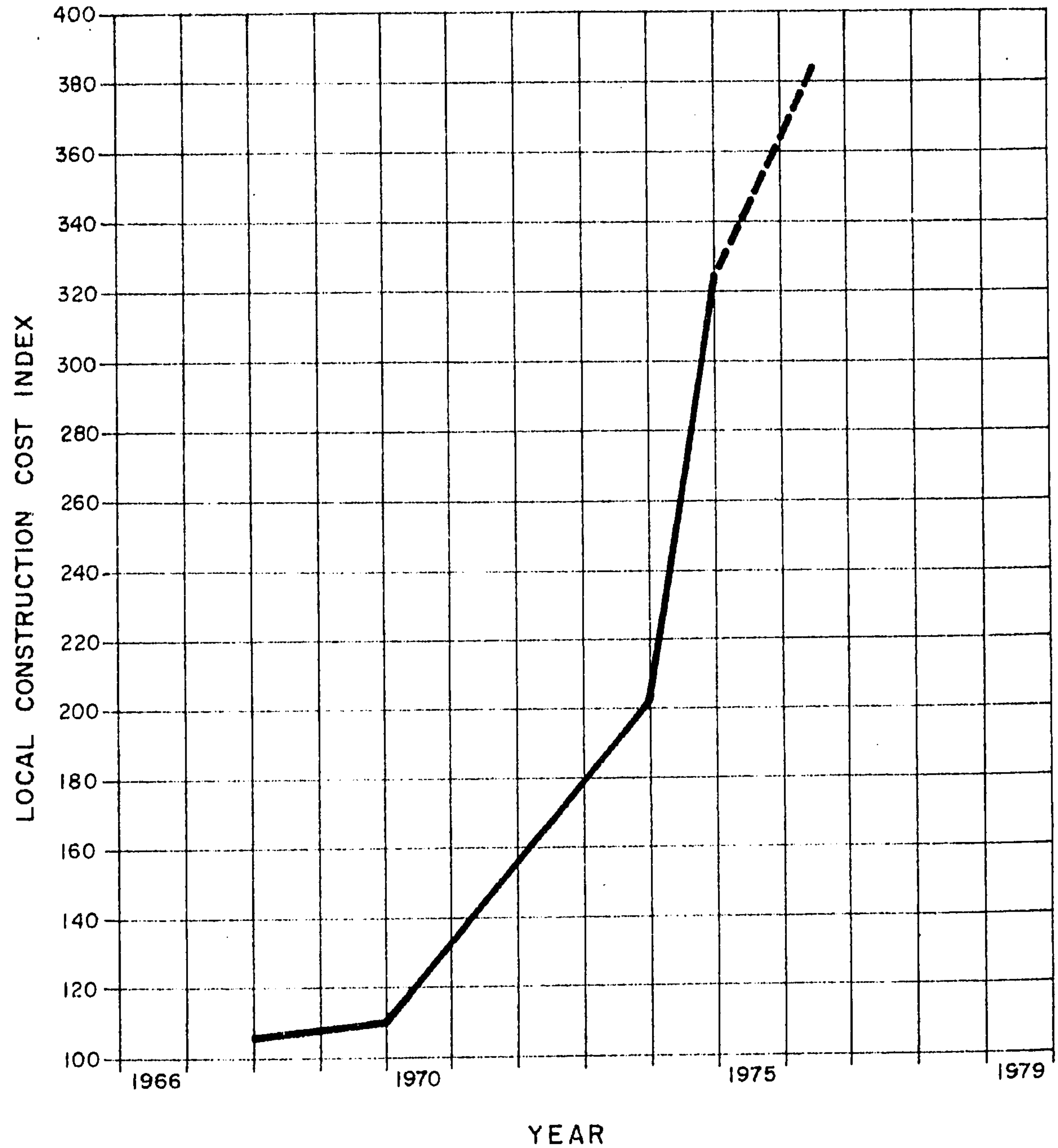
[1] Excluding interest during construction.



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	

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

[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	2.5	Tunnel "D" in meters 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

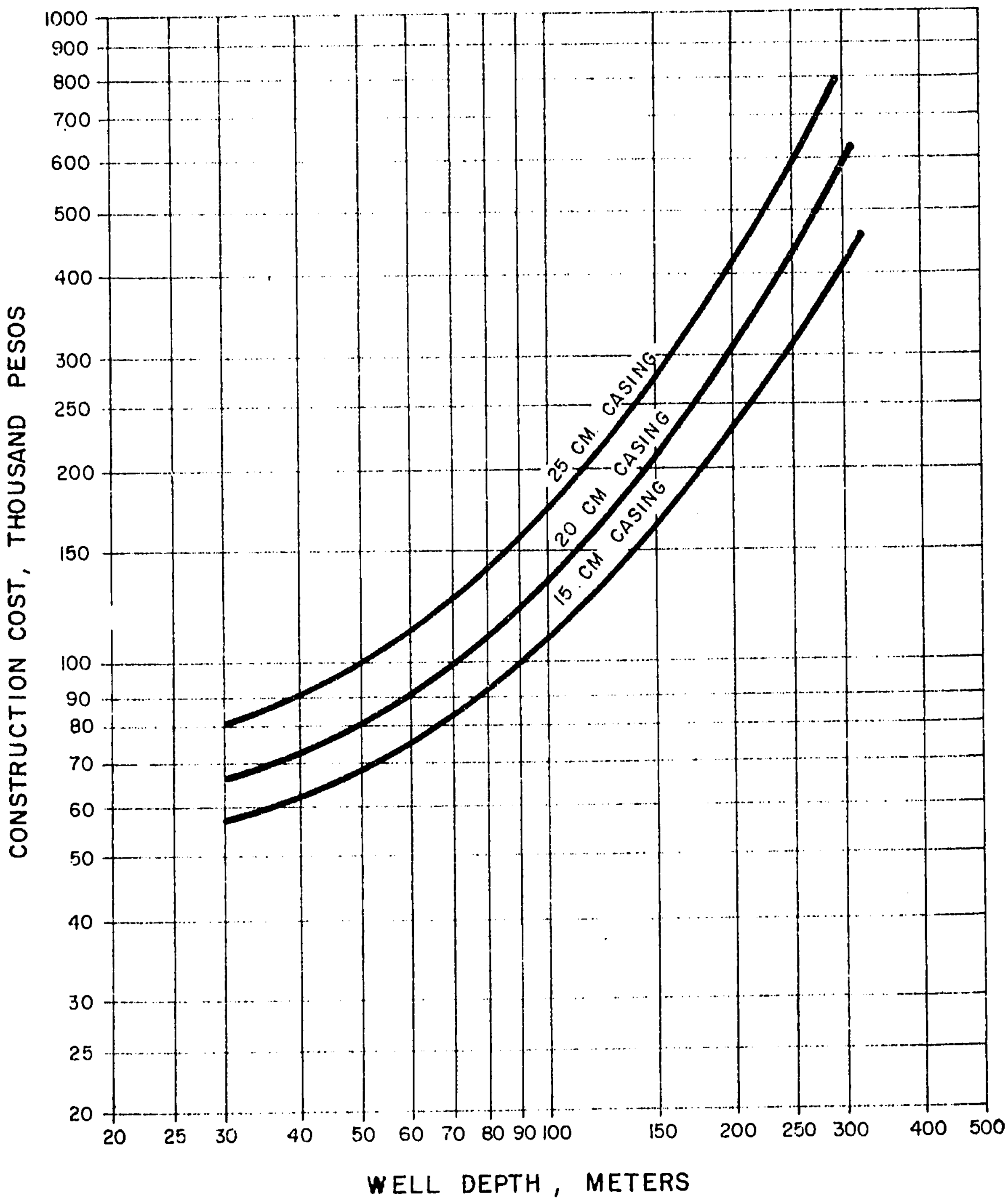
The cost curves which are shown in Appendix Figure B-5 are for a pump station adjacent to a river or lake. The cost of this type of pump station includes an approach channel, intake structure and a pump wet well. A superstructure for housing pump, motors and controls and necessary piping are also included. 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

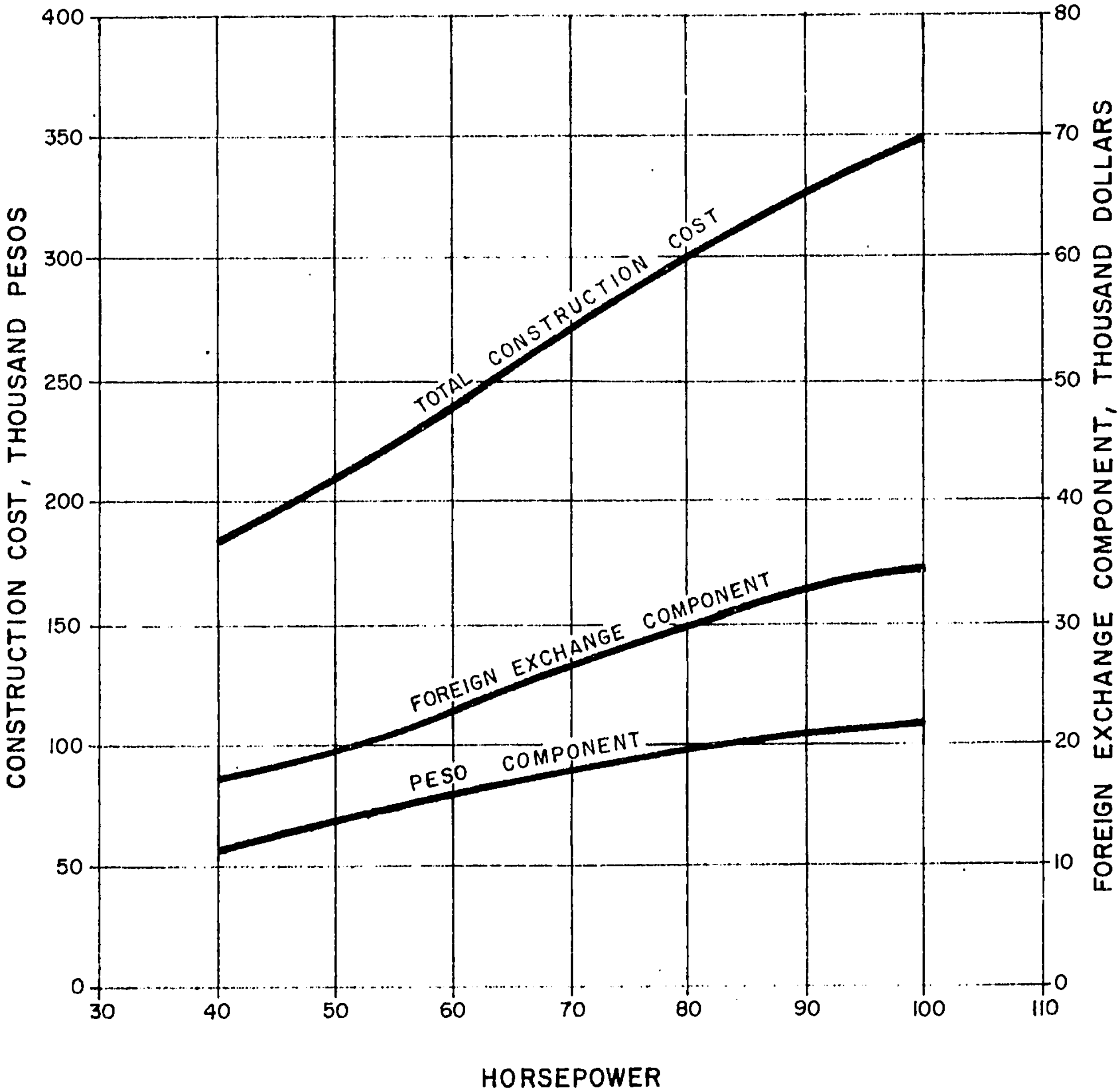
Cost studies have been made on pipe of various materials including cast iron, asbestos cement, steel, ductile iron and prestressed concrete. The unit costs of pipelines are based on the assumption that the least cost pipe, whether locally manufactured or imported, will be utilized. The estimated unit 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.

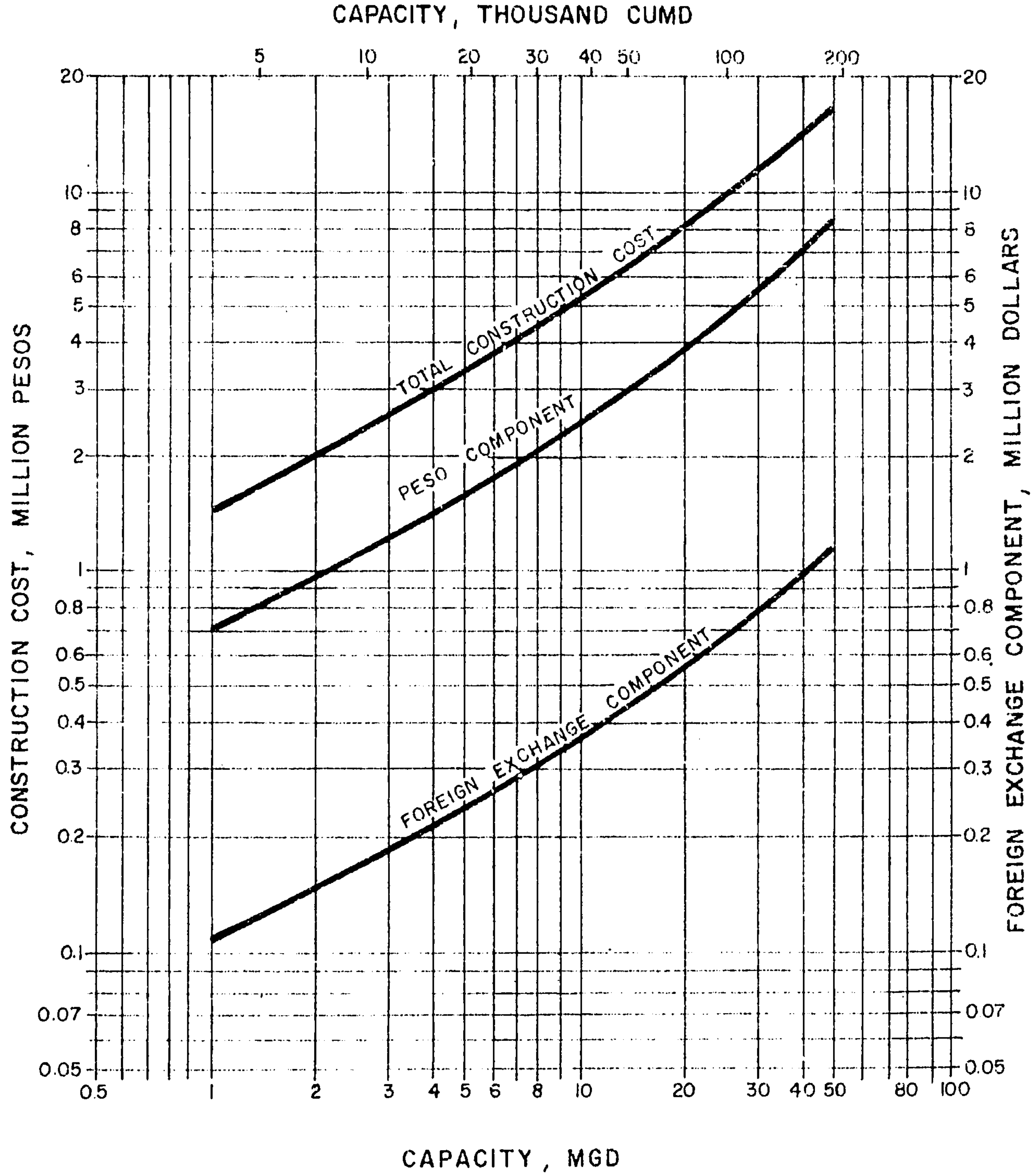
APPENDIX FIGURE B-3
DEEP WELL CONSTRUCTION COSTS
(JULY 1976 PRICES)



NOTE :

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

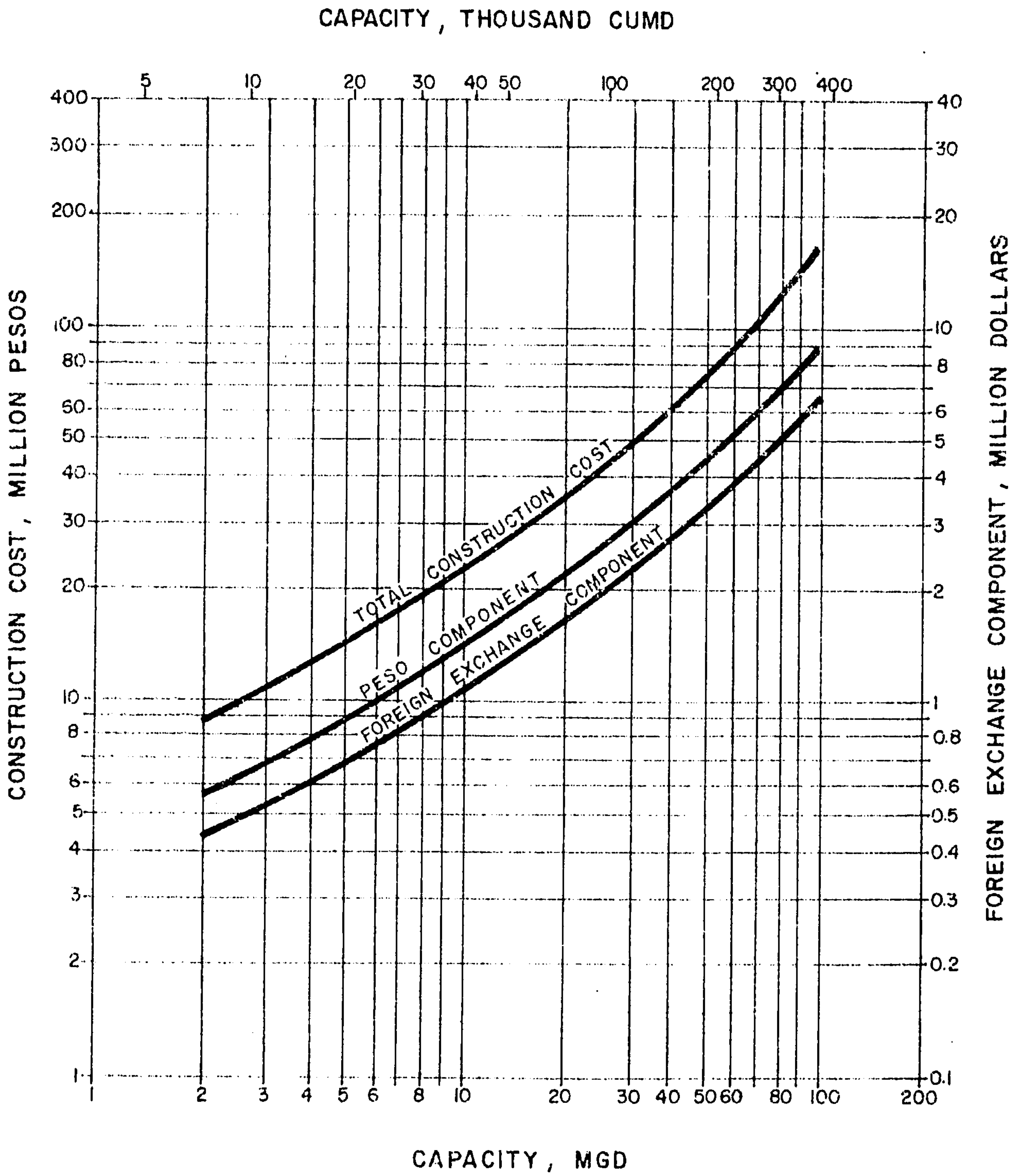
APPENDIX FIGURE B-4
DEEP WELL PUMP STATION COSTS
(JULY 1976 PRICES)



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, ISO 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

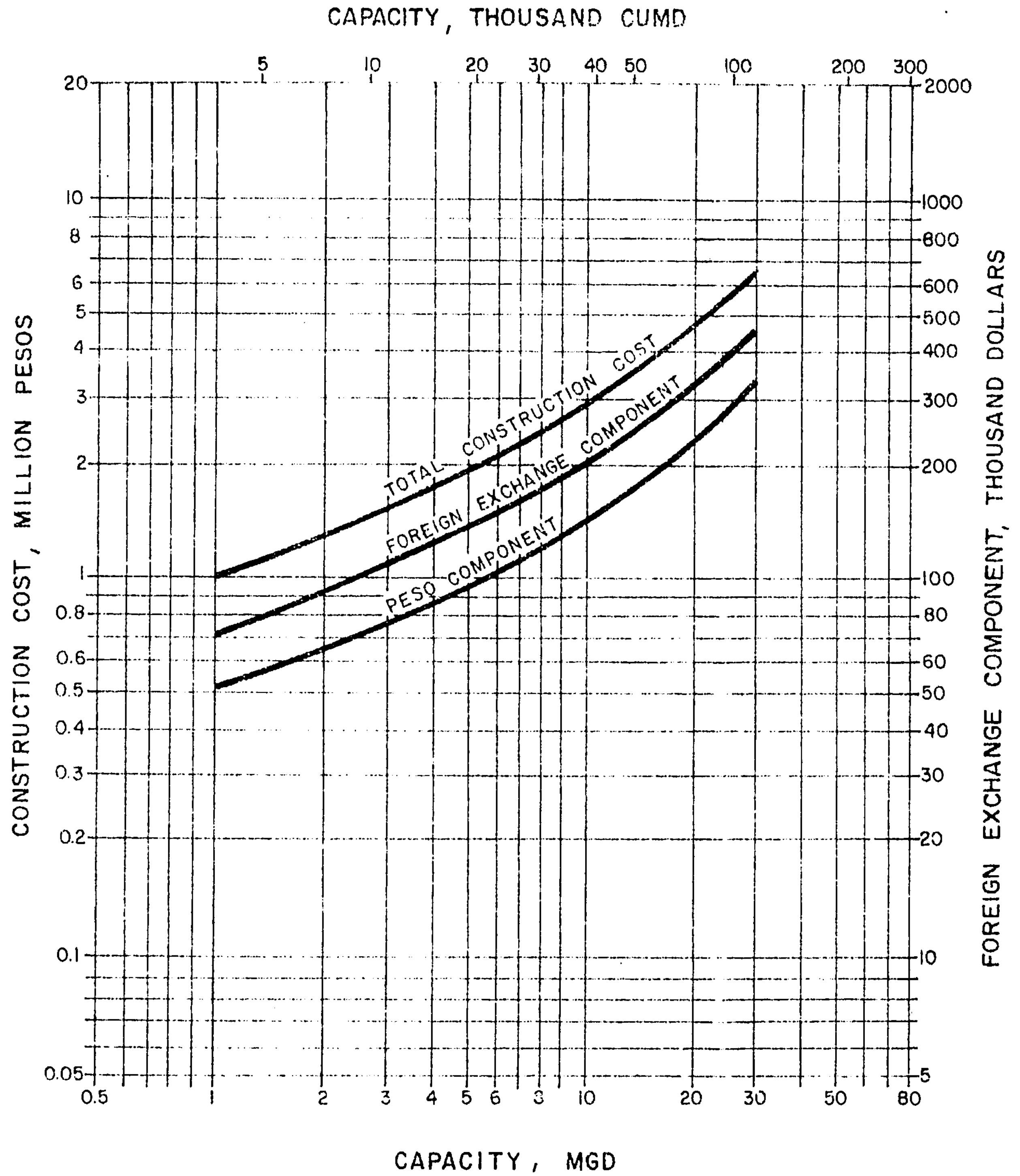
Construction cost estimates of ground storage reservoirs are presented in Appendix Figure B-8 including steel, reinforced concrete and prestressed concrete tanks. The costs for steel and reinforced concrete tanks are based on updated costs of actual construction in the past in the Philippines and in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

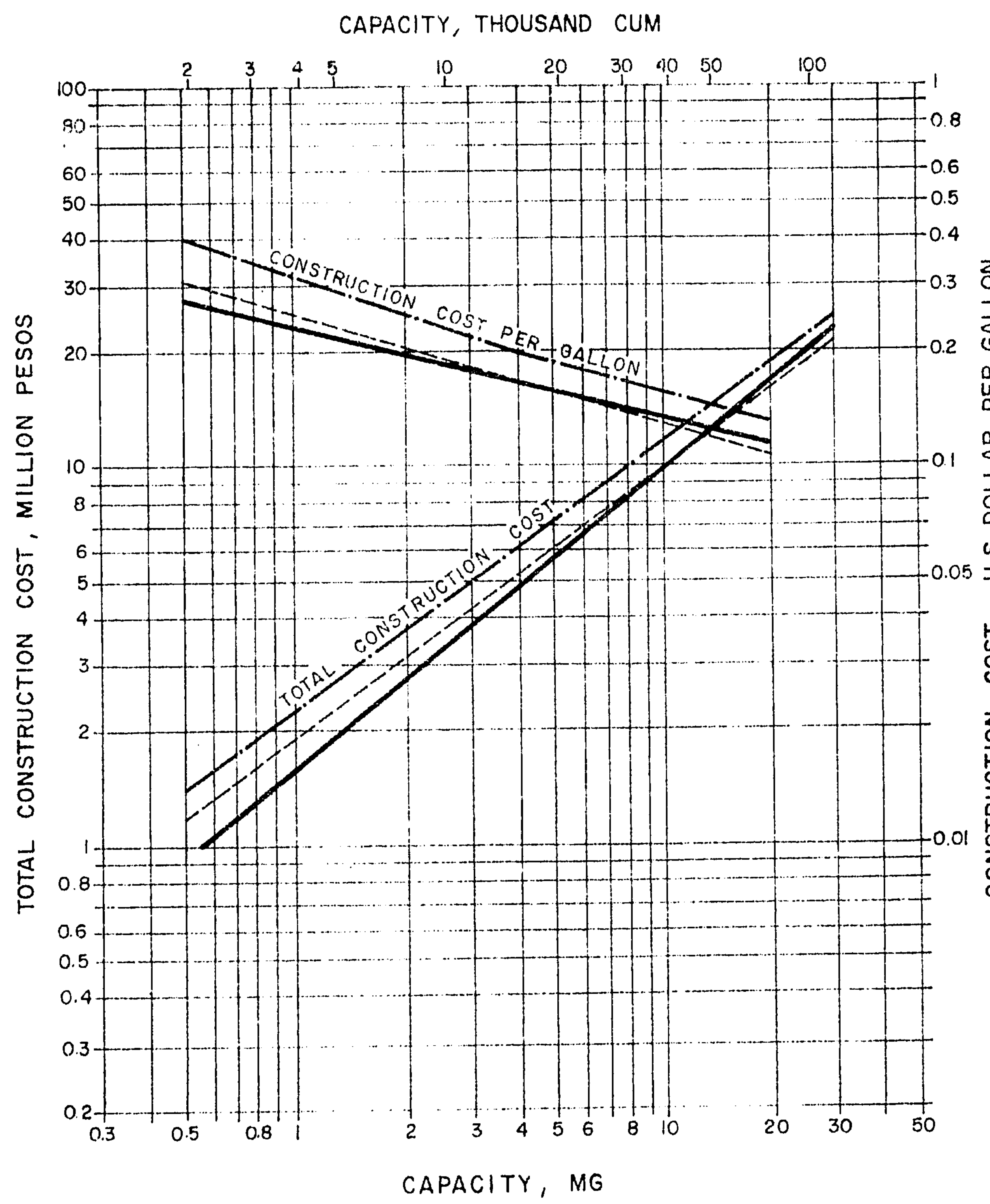
Tank costs include ordinary piping, valving, and tank accessories such as vent, access manhole, ladder rungs, etc. The costs do not include special valves and controls, land taking and access road.

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,[illegible]00	11,200	39,800	51,000
1,[illegible]0	12,600	47,400	60,000
1,[illegible]0	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

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[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/ (P) 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 occur 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.

<u>Cast Iron and Ductile Iron Pipe</u>

<u>General</u>. 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 leadcaulked 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 polyvinylidine 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 Stryrene). 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 . 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-standardization 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:

	Trench Width (cm)	
Pipe Diameter (mm)	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 age, 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 "packor" 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, totalizing 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.

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OUTLINE SPECIFICATIONS

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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 IWUA 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 $8^{o} - 10'$. 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.

<u>Diesel Engine</u>

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 a [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

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
TLC-1	Barrio San Jose, Tarlac	100	25	23	-1.5	3.1	0.6	0.4	1961
TLC-2	Barrio Dalayap, Tarlac	100	21	18	-0.9	3.1	0.6	0.2	1962
TLC-3	Bo. Mababanaba, Tarlac	112	46	14	-5.8	10.7	0.3	0.1	1973
TLC-4	Bo. Culipat, Tarlac	112	61	56	-3.1	3.4	0.6	2.1	1973
TLC-5	Mabini Street, Tarlac	200-150	169	162	-3.7	9.8	4.7	0.8	1970
TLC-6	Bo. Pao, Tarlac	100	22	10	-5.5	7.0	0.6	0.4	1962
TLC-7	San Vicente, Tarlac	200	50	41	-0.3	15.6	13.9	1.3	1958
TLC-8	Bo. San Nicolas, Tarlac	200-150	237	20-186	-2.4	15.2	6.3	0.4	1954
TLC-9	Bo. Balingcanaway, Tarlac, Tarlac	62	24		-3.1	4.6	1.3	0.8	1957
TLC-10	Romulo Blvd. Tarlac, Tarlac	200	170	106	-2.4	6.4	4.7	1.3	1950
TLC-11	NIA P102 (T-14), Tarlac				-1.6	29.7	70.8	2.5	
TLC-12	Camp O'Donell San Miguel	250	139	133	-8.5	29.0	7.3	0.4	1962
TLC-13	Taguipore, Sta. Ignacia	100	52	37	-2.1	10.9	1.5	0.2	1958
TLC-14	Camiling, Tarlac	100	90	63	-2.9	15.3	5.3	0.4	1961
TLC-15	Public Market Cpd. Concepcion	100	152	152	-1.2	11.3	10.3	1.1	1960
TLC-16	Public Market Cpd. La Paz	100	152	152	-0.3	19.8	9.8	0.4	1960
TLC-17	Tambugan, Camiling	100	80	778	-2.5	5.2	3.7	1.5	1958
TLC-18	Legaspi, San Miguel	100	152	134	-4.2	5.7	3.8	2.5	1958
TLC-19	Atencio, Moncada	100	193	137	-1.2	2.1	3.8	4.2	1958
TLC-20	Butao Tibag, Tarlac	100	168	46	-10.0	14.7	4.0	0.4	1957
TLC-21	Calumpit, Capas	100	247	54	-8.0	12.7	3.8	0.8	1957
TLC-22	Salapungan, Gerona (NIA P53)	200	205	181	-2.4	21.2	101.8	5.4	1974
TLC-23	Bo. Baculong, Victoria (NIA P57)	200	263	180	-0.1	23.6	98.2	4.2	1974
TLC-24	Bo. Tinapatan, Tarlac (NIA P60)	334	200	145	-0.4	15.5	106.4	7.1	1974
TLC-25	Bo. Sta. Cruz, Tarlac (NIA P62)	319	124	95	-2.9	26.3	39.4	1.7	1974

1/ Static Water Level

2/ Pumping Water Level

TABLE VII-B-1 (Continued)

WATER WELL DATA SUMMARY

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m) Total	Cased	SWL	Test PWL	Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
TLC-26	Bo. San Jose, Tarlac (NIA P63)	200	200	171	-2.1	30.1	63.6	2.3	1974
TLC-27	Tarini, Tarlac (NIA P65)	200	199	154	-0.5	27.3	78.3	2.9	1974
TLC-28	San Andres, Victoria (NIA P67)	334	205	155	-2.1	31.1	46.6	1.7	1974
TLC-29	Bo. Naya, Pura (NIA P68)	334	190	145	-1.7	29.5	73.4	2.7	1974
TLC-30	Caluluan, Concepcion (NIA P77)	200	154	144	+1.4	17.7	100.8	5.3	
TLC-31	San Manuel, Tarlac (NIA P81)	340	303	235					
TLC-32	Tinapatan, Tarlac (NIA P83)	400	221	218					
TLC-33	Tinapatan, Tarlac (NIA P90)	400	240	175	-1.9	18.7	91.6	5.5	1975
TLC-34	Tinapatan, Tarlac (NIA P92)	400	219	217	-1.8	21.6	84.6	4.3	1975
TLC-35	Buenavista, Tarlac (NIA P95)	400	245	207					
TLC-36	Matindog, Pura (NIA P97)	400	297	237				8.9?	
TLC-37	San Fernando, San Fernando (NIA P39)	400	263	231	+1.4	5.9	10.1	1.4	1974
TLC-38	Palacpalac, Victoria (NIA P78)	400	195	139	+0.8	26.1	95.0	3.5	1974
TLC-39	Trinidad, Tarlac (NIA P85)	400			-2.7	17.8	90.8	6.0	1975
TLC-40	Sta. Cruz, La Paz (NIA P84)	400	257	247	-1.0	17.7	104.1	6.2	1975
TLC-41	Sta. Cruz, La Paz (NIA P80)	400	212	157	-1.7	23.9	96.4	4.3	1975
TLC-42	Comino, Capaz (NIA P75)	330	202	89	-2.2	31.5	57.1	2.0	1974
TLC-43	Comino, Capaz (NIA P76)	400	197	90	-4.0	25.9	69.4	3.2	1974
TLC-44	Chino, Capaz (NIA P74)	400	212	185	-3.7	28.8	84.5	3.4	1974
TLC-45	Dumaraig, La Paz (NIA P79)	400	189	129	-0.03	29.8	63.0	2.1	1974
TLC-46	Hacienda Luisita Well No. 2 Pasàje, San Miguel, Tarlac	300-230	198	183	-4.1	8.3?	54.5	13.0?	1952
TLC-47	Hacienda Luisita Well No. 3 San Sebastian, San Miguel Tarlac	300-230	195	144	-2.9	32.1	54.5	2.1	1952

TABLE VII-B-1 (Continued)

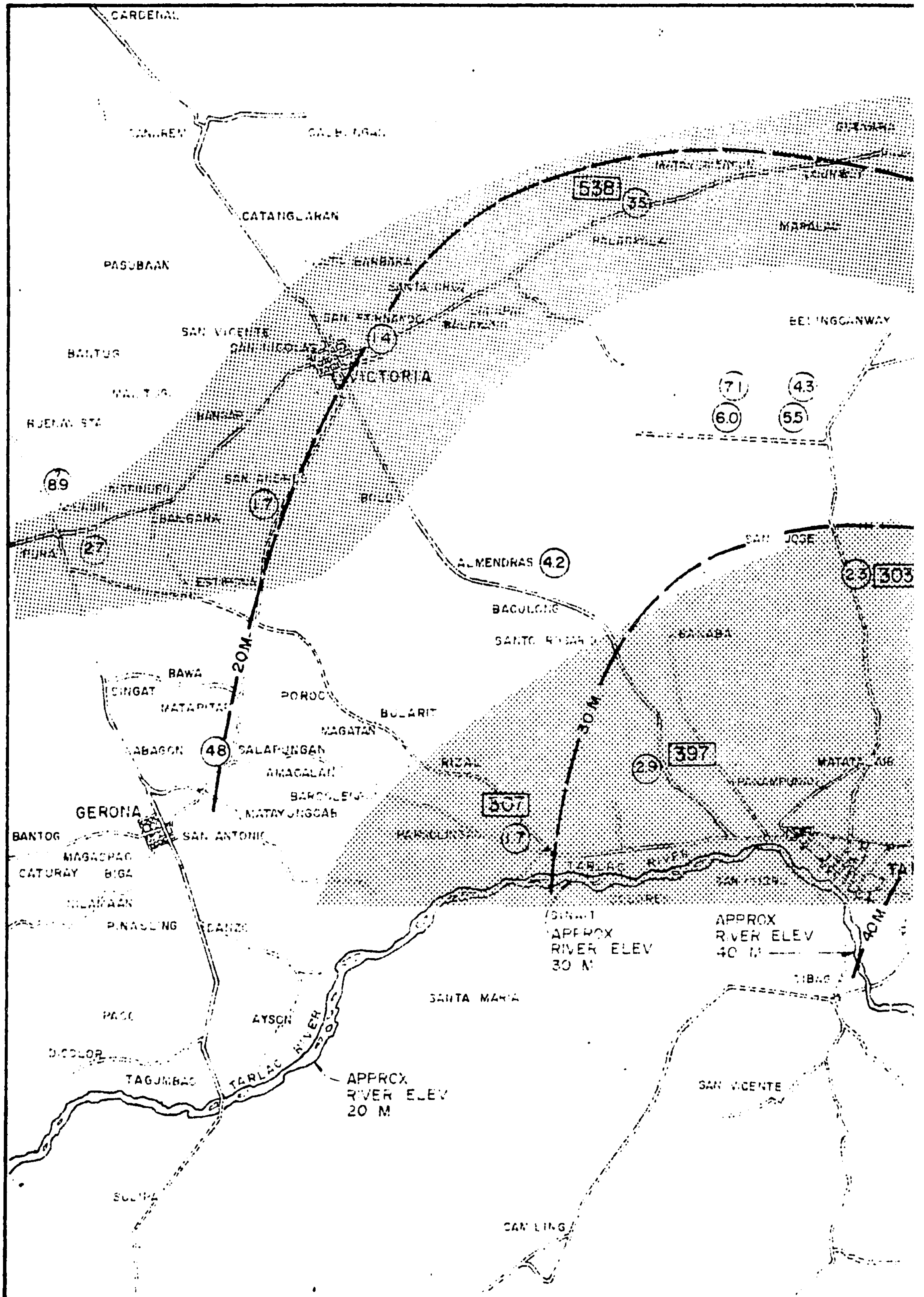
WATER WELL DATA SUMMARY

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m) Total	Cased	SWL 1/	Test PWL	Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
TLC-48	Hacienda Luisita Well No. 4 Pando, San Miguel, Tarlac	300-200	163	162					1952
TLC-49	Hacienda Luisita Well No. 5 Cutcut, San Miguel, Tarlac	300-150	183	183					1953
TLC-50	Hacienda Luisita Well No. 6 Bantog, San Miguel, Tarlac	300-150	137	137					1953
TLC-51	Hacienda Luisita Well No. 7 Cutcut, San Miguel, Tarlac	300-150	198	198					1953
TLC-52	Hacienda Luisita Well No. 8 San Miguel, Tarlac	300-150	189	189					1953
TLC-53	Hacienda Luisita Well No. 9 Mabilog, San Miguel, Tarlac	300-150	203	203					1958
TLC-54	Hacienda Luisita Well No. 10 Motrico, San Miguel, Tarlac	300-150	203	203	-0.4	7.6	37.9	5.2	1954
TLC-55	Hacienda Luisita Well No. 11 San Miguel, Tarlac	300-150	223	223					1954
TLC-56	Hacienda Luisita Well No. 12 Mabilog, San Miguel, Tarlac	300-150	198	197					1954
TLC-57	Hacienda Luisita Well No. 13 Mapalacsiao, Tarlac				-4.1		37.9		
TLC-58	Hacienda Luisita Well No. 14 Asturias, Tarlac	400-200	238	229	-7.6	28.4	142.0	6.9	1961
TLC-59	Hacienda Luisita Well No. 15 Buenavista, Tarlac	350-200	244	229	-15.2	59.5	63.1	1.4	1961
TLC-60	Hacienda Luisita Well No. 16 Mabilog, San Miguel, Tarlac	400-200	213	211					1968

TABLE VII-B-1 (Continued)

WATER WELL DATA SUMMARY

Number	Location	Nominal Diameter (mm)	Depth from Ground Surface (m) Total	Cased	SWL	Test PWL	Test Yield (lps)	Specific Capacity (lps/m)	Year Completed
TLC-61	Hacienda Luisita Well No. 17 Ungot, Tarlac				-1.6	10.7	41.0	4.6	
TLC-62	Hacienda Luisita Well No. 18 Cutcut, San Miguel, Tarlac	400-350	183	168	0	17.7	107.3	6.1	1970
TLC-63	Hacienda Luisita Well No. 19 Pando, San Miguel, Tarlac	400-200	213	213	-1.8	14.0	91.2	7.5	
TLC-64	Hacienda Luisita Well No. 20 Pando, San Miguel, Tarlac	400-200	214	214	0	20.7	94.6	4.6	
TLC-65	Hacienda Luisita Well No. 21 Pando, San Miguel, Tarlac	400-200	230	229	0	17.1	75.7	4.4	1966
TLC-66	Hacienda Luisita Well No. 22 San Miguel, Tarlac	400-300	134	132	-3.1	19.2	94.6	5.9	
TLC-67	Hacienda Luisita Well No. 24 Balete, San Miguel, Tarlac	400-300	183	183	-6.7	42.1	44.4	1.3	1974
TLC-68	Central Luisita Well No. 1 Obrero, Tarlac	200-125	195	191	-4.6	18.3	22.1	1.6	1951
TLC-69	Central Luisita Well No. 2 Distillery - Tarlac	300-150	183	183	0	11.3	41.6	3.7	1951
TLC-70	Central Luisita Well No. 3 Proxima Al Descargada, Tarlac	300-150	213	213	-2.7	13.7	37.9	3.5	1951
TLC-71	Central Luisita Well No. 4 Prente Ala Oficina Molina	300-150	213	213	-2.7	13.1	39.8	3.8	1951
TLC-72	Central Luisita Well No. 5 Camino Al Club (Bagazo) Tarlac	300-150	213	213	-2.0	9.6	37.9	5.0	
TLC-73	Central Luisita Well No. 8 Detras Oficina Gral-Tarlac	300-150	274	269	-1.5		37.9		



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

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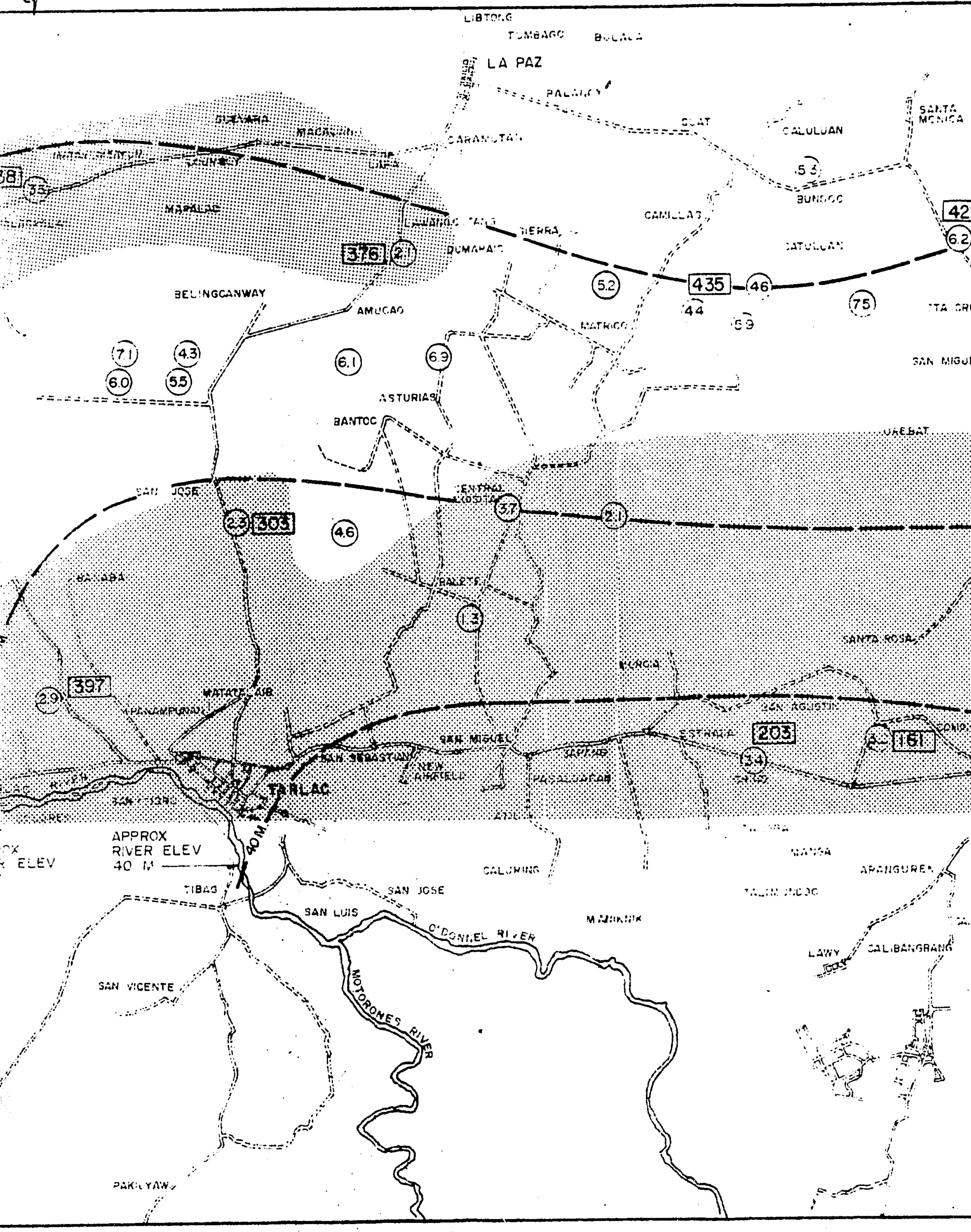
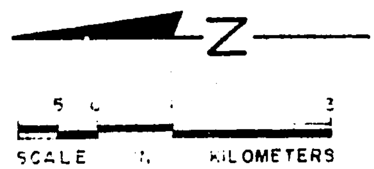
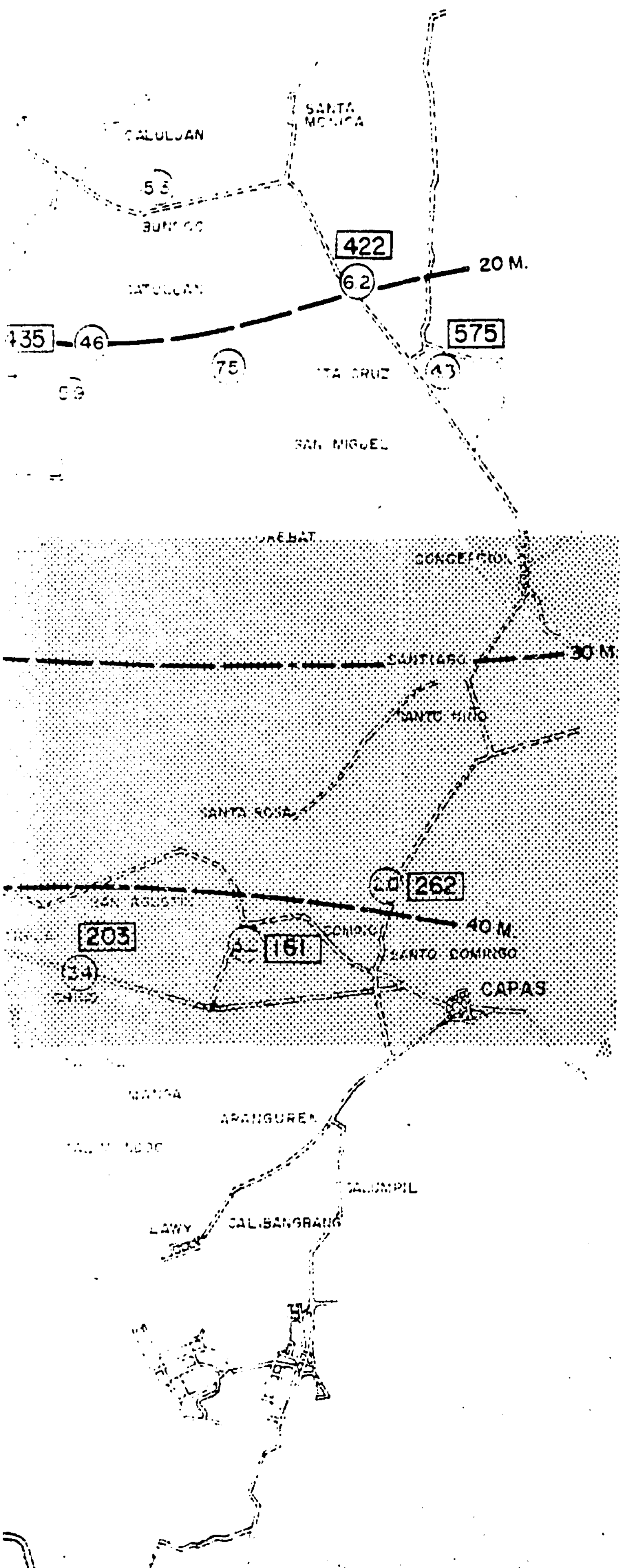


LIBTONG
TUMBAGO
LA PAZ
SANTA MONICA
CALULUAN
CARANGTAN
MAPALAD
BELINGCANWAY
AMUCAO
376
2.1
DUMARAIS
TIERRA
CAMILLAS
BUNDOC
CATULUAN
435
46
5.2
44
5.9
75
422
6.2
MATRICO
7.1
4.3
6.0
5.5
6.1
6.9
ASTURIAS
BANTOC
SAN MIGUE
SAN JOSE
CENTRAL
3.7
2.1
2.3
303
4.6
1.3
SANTA ROSA
397
2.9
MATATE AB
SAN MIGUEL
SAN SEBASTIAN
NEW
ESTRADA
203
161
3.4
SAN AGUSTIN
TARLAC
40M
APPROX
RIVER ELEV
40 M
TIBAG
SAN JOSE
SAN LUIS
O'DONNEL RIVER
MOTORONES RIVER
CALURING
ARANGUREN
LAWY
CALIBANGBANG
SAN VICENTE
PAKILYAW





LEGEND

(4.3) SPECIFIC CAPACITY IN LPS/M OF A WELL OF GOOD CONSTRUCTION

AREA OF SPECIFIC CAPACITY LESS THAN 4.0 LPS/M

—— 30 M —— APPROXIMATE CONTOUR SURFACE OF ARTESIAN AQUIFER IN METERS ABOVE SEA LEVEL

[262] WATER QUALITY OF DEEP WELL IN MG/L TOTAL DISSOLVED SOLIDS

APPENDIX FIGURE VII-B-1
SPECIFIC CAPACITY AND PIEZOMETRIC SURFACE MAP
TARLAC AREA

DESCRIPTIVE DATA	
WELL NO (CDM)	TLC 5
(OTHER)	BPW 21905
LOCATION	MABINI ST.
CITY	TARLAC
PROVINCE	TARLAC
CONST. BY	
DRILLER	
STARTED	
COMPLETED	7 OCTOBER 1960
OWNER	
STATUS	
CASING DIAMETER	200 MM , 150 MM

DRILLER'S TEST DATA:

- DATE
- STATIC WATER LEVEL 3.7 M
- PUMPING WATER LEVEL 9.8 M
- TEST PUMP YIELD 4.7 LPS

WATER QUALITY DATA:

REMARKS:

SPECIFIC CAPACITY — 0.8 LPS / M

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
2.4	8	YELLOW STICKY CLAY
7.0	23	COARSE SAND
12.2	40	YELLOW STICKY CLAY
14.3	47	YELLOW, BLUE SANDY CLAY
20.7	68	YELLOW SHALE WITH LIMESTONE & ADOBE ROCK
22.9	75	SANDSTONE
35.1	115	YELLOW STICKY CLAY
38.1	125	BLUE SANDY CLAY
39.6	130	SANDSTONE
47.2	155	BLUE SANDY CLAY & SANDSTONE
54.3	178	STICKY CLAY AND ADOBE ROCK
68.6	225	BLUE STICKY CLAY, LIME ROCK & ADOBE ROCK
77.7	255	BLUE STICKY CLAY & BLUE ADOBE
85.3	280	BLUE STICKY CLAY
103.6	340	BLUE STICKY CLAY & ADOBE ROCK
117.3	385	BLUE STICKY CLAY
123.4	405	BLUE SANDY ADOBE CLAY
129.5	425	BLUE SANDY CLAY
149.4	490	BLUE STICKY CLAY
151.8	498	YELLOW STICKY CLAY
155.5	510	LOOSE BLUE ADOBE
161.5	530	SANDY CLAY & SAND-STONE
169.2	555	

APPENDIX FIGURE VII-B-2

WELL DATA SHEET

WELL TLC - 5

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-8
(OTHER) BPW 6482
LOCATION BO. SAN NICOLAS

CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED 25 MAY 1954
COMPLETED 20 JULY 1954
OWNER

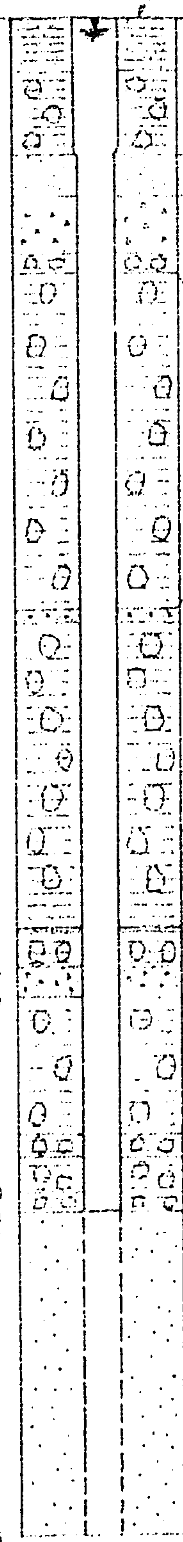
STATUS

CASING DIAMETER 200 MM - 150 MM

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL 2.40 M
PUMPING WATER LEVEL 15.2 M
TEST PUMP YIELD 6.3 LPS

REMARKS:
CASED DEPTH: 200 MM; 0 - 20.4 M
150 MM; 20.4 - 185.7 M
SPECIFIC CAPACITY = 0.5 LPS/M

GRAPHIC LOG



DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
		SANDY CLAY
7.6	25	BLUE ADOBE CLAY
21.0	69	SAND
27.4	90	SAND AND GRAVEL
36.6	120	YELLOW ADOBE CLAY
39.6	130	BLUE ADOBE CLAY
		SAND WITH GRAVEL
92.1	302	
92.7	304	BLUE ADOBE CLAY
		SAND
140.9	462	
141.5	464	BLUE ADOBE CLAY
147.6	484	FINE SAND WITH GRAVEL
152.4	500	BLUE ADOBE CLAY
173.8	570	YELLOW ADOBE CLAY
177.4	582	BLUE ADOBE CLAY
182.9	600	BLUE STICKY CLAY
185.7	609	SANDSTONE
236.6	776	

APPENDIX FIGURE VII-B-3
WELL DATA SHEET
WELL TLC-8

DESCRIPTIVE DATA	GRAPHIC LOG

WELL NO. (CDM) TLC - 10
(OTHER) BPW - 5192
LOCATION ROMULO BLVD
CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED 23 FEBRUARY 1950
COMPLETED 22 APRIL 1950
OWNER
STATUS
CASING DIAMETER 200 MM

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL 2.4 M
PUMPING WATER LEVEL 6.4 M
TEST PUMP YIELD 4.7 LPS

REMARKS.
CASED DEPTH = 106.1 M
SPECIFIC CAPACITY = 1.2 LPS/M

DEPTH (M)	DEPTH (FT)	CASING	STRATIFICATION
			GROUND SURFACE
10.7	35		YELLOW CLAY
25.9	85		BLUE ADOBE CLAY
36.6	120		YELLOW CLAY
47.3	155		BLUE STICKY CLAY
54.9	180		BLACK SANDY CLAY
69.5	228		BLACK STICKY CLAY
108.2	335		BLUE ADOBE CLAY
109.8	360		SAND ROCK
114.3	375		BLUE ADOBE
117.4	385		SANDY CLAY
126.5	415		ADOBE BLUE CLAY
138.7	455		SANDY CLAY
140.2	460		HARD ROCK
152.4	500		SANDY CLAY
163.7	537		BLUE STICKY CLAY
167.7	550		SANDY CLAY
170.1	558		SAND ROCK

APPENDIX FIGURE VII-B-4
WELL DATA SHEET
WELL TLC - 10

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-22
(OTHER) NIA-P 53
LOCATION SALAPUNGAN

CITY GERONA
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED 15 MARCH 1974
COMPLETED 24 APRIL 1974
OWNER

STATUS

CASING DIAMETER 200 MM

DRILLER'S TEST DATA:
DATE 24 APRIL 1974
STATIC WATER LEVEL 2.39 M.

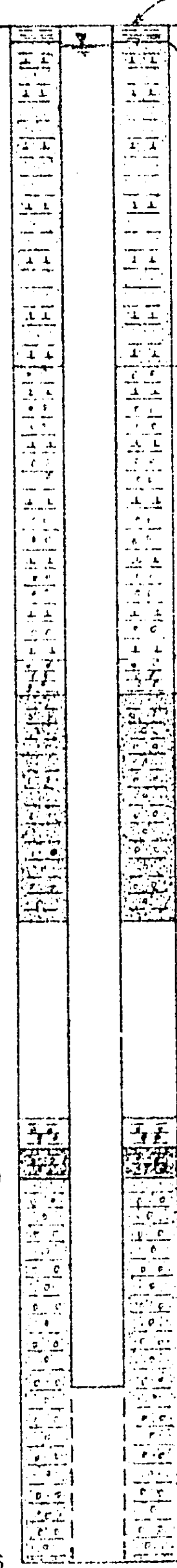
PUMPING WATER LEVEL 21.19 M

TEST PUMP YIELD 101.8 LPS

REMARKS:
SPECIFIC CAPACITY = 5.4 LPS/M
CASING DEPTH = 181 M.

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
2	6.6	CLAY
45	147.6	SAND WITH LITTLE SILT AND CLAY
84	275.6	GRAVEL WITH LITTLE SILT AND CLAY
89	292.0	CLAY WITH SOME GRAVEL
119	390.4	GRAVEL, CLAYEY SOME SAND
145	475.7	? (OMITTED FROM LOG)
149	488.9	CLAY, SOME GRAVEL
153	502.0	GRAVEL, SOME CLAY AND SAND
181	593.8	
205	672.6	CLAY, SOME SAND AND GRAVEL



APPENDIX FIGURE VII-B-5
WELL DATA SHEET
WELL TLC-22

DESCRIPTIVE DATA

WELL NO (CDM) TLC - 23
(OTHER) NIA - P57
LOCATION BARRIO BACULONG

CITY VICTORIA
PROVINCE TARLAC
CONST BY
DRILLER
STARTED 4 APRIL 1974
COMPLETED 14 MAY 1974
OWNER

STATUS

CASING DIAMETER 250 MM, 200 MM

DRILLER'S TEST DATA

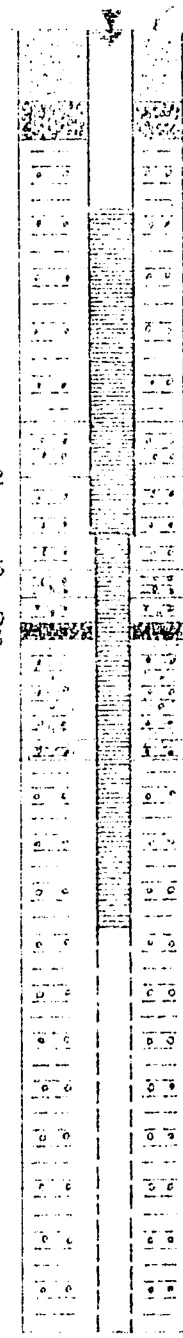
DATE 14 MAY 1974
STATIC WATER LEVEL 0.12 M

PUMPING WATER LEVEL 23.44 M
(SOMETIMES FREE FLOWING)
TEST PUMP YIED 98.3 LPS

REMARKS:

SPECIFIC CAPACITY 4.2 LPS/M
CASING DEPTH 250 MM 0 - 37 M
200 MM 174 - 180 M
SCREEN: 250 MM 37 - 101 M
200 MM 101 - 174 M

GRAPHIC LOG



DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
		SAND
15	49.2	
		GRAVEL
22	72.2	
		CLAY WITH LITTLE GRAVEL
79	259.2	
		GRAVEL, CLAYEY
90	295.2	
		CLAY WITH LITTLE GRAVEL
109	357.6	
		GRAVEL, CLAYEY
114	374.0	
		CLAY, GRAVELLY
120	393.0	
		GRAVEL
122	400.3	
		CLAY, SOME GRAVEL AND SAND
144	472.4	
		GRAVEL, CLAYEY SOME SAND
147	482.3	
180	590.6	
		CLAY WITH LITTLE GRAVEL
263	862.9	

APPENDIX FIGURE VII-B-6
WELL DATA SHEET
WELL TLC - 23

DESCRIPTIVE DATA

WELL NO. (CDM) TLC - 24
(OTHER) NIA - P60
LOCATION BARRIO TINAPATAN

CITY TARLAC
PROVINCE TARLAC
CONST BY
DRILLER
STARTED 14 MAY 1974
COMPLETED 19 JUNE 1974
OWNER

STATUS

CASING DIAMETER 334 MM

DRILLER'S TEST DATA:
DATE 19 JUNE 1974
STATIC WATER LEVEL 0.41 M.

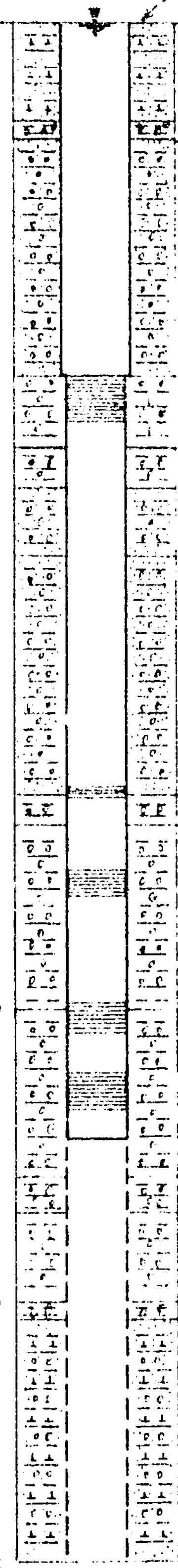
PUMPING WATER LEVEL 15.08 M.

TEST PUMP YIELD 106.6 LPS

REMARKS:

CASING DEPTH = 144.9 M
GRAVEL PACKED
TRANSMISSIVITY = 1080 CUMD/M
SPECIFIC CAPACITY = 7.3 LPS/M
SCREENED INTERVAL - 330 MM SCREEN :
46.0 – 50.4 M
99.0 – 99.9 M
111.0 – 115.4 M
127.0 – 129.9 M
137.0 – 139.9 M
SLOT SIZE – 60 AND 80

GRAPHIC LOG



DEPTH (M)	DEPTH (FT)	STRATIFICATION
0	0	GROUND SURFACE
		SAND WITH SOME SILT AND CLAY
13	42.7	
15	49.2	GRAVEL W/ LITTLE CLAY
		CLAY WITH SOME GRAVEL AND SAND
46	150.9	
		GRAVEL WITH LITTLE CLAY
55	180.5	CLAY, SOME GRAVEL
60	196.9	GRAVEL, CLAYEY WITH SOME SAND
69	226.4	
		CLAY WITH SOME GRAVEL AND SAND
100	328.1	GRAVEL, CLAYEY
104	341.2	
		CLAY, SOME GRAVEL
128	420.0	
		GRAVEL WITH LITTLE CLAY
144.9	475.4	
150	492.1	CLAY AND GRAVEL
153	502.0	
		CLAY, SOME GRAVEL
166	544.6	GRAVEL, CLAYEY
168	551.2	
		CLAY, LITTLE GRAVEL, SILTY, FINE SAND
200	656.2	

APPENDIX FIGURE VII-B-
WELL DATA SHEET
WELL TLC-24

DESCRIPTIVE DATA	
WELL NO. (CDM)	TLC-25
(OTHER)	NIA P 62
LOCATION	STA. CRUZ
CITY	TARLAC
PROVINCE	TARLAC
CONST. BY	
DRILLER	
STARTED	12 JULY 1974
COMPLETED	14 JULY 1974
OWNER	NIA
STATUS	
CASING DIAMETER	400 MM AND 319 MM

DRILLER'S TEST DATA:

DATE	14 JULY 1974
STATIC WATER LEVEL	2.91 M.
PUMPING WATER LEVEL	26.32 M.
TEST PUMP YIELD	39.5 LPS

REMARKS:

SPECIFIC CAPACITY 1.7 LPS/M

CASING DEPTH 400 MM : 0 - 42.06, 319 MM, 42.06 - 48.17, 51.34 - 57.45, 60.62 - 66.06, 89.23 - 95.40 M.

SCREEN : 335 MM (80 SLOTS) : 48.17 - 51.34, 57.45 - 60.62, 66.06 - 89.23 M.

HOLE WAS BACKFILLED FROM 124 M TO 95.40 M

GRAPHIC LOG

DEPTH (M.)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
1	3.28	SAND, MEDIUM TO COARSE
2.91	9.55	GRAVEL, PEBBLE SIZED
5	16.40	SAND, MEDIUM TO VERY COARSE
9	29.52	GRAVEL, PEBBLE SIZED CLAYEY
12	39.36	SAND, FINE TO MEDIUM TO VERY COARSE
15	49.20	GRAVEL, PEBBLE SIZED
31	101.68	CLAY, GRAVELLY
58	190.24	GRAVEL, CLAYEY
61	200.08	CLAY, GRAVELLY, SILTY
68	223.04	SILT, SOME CLAY
84	275.52	CLAY, SILTY, VERY LITTLE GRAVEL
95.40	312.91	
124	406.72	

APPENDIX FIGURE VII-5
WELL DATA SHEET
WELL TLC-25

DESCRIPTIVE DATA

WELL NO. (CDM) TLC 26
(OTHER) NIA P 63
LOCATION BO. SAN JOSE

CITY
PROVINCE TARLAC
CONST. BY
DRILLER NIA
STARTED
COMPLETED 30 JULY 1974
OWNER

STATUS

CASING DIAMETER 335 MM, 200 MM.

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL 2.06 M

PUMPING WATER LEVEL 30.06 M

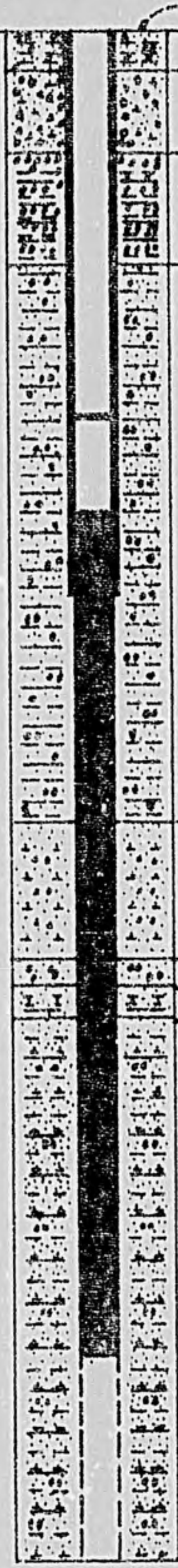
TEST PUMP YIELD 63.6 LPS

WATER QUALITY DATA:
TOTAL DISSOLVED SOLIDS:
314 PPM
292 PPM

REMARKS:
CASING DEPTH – 171.36 M
GRAVEL PACKED
TRANSMISSIVITY – 250 CUMD/M
SPECIFIC CAPACITY – 2.3 LPS/M
SCREENED INTERVAL –
336 MM SCREEN:
50.0 – 61.4 M.
62.0 – 73.5 M.
200 MM SCREEN:
73.8 – 171.4M.
SLOT SIZE – 60

GRAPHIC LOG

DEPTH (M)	DEPTH (FT.)	CASING	STRATIFICATION
			GROUND LEVEL
			CLAY, SAND & GRAVEL
5			SAND, COARSE-GRAINED, SOME SILT & GRAVEL
16			GRAVEL AND LITTLE CLAY
30			CLAY, GRAVELLY & SOME SAND
103			SAND, FINE-GRAINED AND SILTY
121			GRAVEL, SANDY, SILTY
124			SAND, SILTY & CLAYEY
128			CLAY, VERY LITTLE GRAVEL, SILTY & SOME SAND
200			



APPENDIX FIGURE VII-8-9
WELL DATA SHEET
WELL TLC-26

DESCRIPTIVE DATA

WELL NO. (CDM) TLC 27
(OTHER) NIA P 68
LOCATION TARIJI

CITY
PROVINCE TARLAC
CONST. BY
DRILLER NIA
STARTED
COMPLETED 6 AUGUST 1974
OWNER

STATUS

CASING DIAMETER 400 MM, 335 MM, 200 MM.

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL 0.8 M.

PUMPING WATER LEVEL 27.3 M

TEST PUMP YIELD 78.4 LPS

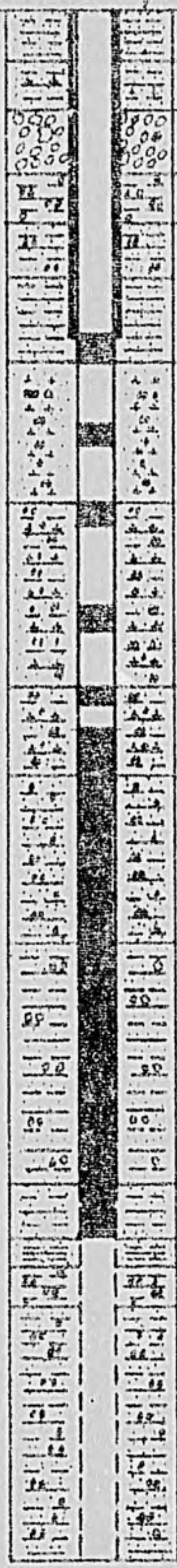
WATER QUALITY DATA:
TOTAL DISSOLVED SOLIDS – 397 PPM

REMARKS:
CASING DEPTH – 157.0 M
GRAVEL PACKED
TRANSMISSIVITY – 378 CUMD/M
SPECIFIC CAPACITY – 3.0 LPS/M
SCREENED INTERVALS –
335 MM SCREEN:
42.0 – 44.9 M
56.5 – 56.9 M
64.3 – 66.9 M
73.0 – 75.9 M
85.5 – 88.4 M
90.0 – 92.9 M
200 MM SCREEN:
92.9 – 126.5 M
138.7 – 153.9 M

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	STRATIFICATION
		GROUND SURFACE
		SAND & SOME CLAY
6		CLAY, SANDY & SILTY
12		GRAVEL COBBLE TO BOULDER SIZED
21		CLAY, GRAVELLY
27		SAND, SOME GRAVEL AND CLAY
34		CLAY, SANDY
46		GRAVEL, SANDY AND SILTY
63		CLAY, SOME SAND AND GRAVEL, SILTY
87		GRAVEL, CLAYEY, SOME SAND
98		CLAY, SOME GRAVEL AND SAND
120		CLAY, LITTLE GRAVEL AND SAND
151		CLAY, SANDY
158		SAND, SILTY & CLAY
161		CLAY, GRAVELLY
167		SAND, SOME GRAVEL AND CLAY
199		

CASING



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

APPENDIX FIGURE VI-B-5?
WELL DATA SHEET
WELL TLC-27

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-28
(OTHER) NIA-P67
LOCATION SAN ANDRES

CITY VICTORIA
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED 17 JULY 1974
COMPLETED 4 SEPTEMBER 1974
OWNER

STATUS

CASING DIAMETER 334 MM

DRILLER'S TEST DATA:
DATE 4 SEPTEMBER 1974
STATIC WATER LEVEL 2.10 M.

PUMPING WATER LEVEL 29.60 M.

TEST PUMP YIELD 46.7 LPS

REMARKS:
SPECIFIC CAPACITY = 1.7 LPS/M.

GRAPHIC LOG

DEPTH (M)	DEPTH (FT.)	CASING	STRATIFICATION
			GROUND SURFACE
2.1	6.9		CLAY
5	16.4		SAND
9	29.5		CLAY, SILTY
14	45.9		GRAVEL
16	52.5		SAND, SILTY
20	65.6		
			CLAY, SILTY SOME SAND AND LITTLE GRAVEL
39	128.0		
			GRAVEL AND CLAY
55	180.5		
			CLAY, SANDY AND SILTY
126	413.4		
			SAND AND CLAY
135	442.9		
			GRAVEL, SANDY WITH LITTLE CLAY
154.5	506.9		
160	524.9		
			CLAY, SOME SAND SILTY WITH GRAVEL
196	643.1		
			GRAVEL AND CLAY
205	672.6		

APPENDIX FIGURE VII-B-[illegible]
WELL DATA SHEET
WELL TLC-28

DESCRIPTIVE DATA	
WELL NO. (CDM)	TLC-29
(OTHER)	NIA-P98
LOCATION	BARRIO NAYA
CITY	PURA
PROVINCE	TARLAC
CONST. BY	
DRILLER	
STARTED	30 JULY 1974
COMPLETED	5 SEPTEMBER 1974
OWNER	
STATUS	
CASING DIAMETER	334 MM

DRILLERS TEST DATA:

DATE	5 SEPTEMBER 1974
STATIC WATER LEVEL	1.38 M.
PUMPING WATER LEVEL	29.90 M.
TEST PUMP YIELD	75.5 LPS

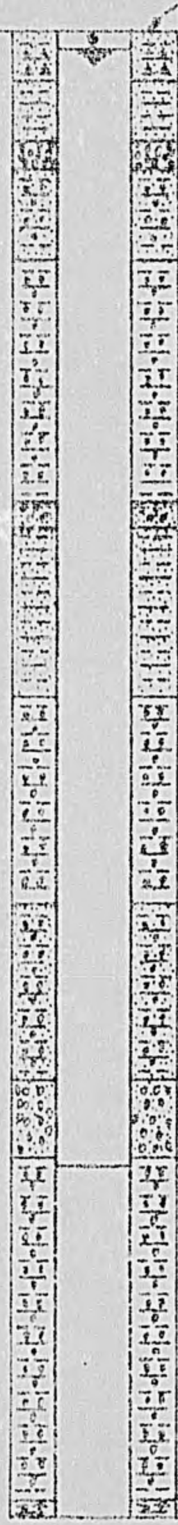
REMARKS:

SPECIFIC CAPACITY = 2.7 LPS/M.

GRAPHIC LOG

DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
		CLAY AND SAND, SILTY
6	19.7	
		CLAY WITH LITTLE SAND
14	45.9	
		GRAVEL
17	55.8	
		CLAY, SANDY AND GRAVELLY
29	95.1	
		GRAVEL AND CLAY
60	196.9	
		SAND AND GRAVEL
63	206.7	
		CLAY, SANDY
85	278.9	
		GRAVEL WITH LITTLE CLAY
111	364.2	
		CLAY, GRAVELLY, SOME SAND
134	439.6	
		GRAVEL, SOME SAND
144	472.4	
144.96	475.6	
		CLAY WITH SOME GRAVEL
188	616.8	
		GRAVEL, CLAYEY
190	623.4	

CASING



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

APPENDIX FIGURE VII-B-12
WELL DATA SHEET
WELL TLC-29

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-30
(OTHER) NIA - 977
LOCATION CALULUAN

CITY CONCEPCION
PROVINCE TARLAC
CONST BY
DRILLER
STARTED 24 NOV. 1974
COMPLETED
OWNER

STATUS

CASING DIAMETER 200 MM

DRILLER'S TEST DATA:

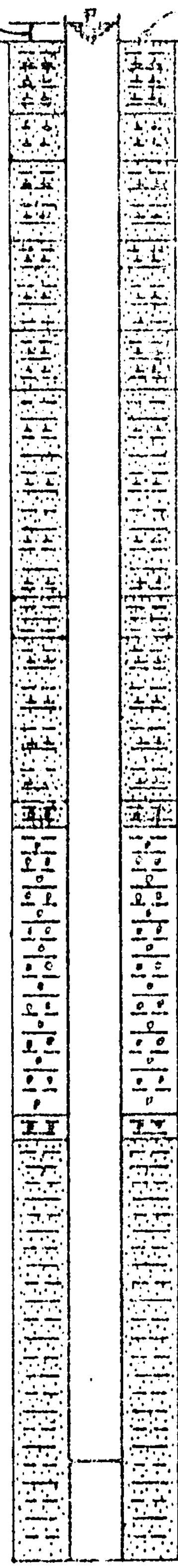
DATE
STATIC WATER LEVEL 1.40 M. ABOVE

PUMPING WATER LEVEL 19.14 M.

TEST PUMP YIELD 101.0 LPS

REMARK.
SPECIFIC CAPACITY 4.9 LPS/M.

GRAPHIC LOG



DEPTH (M)	DEPTH (FT.)	STRATIFICATION
1.4 M.		GROUND SURFACE
		SAND AND CLAY SILTY
7	23.0	SAND, SILTY
12	39.4	CLAY, SOME SAND, SILTY
20	65.6	SAND, SILTY, CLAYEY
29	95.1	CLAY AND SAND, SILTY
35	114.8	CLAY, SANDY WITH LITTLE SILT
56	183.7	SAND AND CLAY
60	196.9	SAND AND CLAY, SILTY
77	252.6	GRAVEL, SOME SAND AND LITTLE CLAY
79	259.2	CLAY WITH LITTLE GRAVEL
109	357.6	GRAVEL, SOME CLAY
111	364.2	CLAY SOME SAND
144.2	473.1	
154	505.3	

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

APPENDIX FIGURE VII-2-13
WELL DATA SHEET
WELL TLC - 30

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-31
(OTHER) NIA-P8I (T-I)
LOCATION SAN MIGUEL

CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED
COMPLETED
OWNER

STATUS

CASING DIAMETER

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL

PUMPING WATER LEVEL

TEST PUMP YIELD

REMARKS:
SPECIFIC CAPACITY

NO PUMP TEST DATA

GRAPHIC LOG

DEPTH (M)	DEPTH (FT)	CASING	STRATIFICATION
			GROUND SURFACE
			CLAY
4	13.1		SAND AND GRAVEL
14	45.9		CLAY
18	59.0		SAND
22	72.2		CLAY, SOME SAND AND SILT, LITTLE GRAVEL
53	173.8		GRAVEL
57	187.0		CLAY, SOME GRAVEL SAND, AND SILT
86	282.1		CLAY AND GRAVEL
92	301.8		GRAVEL
105	344.4		CLAY
113	370.6		GRAVEL
120	393.6		CLAY, SOME FINE GRAVEL, SAND AND SILT
303	993.8		

APPENDIX FIGURE VII-B-14
WELL DATA SHEET
WELL TLC-31

DESCRIPTIVE DATA

WELL NO. (CDM) TLC-32
(OTHER) NIA P83 (T-2)
LOCATION TINAPATAN

CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED
COMPLETED
OWNER

STATUS

CASING DIAMETER

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL

PUMPING WATER LEVEL

TEST PUMP YIELD

REMARK
SPECIFIC CAPACITY

NO PUMP TEST DATA

GRAPHIC LOG

DEPTH (M.)	DEPTH (FT.)	CASING	STRATIFICATION
			GROUND SURFACE
			CLAY AND SAND
13	42.6		GRAVEL
21	68.9		CLAY, LITTLE GRAVEL, SAND AND SILT
92	301.8		GRAVEL AND SAND
103	337.8		CLAY
108	354.2		GRAVEL
111	364.1		CLAY
122	400.2		GRAVEL
128	419.8		CLAY
132	433.0		GRAVEL, LITTLE SAND AND CLAY
145	475.6		CLAY, GRAVELLY
150	492.0		GRAVEL, SANDY
167	547.8		CLAY, SOME GRAVEL
173	567.4		GRAVEL, SOME SAND CLAY, AND SILT
200	656.0		CLAY
205	672.4		SANDSTONE
209	685.5		CLAYSTONE
221	724.9		

APPENDIX FIGURE VII-B-15
WELL DATA SHEET
WELL TLC-32

DESCRIPTIVE DATA	GRAPHIC LOG

WELL NO. (CDM) TLC-33
(OTHER) NIA-P90(T-8)
LOCATION TINAPATAN

CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED
COMPLETED
OWNER

STATUS

CASING DIAMETER

DRILLER'S TEST DATA:
DATE
STATIC WATER LEVEL

PUMPING WATER LEVEL

TEST PUMP YIELD

REMARKS:
NO PUMP TEST DATA

DEPTH (M)	DEPTH (FT.)	CASING	STRATIFICATION
			GROUND SURFACE
			BROWN CLEAN SAND WITH SILT AND CLAY IN PART
7	23.0		BROWN GRAVEL WITH SAND AND SILT
14	45.9		BROWN SILT AND CLAY
22	72.2		BROWN SAND WITH SILT AND GRAVEL
30	98.4		BROWN SILT AND CLAY
43	141.0		GRAY SAND AND GRAVEL WITH SILT
74	242.7		GRAY GRAVEL, SILT AND CLAYEY
101	331.3		GRAY GRANULE GRAVEL AND SAND
117	383.8		SILT AND CLAY
131	429.7		LIGHT GRAY TO WHITE SILT AND SAND WITH ASH
156	511.7		GREEN GRAY SILT AND SAND
175	574.0		SAND
185	606.8		ASH
200	656.0		SANDY SILT AND CLAY
205	672.4		GRAVEL AND SAND WITH LITTLE SILT AND CLAY
225	738.0		CEMENTED GRAVEL AND SAND
240	787.2		

APPENDIX FIGURE VII-8.48
WELL DATA SHEET
WELL TLC-33

DESCRIPTIVE DATA

WELL NO (CDM) TLC-34
(OTHER) NIA-P 92
LOCATION TINAPATAN

CITY TARLAC
PROVINCE TARLAC
CONST. BY
DRILLER
STARTED
COMPLETED
OWNER

STATUS

CASING DIAMETER

DRILLER'S TEST DATA:
- DATE
- STATIC WATER LEVEL
- PUMPING WATER LEVEL
- TEST PUMP YIELD

REMARKS:
NO PUMP TEST DATA

GRAPHIC LOG

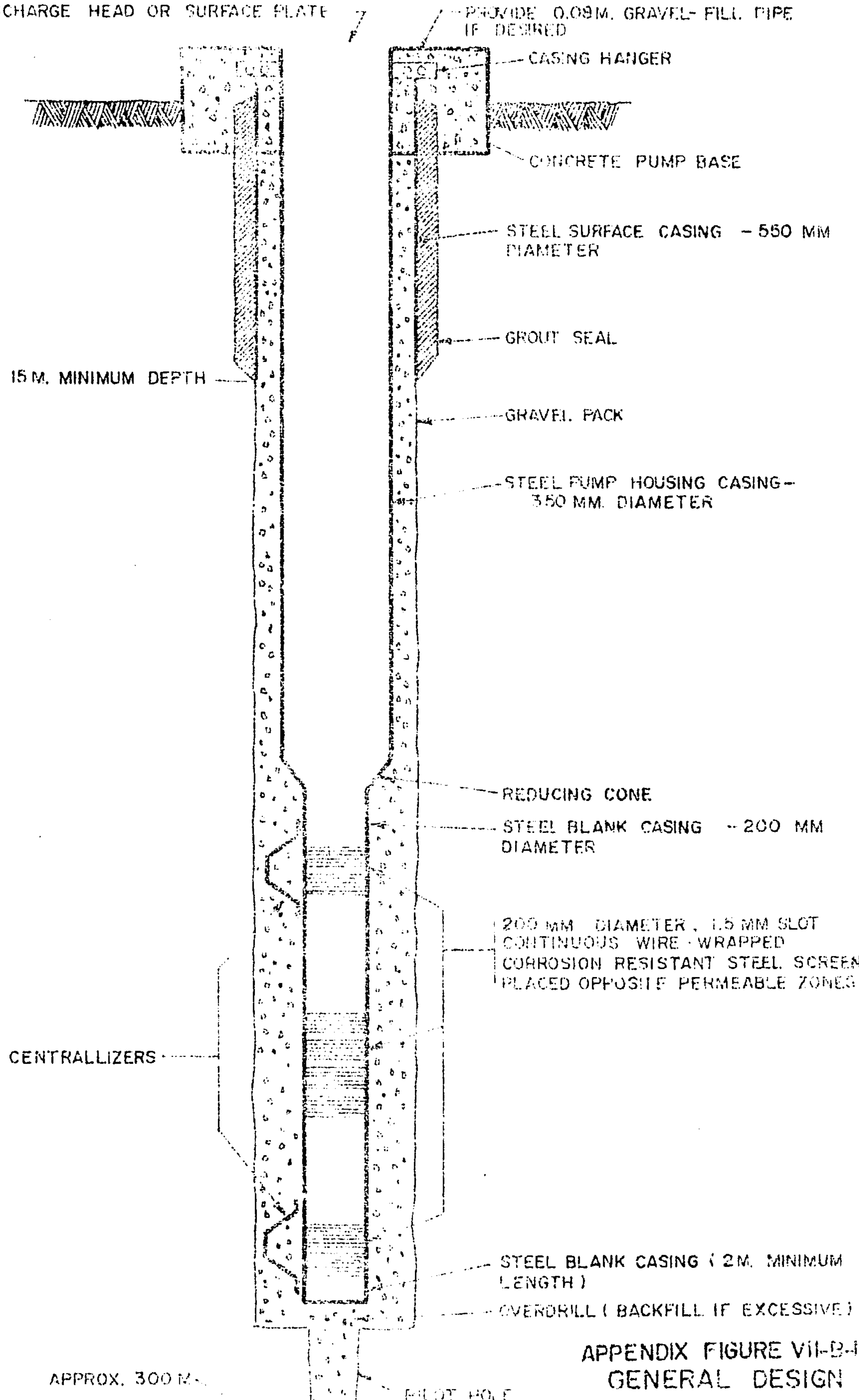
DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
		GRAY SAND AND SILT
10	32.8	BROWN SILT AND CLAY
19	62.3	BROWN SAND AND SILT
31	101.7	BROWN SILT AND CLAY
57	187.0	GRAY SILT WITH SOME SAND AND CLAY
74	242.7	GRAY BROWN SILT AND CLAY WITH SOME SAND AND GRAVEL
96	314.9	GRAY BROWN SAND AND GRAVEL
117	383.8	BROWN CLAY WITH SOME SAND AND GRAVEL
136.5	447.7	SILTY SAND
151	495.3	SILT, CLAY AND FINE SAND WITH SOME GRAVEL
171	560.9	BROWN GRAY CLAY WITH SILT
181	593.7	SANDY SILT WITH CLAY
218.95	718.2	

CASING

APPENDIX FIGURE VII-B-17
WELL DATA SHEET
WELL TLC-34

NOTE:

PROVIDE OPENING FOR WELL SOUNDING IN PUMP DISCHARGE HEAD OR SURFACE PLATE



APPENDIX FIGURE VII-B-18
GENERAL DESIGN
GRAVEL PACKED WELL
ROTARY DRILLED

SUPPLEMENT TO FIGURE VII-B-18

GENERAL CONSTRUCTION SUGGESTIONS

Gravel Packed Well - Rotary Drilled

1. Drill oversized hole to 15m 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 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 suitable 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.

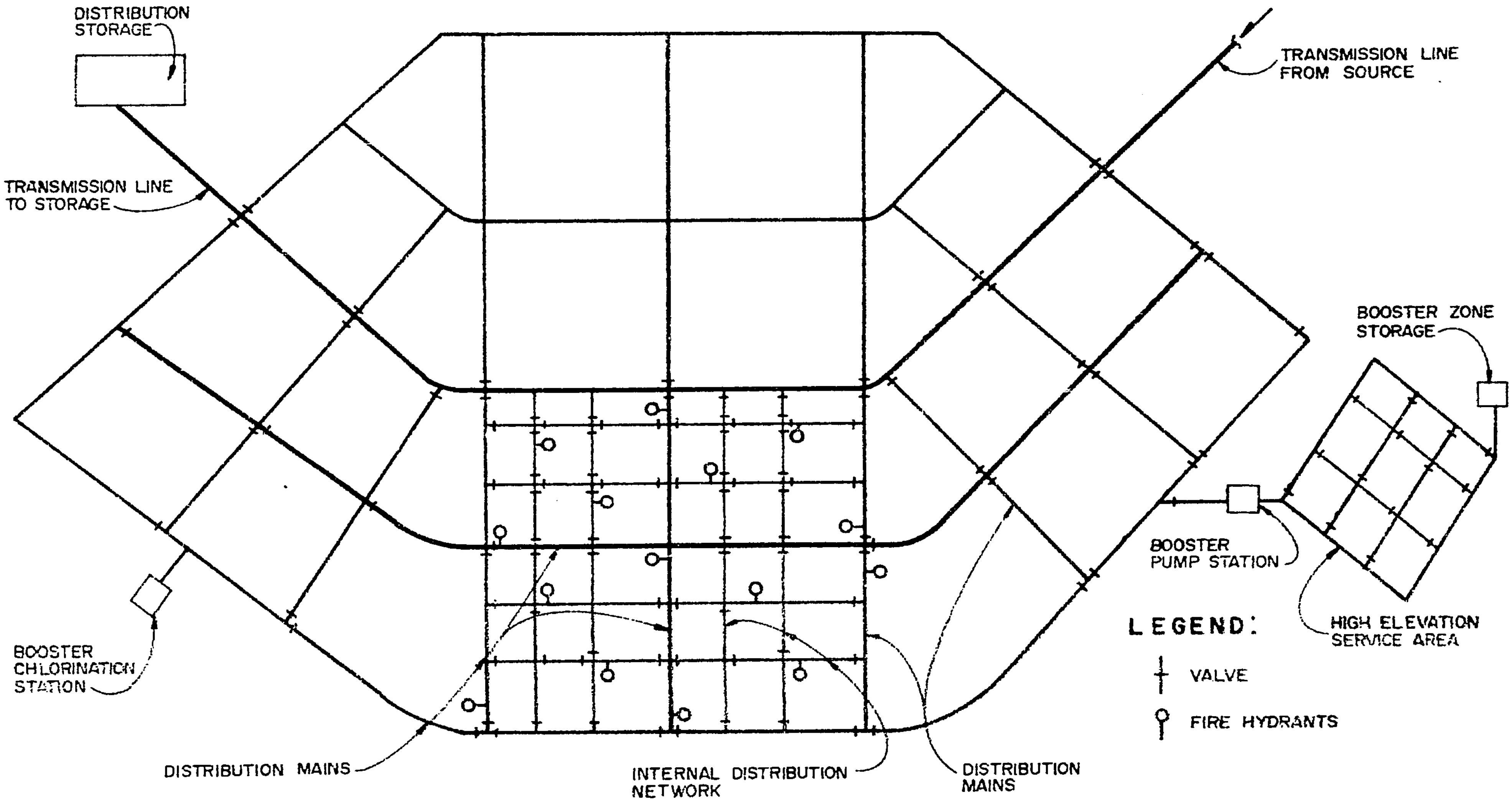
Each component of the distribution system and its respective alternatives are discussed in subsequent sections.

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-1

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] (₱/m)	Project Cost (₱/m)	Annual Cost (₱/m)	Capital Cost (₱/m)	Annual Cost (₱/m)	Salvage Value (₱/m)	Net Cos (₱/
Single-Stage	1980	250	475	648	3	412	14	19	40
							Total		40
Two-Stage	1980	200	360	491	2	312	9	14	30
	1990	200	414	565	2	116	2	19	9
							Total		40

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 the source elevation 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 (or continuous manual control) 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 meters. 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 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

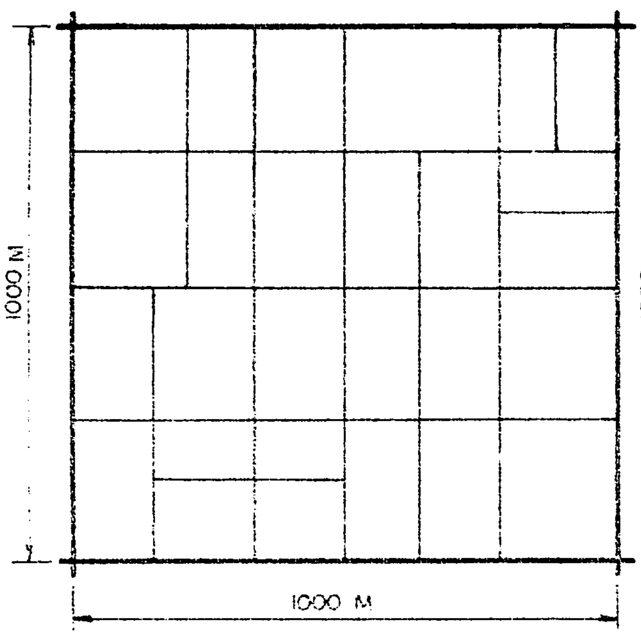
The internal network design is controlled by either of two conditions: peak-hour demands with minimum main pressure of 10 m or fire-flow demand coincident with maximum-day demands with a minimum hydrant pressure of 7 meters.

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:

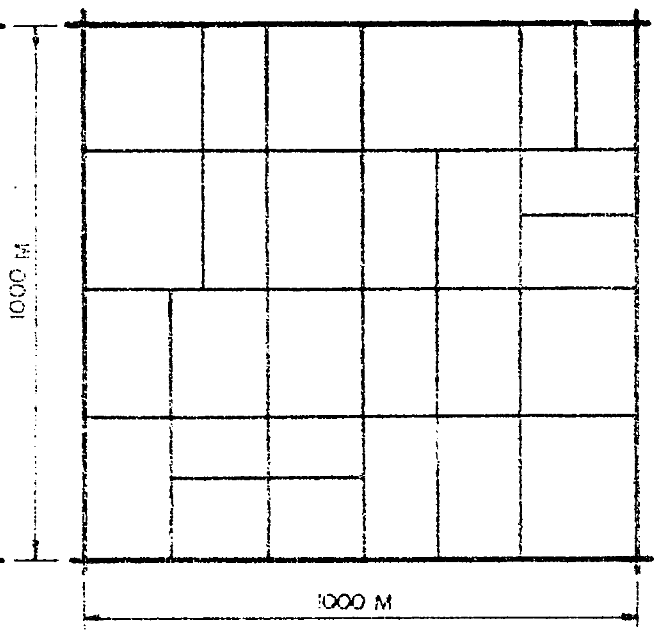
Alternative 1. All internal network pipe is 100 mm in diameter.

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

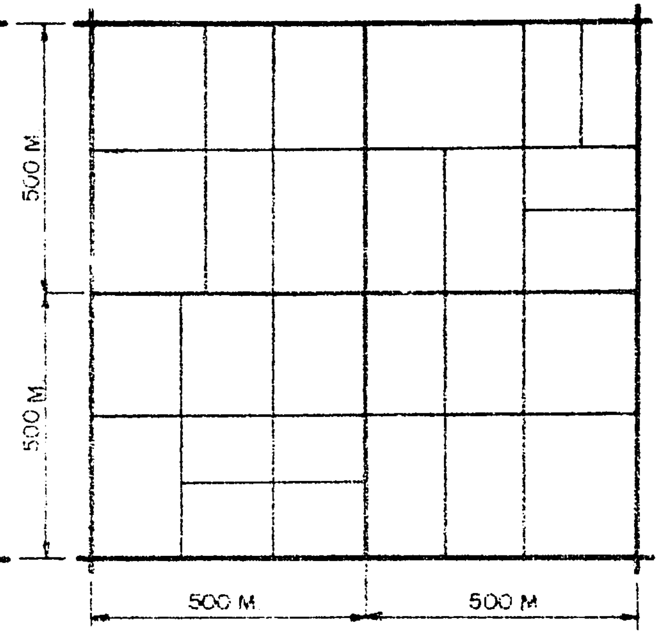
Alternative 3. The ratio of 100 mm pipe to 150 mm pipe is 3:1, i.e., 6,000 m of 100 mm pipe and 2,000 m of 150 mm pipe.



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:.	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 booster 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.

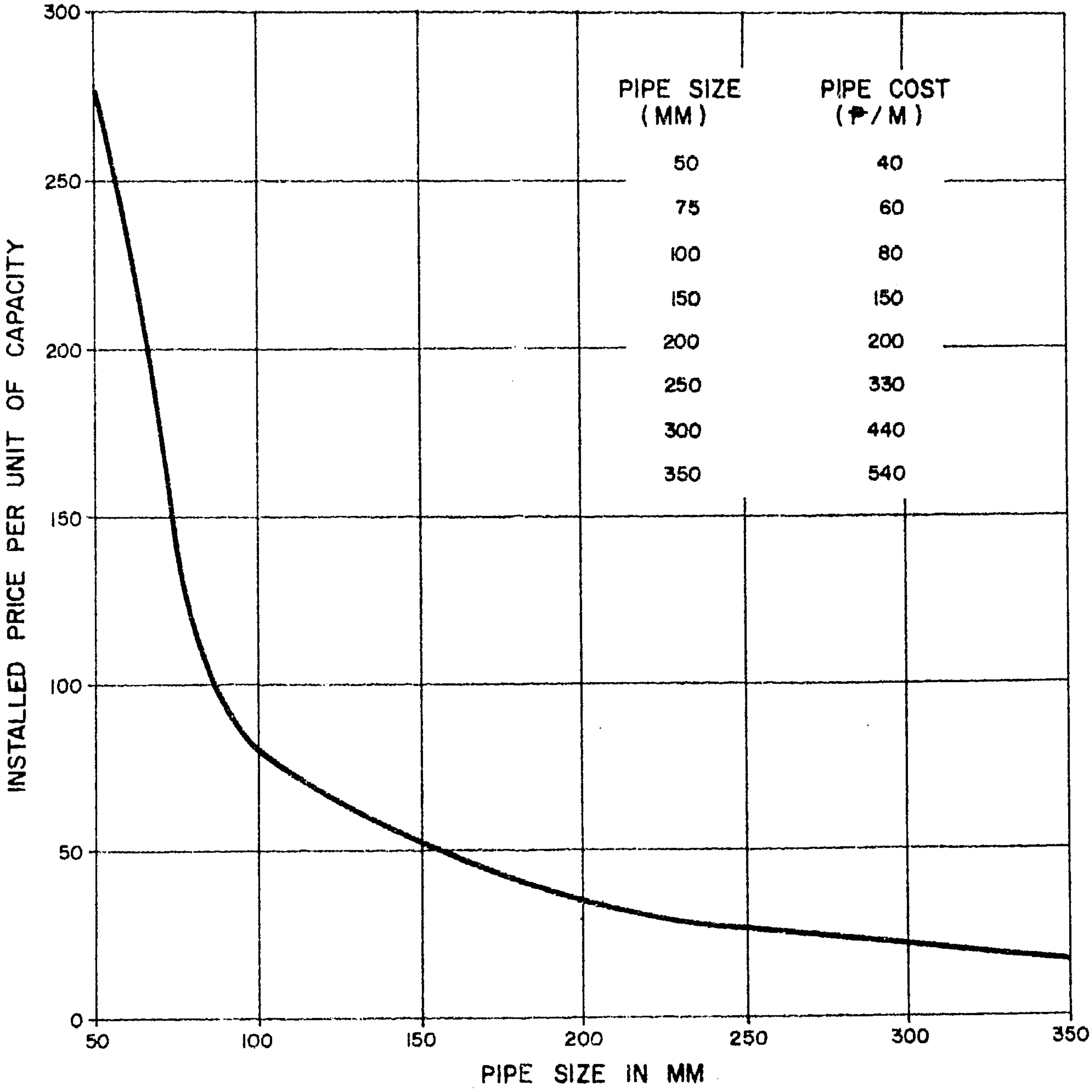
Booster pump station should have at least two pumps to permit maintenance without interrupting service. One of the units should be diesel or gas-powered to permit a minimum level of service interruption during power shortages.

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.

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 wastewater 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).

<u>Deep Water Level HPW</u>. 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 stream-bank 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 minimize 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 utilizing 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 EXISTING SYSTEM

Item 1. Install approximately 1,020 m of 200-mm pipe parallel to the existing 75-mm pipeline between the connection to Well No. 6 and the end of the 100-mm pipeline on Vicente Street. This will permit considerably greater production from Wells No. 6 and 7, presently constricted by the high head losses in the existing 75-mm pipeline. The new 200-mm pipeline will be incorporated into the long-term program. Materials required for this work include:

(a) One-1,020 m - 200-mm Pipe
(b) Two-200 x 75 mm Tees
(c) One-200 x 100 mm Reducer
(d) Two-150 mm Valves

In addition, the two valves located along the existing 75-mm pipeline (presently throttled or closed) should be fully opened.

Item 2. The existing elevated storage tank, which presently serves only nearby government offices, should be re-piped to "float" on the distribution system. This will permit better service and pressures within the system.

In addition, Wells No. 1 and 4 should be disconnected from the individual 100-mm fill pipes leading to the top of the elevated reservoir, and connected directly to the 200-mm pipeline along Makabulos Street. By making the alterations and operating the appropriate reservoir valves, both the wells and the reservoir will "float" on the system, providing better customer service at greater efficiency.

The following materials will be required for these works:

<u>For Reconnection of Well No. 4</u>

(a) 10 m - 100-mm Pipe
(b) One - 200 x 100 mm Tee

For Reconnection of Well No. 1 to Discharge Line of Well No. 2

(c) One - 100 x 100 mm Tee

For Connection of Reservoir to 100-mm Pipeline to Romulo Street

(d) One - 150 x 100 mm Reducer

In addition, approximately 200 m of 100-mm pipe previously connected to Wells No. 1 and 4 can be recovered for use elsewhere.

Item 3. As system performance is improved by additional source capacity and reconnection of the reservoir, certain pipeline connections and interties between existing pipes should be made as follows:

Elimination of Pipeline "Gaps"

(a) Between 100-mm pipes along Tañedo St. at J. Luna Street.

(b) Between 100-mm pipes along Mabini St. at J. Luna Street.

(c) Between 100-mm pipes along Espino St. between Mabini St. at Hilario Street.

(d) Between 75-mm pipes along Espino St. between Mabini St. and Tañedo Street.

Interties

(e) Between "old" and "new" 100-mm pipes along Mabini St. at Espinos Street

(f) Between 100-mm and 100-mm pipes at the intersection of Romulo St. and Zamora Street

(g) Between the 75-mm and 100-mm pipes along Romulo St. near the reservoir.

Item 4. Well No. 3 was previously removed from service, reportedly because of surface contamination. If the following steps prove successful, this well may be returned to service.

(a) Remove concrete pumphouse floor in the vicinity of the well.

(b) Excavate a 0.3-m annulus around the well casing.

(c) Fill the excavated annulus with cement-rich concrete.

(d) Repair the pumphouse floor ensuring that a slope is maintained away from the well.

(e) Remove the source of contamination.

(f) Disinfect the well with any available chlorinating agent, surging the well to ensure good penetration.

(g) Pump the well to waste until the water produced is "clean" and free of chlorine.

(h) Re-test well water samples to check the absence of pollutants.

(i) If the well water proves to be potable and safe, re-commission the well, if not, abandon it.

Item 5. 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 one new pumpset has been included in the Early Action Program cost estimates shown in Table IX-1.

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 consumers 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 Projections			
	1975	1985	1990	2000
A. Present Service Area	5,100	31,500	38,600	48,500
B. 1990 Study Area		30,700	43,400	72,800
C. Year 2000 Study Area			5,500	15,800
Total	5,100	62,200	87,500	137,100

Number of Consumers Served per Connection

The present average number of consumers per connection in the TWD is estimated to be 6.9. 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 shown below:

	Number of Persons per Connection			
	1975	1985	1990	2000
A. Present Service Area	6.9	6.5	6.3	6
B. 1990 Study Area		6.5	6.2	5.5
C. Year 2000 Study Area			6.0	5.5

The number of persons per connection is assumed to be higher in the present service area than in the service area extension.

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 TWD serves approximately 275 ha in and around the core city. There are 746 concessionaires or an average of 2.7 connections per hectare. This is a low figure for connections per hectare and reflects the poor water service presently provided by the TWD. Such conditions indicate that as service is improved the water district should expect a significant increase in the number of concessionaires. The number of connections per hectare in the present service area is expected to increase to 27 by 1990 and to 35 by year 2000. The method used in calculating the number of hectares to be served in the 1990 study area for the year 1985 is illustrated below:

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

$$= \frac{30,700}{6.5 \times 15} = 315 \text{ ha}$$

The 315 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 would be approximately 375 hectares. The projected service area for the TWD is listed in Appendix Table IX-C-1.

Area Served by Internal Network System

The present service area of 275 ha in the TWD can be separated into two types of service - service by connection to the internal

APPENDIX TABLE IX-C-1

PROJECTION OF AREA SERVED

Study Area	Year 1985: Number of Connections per Hectare	Year 1985: Area Served (ha) Net	Year 1985: Area Served (ha) Gross	Year 1990: Number of Connections per Hectare	Year 1990: Area Served (ha) Net	Year 1990: Area Served (ha) Gross	Year 2000: Number of Connections per Hectare	Year 2000: Area Served (ha) Net	Year 2000: Area Served (ha) Gross
A. Present Service Area	21	229	275	27	229	275	35	229	275
B. 1990 Study Area	15	315	375	18	389	465	25	529	[illegible]
C. Year 2000 Study Area			—	12	78	90	15	192	[illegible]
Total			650			830			1,[illegible]

network system and service by connection to distribution mains. Concessionaires served by mains of 100 mm in diameter or smaller are those served by the internal network system. Those concessionaires connected to pipe 150 mm in diameter and larger are considered to be served by distribution mains. Of the present service area of 275 ha, 270 ha is served by the internal network system and five hectares, by distribution mains. It is assumed that the practice of making direct connection to distribution mains will continue, and therefore, the area served by distribution 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 distribution mains. No service area is attributed to those distribution mains passing through areas of very low population density or to mains parallel to existing mains. The areas served by distribution mains are listed below:

	1985	1990	2000
Length (m) of distribution mains likely to support concessionaires	26,000	37,000	59,200
Corresponding area (ha)	130	185	300

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 TWD will have extended internal network service to serve a total of 645 ha and by year 2000 to serve a total of 840 ha. The present area served by internal network will be reduced as larger distribution mains will serve a wider part of the existing service area.

Besides installing 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 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 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:

APPENDIX TABLE IX-C-2

PROJECTED AREA SERVED BY INTERNAL NETWORK SYSTEM

	Area (ha) Served			
	1975	1985	1990	2000
A. Area served by distribution mains	5	130	185	300
B. Area served by internal network system				
1. Existing	270	230	230	230
2. New System		290	415	610
3. Total	270	520	645	840
C. Total Service Area	275	650	830	1,140

Construction Period	Area (ha) of Internal Network Reinforcement	New Service Area
I. First Stage		
A. 1978-82	150	120
B. 1982-86	40	180
C. 1986-90	40	115
Sub-total	230	415
II. Second Stage		
A. 1990-95		95
B. 1995-2000		100
Sub-total		195
Grand Total	230	610

The cost of reinforcing the existing internal network system is listed in Item A of Table VIII-2. 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-2, Item B.

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 150 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-2. Provision is also made for upgrading 80 ha of residential fire service to high-value fire service. The schedule for fire hydrants installation is listed below:

Construction Period	Area (ha) Receiving Fire Protection Service: High-Value Area	Area (ha) Receiving Fire Protection Service: Residential Area
I. First Stage		
A. 1978-82	100	50
B. 1982-86	30	150
C. 1986-90	30	200
II. Second Stage		
A. 1990-95	40	380
B. 1995-2000	80 [1]	160

[1] Corresponds to upgrading residential fire service to high-value fire service.

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 below:

	Number of Service Connections 1975	1985	1990	2000
A. Present Service Area	746	4,846	6,127	8,083
B. 1990 Study Area		4,723	7,000	13,236
C. Year 2000 Study Area	___	___	917	2,873
Total	746	9,569	14,044	24,192
Rounded		9,600	14,000	24,200

Between 1975 and 1978 the number of connections is projected to increase from 746 to 1,000. The water supply for the new concessionaires can be obtained by an effective metering program to eliminate waste at flat-rate connections. This can be achieve by reducing leakage in distribution mains and by increase in supply when pumping heads are lowered at Wells No. 6 and 7. 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)	254	1,000
II. First Stage		
A. 1978-82	4,900	5,900
B. 1982-86	4,600	10,500
C. 1986-90	3,500	14,000
III. Second Stage		
A. 1990-95	5,000	19,000
B. 1995-2000	5,200	24,200

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 leakages 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	346

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 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 cost of the meter or ₱160.

2/ Connection cost includes ₱100 for pavement replacement.

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

Summary

The recommended improvement program for each component of the distribution system has been presented in this section. For each component, 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. The phased construction costs are summarised below:

Construction Period		Total Construction Cost (₱1,000)
I.	First Stage	
	A. 1978-82	5,924
	B. 1982-86	5,551
	C. 1986-90	4,240
	Total	15,715
II.	Second Stage	
	A. 1990-95	4,836
	B. 1995-2000	4,979
	Total	9,815

In all phases, the largest portion of the internal distribution system construction cost is the service connections. The service connections are 60 per cent of the construction cost in the first stage and 70 per cent in the second stage. Approximately two-thirds of the service connection costs will be borne by the concessionaire, thus, reducing the long-term capital cost requirements of the water district.

APPENDIX TABLE IX-C-3

SUMMARY OF DISTRIBUTION SYSTEM COSTS

onstruction Period	Item/Description	Quantity	Unit Cost	Total Consumption Cost (₱)	FEC (₱)
. First Stage					
A. 1978-82	Internal Network Reinforcement	150 ha	₱ 5,800/ha	870,000	375,000
	New Service Area	120 ha	₱10,200/ha	1,224,000	540,000
	Fire Hydrants High-High Value Area	100 ha	₱ 3,100/ha	310,000	180,000
	Residential Area	50 ha	₱ 770/ha	38,500	22,500
	Service Connections				
	Replacement	200	₱ 500	100,000	48,000
	New Connections	4,900	₱ 690	3,381,000	1,960,000
	Sub-Total (Rounded)			5,924,000	3,126,000
B. 1982-86	Internal Network Reinforcement	40 ha	₱ 5,800/ha	232,000	100,000
	New Service Area	180 ha	₱10,200/ha	1,836,000	810,000
	Fire Hydrants				
	High-Value Area	30 ha	₱ 3,100/ha	93,000	54,000
	Residential Area	150 ha	₱ 770/ha	115,500	67,500
	Service Connections				
	Replacement	200	₱ 500	100,000	48,000
	New Connections	4,600	₱ 690	3,174,000	1,840,000
	Sub-Total (Rounded)			5,551,000	2,920,000

APPENDIX TABLE IX-C-3 (Continued)

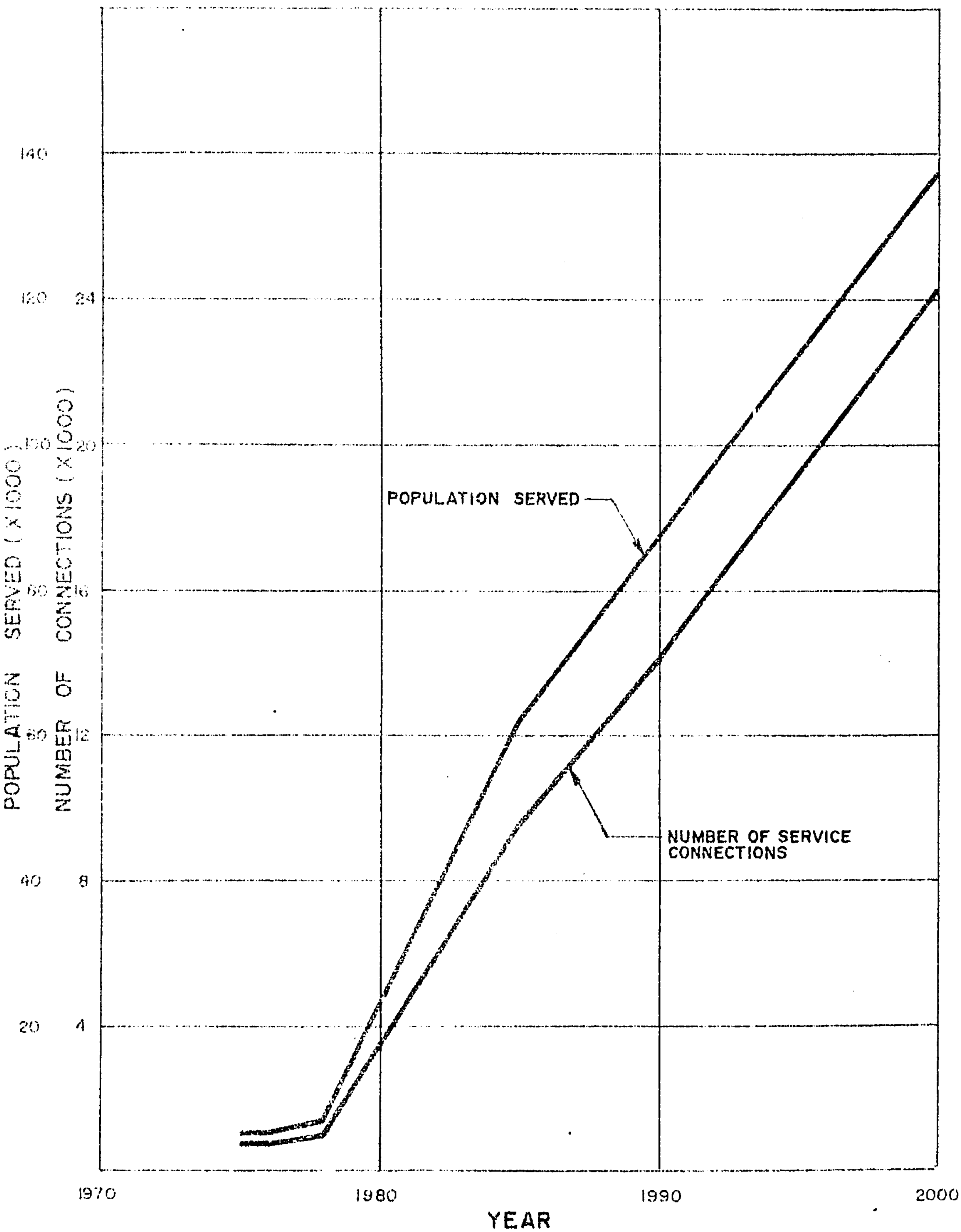
SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period	Item/Description	Quantity	Unit Cost	Total Consumption Cost (₱)	FEC (₱)
C. 1986-90	Internal Network				
	Reinforcement	40 ha	₱ 5,800/ha	232,000	100,000
	New Service Area	115 ha	₱10,200/ha	1,173,000	517,500
	Fire Hydrants				
	High-Value Area	30 ha	₱ 3,100/ha	93,000	54,000
	Residential Area	200 ha	₱ 770/ha	154,000	90,000
	Service Connections				
	Replacement	346	₱ 500	173,000	83,040
	New Connections	3,500	₱ 690	2,415,000	1,400,000
	Sub-total (Rounded)			4,240,000	2,245,000
	Grand Total (Rounded)			15,715,000	8,291,000
II. Second Stage					
A. 1990-95	Internal Network				
	New Service Area	95 ha	₱10,200/ha	969,000	427,500
	Fire Hydrants				
	High-Value Area	40 ha	₱ 3,100/ha	124,000	72,000
	Residential Area	380 ha	₱ 770/ha	292,000	171,000
	Service Connections				
	New Connections	5,000 ha	₱ 690	3,450,000	2,000,000
	Sub-Total (Rounded)			4,836,000	2,671,000

APPENDIX TABLE IX-C-3 (Continued)

SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period	Item/Description	Quantity	Unit Cost	Total Consumption Cost (₱)	FEC (₱)
B. 1995-2000	Internal Network				
	New Service Area	100 ha	₱10,200/ha	1,020,000	450,000
	Fire Hydrants				
	High-Value Area	80 ha	₱ 3,100/ha	248,000	144,000
	Residential Area	160 ha	₱ 770/ha	123,200	72,000
	Service Connections				
	New Connections	5,200 ha	₱ 690	3,588,000	2,080,000
	Sub-Total			4,979,000	2,746,000
	Grand Total			9,815,000	5,417,000



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

APPENDIX TABLE [illegible]

SYSTEM DATA

TARLAC WATER DISTRICT 2000 DISTRIBUTION SYSTEM AT PRESSURE ZONE 1 PK HR CONDI

INPUT AND OUTPUT IN	LPS
NO OF NODES	43
NO OF PIPES	47
MAX NO OF ITERATIONS	20
PEAKING FACTOR	1.50000
ALLOW P-DROP FR/STATIC - PCT	50.0
STATIC HGL FOR P-DROP CALC	70.0
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	0.50
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	82.57
BANDWIDTH	3

ITER 1 UNBAL 18.17 LPS
ITER 2 UNBAL 9.35 LPS
ITER 3 UNBAL 2.89 LPS
ITER 4 UNBAL 0.97 LPS
ITER 5 UNBAL 0.16 LPS
ITER 6 UNBAL 0.00 LPS

SOLUTION NO. 1 REACHED IN 6 ITERATIONS
0.0043 LPS UNBALANCE

APPENDIX TABLE IX-C-4 (Continued)

PIPE DATA

:PE IO	NODES FROM	-TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
1	1	2	200	1000.	110	0.123E-01	1.64	0.05	LO	0.03	0.03	LO
2	2	3	200	900.	110	0.111E-01	0.48	0.02	LO	0.00	0.00	LO
3	48	3	296	850.	110	0.156E-02	7.89	0.11	LO	0.07	0.08	LO
4	1	47	296	1080.	110	0.198E-02	7.43	0.11	LO	0.08	0.08	LO
5	5	1	250	320.	110	0.133E-02	46.57	0.95		1.64	5.11	
6	5	7	200	1140.	110	0.141E-01	5.08	0.16	LO	0.29	0.25	LO
7	8	7	200	1200.	110	0.148E-01	17.82	0.57		3.07	2.56	
8	8	2	261	1520.	110	0.513E-02	38.60	0.72		4.45	2.93	
9	9	8	200	1500.	110	0.185E-01	25.17	0.80		7.28	4.85	
10	7	10	150	1400.	100	0.837E-01	9.96	0.56		5.91	4.22	
11	6	5	250	920.	110	0.383E-02	32.69	0.67		2.44	2.65	
12	3	12	200	560.	110	0.692E-02	18.33	0.58		1.51	2.70	
13	12	11	200	600.	110	0.741E-02	5.54	0.18	LO	0.18	0.29	LO
14	13	12	200	1320.	110	0.163E-01	10.73	0.34	LO	1.32	1.00	
15	14	13	250	370.	110	0.154E-02	14.62	0.30	LO	0.22	0.60	
16	15	14	250	980.	110	0.408E-02	28.07	0.57		1.96	2.00	
17	16	14	200	1540.	110	0.190E-01	8.03	0.26	LO	0.90	0.58	
18	17	16	200	1440.	110	0.178E-01	25.41	0.81		7.11	4.94	
19	17	18	150	1300.	100	0.777E-01	2.11	0.12	LO	0.31	0.24	LO
20	19	17	250	1900.	110	0.791E-02	42.32	0.86		8.14	4.28	
21	15	11	150	1800.	110	0.902E-01	7.41	0.42		3.68	2.04	
22	29	9	200	1200.	110	0.148E-01	40.29	1.28		13.91	11.59	HI
23	29	30	150	1250.	100	0.748E-01	1.27	0.07	LO	0.12	0.09	LO
24	26	6	200	1350.	110	0.167E-01	35.19	1.12		12.18	9.02	
25	27	26	200	1100.	110	0.136E-01	39.09	1.24		12.05	10.96	HI
26	28	27	200	1500.	110	0.185E-01	42.20	1.34		18.94	12.63	HI
27	20	6	200	600.	110	0.741E-02	13.71	0.44		0.94	1.57	
28	21	20	200	1600.	110	0.198E-01	14.94	0.48		2.95	1.85	
29	21	24	150	1550.	100	0.927E-01	3.67	0.21	LO	1.03	0.67	
30	24	25	150	1150.	100	0.688E-01	1.56	0.09	LO	0.16	0.14	LO
31	22	21	200	1500.	110	0.185E-01	21.16	0.67		5.28	3.52	
32	22	23	200	1600.	110	0.198E-01	1.56	0.05	LO	0.05	0.03	LO
33	31	13	250	1400.	110	0.583E-02	16.91	0.34	LO	1.10	0.78	
51	45	4	500	1000.	120	0.121E-03	65.66	0.33	LO	0.28	0.28	LO
53	2	46	250	520.	110	0.217E-02	7.34	0.15	LO	0.09	0.17	LO
54	47	46	250	220.	110	0.916E-03	7.34	0.15	LO	0.04	0.17	LO
55	48	46	150	400.	100	0.239E-01	2.75	0.16	LO	0.16	0.39	LO
56	4	47	250	720.	110	0.300E-02	18.60	0.38	LO	0.67	0.93	
57	4	48	250	270.	110	0.112E-02	28.45	0.58		0.55	2.05	
.01	101	5	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
102	102	3	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
.03	103	8	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
.04	104	15	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
.07	107	19	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
.08	108	29	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
.09	109	28	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
110	110	22	250	50.	110	0.208E-03	24.00	0.49		0.07	1.50	

APPENDIX TABLE II-C-4 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM	CK	PCT DROP	CK
1	41.0	-37.50	67.13U	26.13	2.53		9.91	
2	40.0	-32.43	67.10U	27.10	2.62		9.68	
3	41.0	-34.03	67.09U	26.09	2.53		10.02	
4	40.0	-18.60	67.72U	27.72	2.68		7.60	
5	41.0	-25.03	68.76U	27.76	2.69		4.27	
6	42.0	-16.21	71.21U	29.21	2.83		-4.31	
7	40.0	-12.94	68.48U	28.48	2.76		5.08	
8	37.0	-12.75	71.55U	34.55	3.34		-4.69	
9	36.0	-15.12	78.83U	42.83	4.15		-25.96	
10	38.0	-9.96	62.57U	24.57	2.38		23.21	
11	40.0	-12.94	65.41U	25.41	2.46		15.31	
12	40.0	-23.52	65.58U	25.58	2.48		14.72	
13	43.0	-20.80	66.90U	23.90	2.31		11.47	
14	43.0	-21.48	67.12U	24.12	2.34		10.65	
15	41.0	-8.52	69.09U	28.09	2.72		3.15	
16	48.0	-17.38	68.03U	20.03	1.94		8.98	
17	49.0	-14.79	75.14U	26.14	2.53		-24.46	
18	40.0	-2.11	74.82U	34.82	3.37		-16.08	
19	46.0	-1.68	83.27U	37.27	3.61		-55.30	
20	40.0	-1.23	72.15U	32.15	3.11		-7.17	
21	33.0	-2.55	75.10U	42.10	4.08		-13.79	
22	31.0	-1.27	80.38U	49.38	4.78		-26.62	
23	30.0	-1.56	80.34U	50.34	4.87		-25.84	
24	30.0	-2.11	74.07U	44.07	4.27		-10.18	
25	25.0	-1.56	73.92U	48.92	4.74		-8.70	
26	33.0	-3.90	83.39U	50.39	4.88		-36.18	
27	30.0	-3.10	95.45U	65.45	6.34		-63.61	
28	25.0	-1.80	114.39U	89.39	8.65	HI	-98.64	
29	30.0	-2.43	92.74U	62.74	6.07		-56.84	
30	27.0	-1.27	92.62U	65.62	6.35		-52.61	
31	65.0	16.91U	68.00	3.00	0.29	LO	40.00	
45	55.0	65.66U	68.00	13.00	1.26		13.33	
46	42.0	-17.43	67.01U	25.01	2.42		10.68	
47	42.0	-18.69	67.05U	25.05	2.42		10.55	
48	44.0	-17.82	67.16U	23.16	2.24		10.91	
101	41.0	44.00	68.99U	27.99	2.71		3.47	
102	41.0	44.00	67.32U	26.32	2.55		9.23	
103	37.0	44.00	71.78U	34.78	3.37		-5.39	
104	41.0	44.00	69.32U	28.32	2.74		2.35	
107	46.0	44.00	83.50U	37.50	3.63		-56.26	
108	30.0	44.00	92.97U	62.97	6.10		-57.42	
109	25.0	44.00	114.62U	89.62	8.68	HI	-99.15	
110	31.0	24.00	80.46U	49.46	4.79		-26.81	

APPENDIX TABLE IX-C-5

SYSTEM DATA

```
:RLAC WATER DISTRICT 2000 DISTRIBUTION SYSTEM AT PRESSURE ZONE 1 FILLING COND

NPUT AND OUTPUT IN              LPS
) OF NODES                       43
) OF PIPES                       47
X NO OF ITERATIONS               20
:AKING FACTOR               0.30000
LOW P-DROP FR/STATIC - PCT   50.0
'ATIC HGL FOR P-DROP CALC    70.0
X UNBAL - LPS               0.10000
X ALLOW VEL -MPS              3.000
N ALLOW VEL - MPS             0.400
X ALLOW HL -  M/1000 M        10.00
N ALLOW HL -  M/1000 M         0.50
X ALLOW PRESS - ATM           7.000
N ALLOW PRESS - ATM           0.700
 OF HEADS TO BE READ              2
 OF UNKNOWN CONSUMPTIONS          2
M OF FIXED DEMANDS          -115.09
NDWIDTH                           3
ER  1 UNBAL   34.62 LPS
ER  2 UNBAL   18.98 LPS
ER  3 UNBAL    6.12 LPS
ER  4 UNBAL    0.28 LPS
ER  5 UNBAL    0.00 LPS

LUTION NO. 1 REACHED IN   5 ITERATIONS
     0.0028 LPS  UNBALANCE
```

APPENDIX TABLE IX-C-5 (Continued)

PIPE DATA

IPE NO	NODES FROM-TO		DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
1	1	2	200	1000.	110	0.123E-01	9.48	0.30	LO	0.80	0.80	
2	2	3	200	900.	110	0.111E-01	12.02	0.38	LO	1.11	1.24	
3	3	48	296	850.	110	0.156E-02	40.76	0.59		1.49	1.75	
4	1	47	296	1080.	110	0.198E-02	37.38	0.54		1.61	1.49	
5	5	1	250	320.	110	0.133E-02	54.36	1.11		2.18	6.81	
6	7	5	200	1140.	110	0.141E-01	0.43	0.01	LO	0.00	0.00	LO
7	8	7	200	1200.	110	0.148E-01	5.01	0.16	LO	0.29	0.24	LO
8	8	2	261	1520.	110	0.513E-02	32.68	0.61		3.27	2.15	
9	8	9	200	1500.	110	0.185E-01	3.76	0.12	LO	0.22	0.14	LO
10	7	10	150	1400.	100	0.837E-01	1.99	0.11	LO	0.30	0.21	LO
11	6	5	250	920.	110	0.383E-02	14.94	0.30	LO	0.57	0.62	
12	3	12	200	560.	110	0.692E-02	8.46	0.27	LO	0.36	0.64	
13	11	12	200	600.	110	0.741E-02	3.88	0.12	LO	0.09	0.15	LO
14	12	13	200	1320.	110	0.163E-01	7.63	0.24	LO	0.70	0.53	
15	14	13	250	370.	110	0.154E-02	24.34	0.50		0.57	1.54	
16	15	14	250	980.	110	0.408E-02	35.83	0.73		3.08	3.15	
17	14	16	200	1540.	110	0.190E-01	7.19	0.23	LO	0.73	0.48	LO
18	16	17	200	1440.	110	0.178E-01	3.72	0.12	LO	0.20	0.14	LO
19	17	18	150	1300.	100	0.777E-01	0.42	0.02	LO	0.02	0.01	LO
20	17	19	250	1900.	110	0.791E-02	0.34	0.01	LO	0.00	0.00	LO
21	15	11	150	1800.	110	0.902E-01	6.46	0.37	LO	2.86	1.59	
22	9	29	200	1200.	110	0.148E-01	0.74	0.02	LO	0.01	0.01	LO
23	29	30	150	1250.	100	0.748E-01	0.25	0.01	LO	0.01	0.00	LO
24	26	6	200	1350.	110	0.167E-01	20.24	0.64		4.37	3.24	
25	27	26	200	1100.	110	0.136E-01	21.02	0.67		3.82	3.47	
26	28	27	200	1500.	110	0.185E-01	21.64	0.69		5.50	3.67	
27	6	20	200	600.	110	0.741E-02	2.06	0.07	LO	0.03	0.05	LO
28	20	21	200	1600.	110	0.198E-01	1.81	0.06	LO	0.06	0.04	LO
29	21	24	150	1550.	100	0.927E-01	0.73	0.04	LO	0.05	0.03	LO
30	24	25	150	1150.	100	0.688E-01	0.31	0.02	LO	0.01	0.01	LO
31	21	22	200	1500.	110	0.185E-01	0.57	0.02	LO	0.01	0.00	LO
32	22	23	200	1600.	110	0.198E-01	0.31	0.01	LO	0.00	0.00	LO
33	13	31	250	1400.	120	0.496E-02	27.81	0.57		2.35	1.68	
51	4	45	500	1000.	120	0.121E-03	87.28	0.44		0.48	0.48	LO
53	2	46	250	520.	110	0.217E-02	23.64	0.48		0.76	1.46	
54	46	47	250	220.	110	0.916E-03	9.71	0.20	LO	0.06	0.28	LO
55	46	48	150	400.	100	0.239E-01	10.45	0.59		1.85	4.61	
56	47	4	250	720.	110	0.300E-02	43.35	0.88		3.22	4.48	
57	48	4	250	270.	110	0.112E-02	47.65	0.97		1.44	5.33	
101	101	5	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
102	102	3	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
103	103	8	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
104	104	15	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
107	107	19	250	50.	110	0.208E-03	0.0	0.0	LO	0.0	0.0	LO
108	108	29	250	50.	110	0.208E-03	0.0	0.0	LO	0.0	0.0	LO
109	109	28	250	50.	110	0.208E-03	22.00	0.45		0.06	1.28	
110	110	22	250	50.	110	0.208E-03	0.0	0.0	LO	0.0	0.0	LO

APPENDIX TABLE IX-C-5 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	--------PRESSURE-------- ATM---CK		PCT DROP---CK	
1	41.0	-7.50	71.81U	30.81	2.98		-6.26	
2	40.0	-6.49	71.02U	31.02	3.00		-3.40	
3	41.0	-6.81	69.91U	28.91	2.80		0.32	
4	40.0	-3.72	66.98U	26.98	2.61		10.08	
5	41.0	-5.01	73.99U	32.99	3.19		-13.77	
6	42.0	-3.24	74.57U	32.57	3.15		-16.30	
7	40.0	-2.59	74.00U	34.00	3.29		-13.32	
8	37.0	-2.55	74.29U	37.29	3.61		-12.99	
9	36.0	-3.02	74.07U	38.07	3.69		-11.98	
10	38.0	-1.99	73.70U	35.70	3.46		-11.55	
11	40.0	-2.59	69.64U	29.64	2.87		1.21	
12	40.0	-4.70	69.55U	29.55	2.86		1.51	
13	43.0	-4.16	68.85U	25.85	2.50		4.28	
14	43.0	-4.30	69.41U	26.41	2.56		2.17	
15	41.0	-1.70	72.50U	31.50	3.05		-8.61	
16	48.0	-3.48	68.68U	20.68	2.00		6.00	
17	49.0	-2.96	68.48U	19.48	1.89		7.25	
18	40.0	-0.42	68.46U	28.46	2.76		5.13	
19	46.0	-0.34	68.48U	22.48	2.18		6.35	
20	40.0	-0.25	74.54U	34.54	3.34		-15.12	
21	33.0	-0.51	74.48U	41.48	4.02		-12.10	
22	31.0	-0.25	74.47U	43.47	4.21		-11.46	
23	30.0	-0.31	74.47U	44.47	4.30		-11.17	
24	30.0	-0.42	74.43U	44.43	4.30		-11.06	
25	25.0	-0.31	74.42U	49.42	4.78		-9.82	
26	33.0	-0.78	78.94U	45.94	4.45		-24.16	
27	30.0	-0.62	82.76U	52.76	5.11		-31.90	
28	25.0	-0.36	88.26U	63.26	6.12		-40.58	
29	30.0	-0.49	74.06U	44.06	4.27		-10.16	
30	27.0	-0.25	74.06U	47.06	4.56		-9.44	
31	65.0	-27.81U	66.50	1.50	0.15	LO	70.00	HI
45	55.0	-87.78U	66.50	11.50	1.11		23.33	
46	42.0	-3.49	70.26U	28.26	2.74		-0.93	
47	42.0	-3.74	70.20U	28.20	2.73		-0.71	
48	44.0	-3.56	68.42U	24.42	2.36		6.09	
101	41.0	44.00	74.22U	33.22	3.22		-14.56	
102	41.0	44.00	70.14U	29.14	2.82		-0.47	
103	37.0	44.00	74.52U	37.52	3.63		-13.69	
104	41.0	44.00	72.73U	31.73	3.07		-9.41	
107	46.0	0.0	68.48U	22.48	2.18		6.35	
108	30.0	0.0	74.06U	44.06	4.27		-10.16	
109	25.0	22.00	88.32U	63.32	6.13		-40.72	
110	31.0	0.0	74.47U	43.47	4.21		-11.46	

APPENDIX TABLE IX-C-5

SYSTEM DATA

TARLAC WATER DISTRICT 2000 DISTRIBUTION SYSTEM AT PRESSURE ZONE 2 PKHR CONDITION

INPUT AND OUTPUT IN	LPS
NO OF NODES	17
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	90.0
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	0.50
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	21.87
BANDWIDTH	2

ITER 1 UNBAL 16.45 LPS
ITER 2 UNBAL 4.11 LPS
ITER 3 UNBAL 0.21 LPS
ITER 4 UNBAL 0.00 LPS

SOLUTION NO. 1 REACHED IN 4 ITERATIONS
0.0006 LPS UNBALANCE

APPENDIX TABLE IX-C-6 (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
34	33	32	200	1860.	110	0.230E-01	16.32	0.52		4.05	2.18	
35	33	34	200	970.	110	0.120E-01	18.42	0.59		2.64	2.72	
36	34	35	200	1100.	110	0.136E-01	11.65	0.37	LO	1.28	1.16	
37	34	37	150	1200.	100	0.718E-01	1.97	0.11	LO	0.25	0.21	LO
38	36	37	150	1200.	100	0.718E-01	0.73	0.04	LO	0.04	0.03	LO
39	35	32	200	750.	110	0.926E-02	4.11	0.13	LO	0.13	0.17	LO
40	36	35	200	1020.	110	0.126E-01	11.00	0.35	LO	1.07	1.05	
41	38	36	261	1450.	110	0.490E-02	25.99	0.49		2.04	1.41	
42	39	38	200	1920.	110	0.237E-01	19.70	0.63		5.92	3.08	
43	40	39	200	1120.	110	0.138E-01	21.83	0.69		4.17	3.73	
44	40	41	200	1440.	110	0.178E-01	12.13	0.39	LO	1.81	1.26	
45	42	41	200	1180.	110	0.146E-01	8.84	0.28	LO	0.82	0.70	
46	43	42	250	1160.	110	0.483E-02	10.82	0.22	LO	0.40	0.34	LO
47	44	38	200	100.	110	0.123E-02	21.87	0.70		0.37	3.74	
48	17	39	200	1050.	110	0.130E-01	13.98	0.44		1.71	1.63	
49	41	17	200	400.	110	0.494E-02	13.98	0.44		0.65	1.63	
105	105	33	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
106	106	40	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
111	111	43	250	50.	110	0.208E-03	12.00	0.24	LO	0.02	0.42	LO

APPENDIX TABLE IX-C-6 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	--------PRESSURE-------- ATM---CK	PCT DROP---CK
17	49.0	0.0	95.26U	46.26	4.48	-12.82
32	40.0	-20.43	84.39U	44.39	4.30	11.22
33	43.0	-9.25	88.44U	45.44	4.40	3.33
34	50.0	-4.80	85.80U	35.80	3.47	10.51
35	52.0	-18.54	84.52U	32.52	3.15	14.43
36	60.0	-14.26	85.58U	25.58	2.48	14.72
37	60.0	-2.70	85.55U	25.55	2.47	14.85
38	64.0	-15.58	87.63U	23.63	2.29	9.13
39	55.0	-16.11	93.54U	38.54	3.73	-10.12
40	57.0	-10.03	97.72U	40.72	3.94	-23.38
41	53.0	-6.99	95.91U	42.91	4.15	-15.97
42	50.0	-1.98	96.73U	46.73	4.52	-16.83
43	51.0	-1.18	97.13U	46.13	4.47	-18.28
44	70.0	21.87U	88.00	18.00	1.74	10.00
105	45.0	44.00	88.67U	43.67	4.23	2.96
106	57.0	44.00	97.95U	40.95	3.96	-24.08
111	51.0	12.00	97.15U	46.15	4.47	-18.33

APPENDIX TABLE IX-0-7

SYSTEM DATA

ARLAC WATER DISTRICT 2000 DISTRIBUTION SYSTEM AT PRESSURE ZONE 2 FILLING COND

```
NPUT AND OUTPUT IN              LPS
) OF NODES                       17
) OF PIPES                       19
AX NO OF ITERATIONS              20
EAKING FACTOR               0.30000
LLOW P-DROP FR/STATIC - PCT    50.0
TATIC HGL FOR P-DROP CALC      90.0
AX UNBAL - LPS              0.10000
AX ALLOW VEL -MPS             3.000
IN ALLOW VEL - MPS            0.400
AX ALLOW HL -  M/1000 M       10.00
IN ALLOW HL -  M/1000 M        0.50
AX ALLOW PRESS - ATM          7.000
IN ALLOW PRESS - ATM          0.700
) OF HEADS TO BE READ             1
) OF UNKNOWN CONSUMPTIONS         1
JM OF FIXED DEMANDS          -31.63
ANDWIDTH                          2
TER  1 UNBAL   16.26 LPS
TER  2 UNBAL    2.02 LPS
TER  3 UNBAL    0.14 LPS
TER  4 UNBAL    0.00 LPS

OLUTION NO. 1 REACHED IN   4 ITERATIONS
     0.0006 LPS UNBALANCE
```

APPENDIX TABLE IX-C-7 (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
34	33	32	200	1860.	110	0.230E-01	16.24	0.58		4.97	2.67	
35	33	34	200	970.	110	0.120E-01	23.90	0.76		4.28	4.41	
36	34	35	200	1100.	110	0.136E-01	14.61	0.47		1.95	1.77	
37	34	37	150	1200.	100	0.718E-01	8.33	0.47		3.64	3.03	
38	37	36	150	1200.	100	0.718E-01	7.79	0.44		3.22	2.68	
39	32	35	200	750.	110	0.926E-02	14.16	0.45		1.25	1.67	
40	35	36	200	1020.	110	0.126E-01	25.06	0.80		4.91	4.81	
41	36	38	261	1450.	110	0.490E-02	30.00	0.56		2.66	1.84	
42	39	38	200	1920.	110	0.237E-01	4.74	0.15	LO	0.42	0.22	LO
43	40	39	200	1120.	110	0.138E-01	2.66	0.08	LO	0.08	0.08	LO
44	41	40	200	1440.	110	0.178E-01	4.67	0.15	LO	0.31	0.21	LO
45	42	41	200	1180.	110	0.146E-01	11.37	0.36	LO	1.31	1.11	
46	43	42	250	1160.	110	0.483E-02	11.76	0.24	LO	0.46	0.40	LO
47	38	44	200	100.	110	0.123E-02	31.63	1.01		0.74	7.40	
48	17	39	200	1050.	110	0.130E-01	5.30	0.17	LO	0.28	0.27	LO
49	41	17	200	400.	110	0.494E-02	5.30	0.17	LO	0.11	0.27	LO
105	105	33	250	50.	110	0.208E-03	44.00	0.90		0.23	4.60	
106	106	40	250	50.	110	0.208E-03	0.0	0.0	LO	0.0	0.0	LO
111	111	43	250	50.	110	0.208E-03	12.00	0.24	LO	0.02	0.42	LO

APPENDIX TABLE IX-C-7 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM----CK	PRESSURE PCT DROP----CK
17	49.0	0.0	87.95U	38.95	3.77	5.01
32	40.0	-4.09	96.06U	56.06	5.43	-12.13
33	43.0	-1.85	101.04U	58.04	5.62	-23.48
34	50.0	-0.96	96.76U	46.76	4.53	-16.90
35	52.0	-3.71	94.81U	42.81	4.14	-12.66
36	60.0	-2.85	89.90U	29.90	2.89	0.32
37	60.0	-0.54	93.12U	33.12	3.21	-10.40
38	64.0	-3.12	87.24U	23.24	2.25	10.61
39	55.0	-3.22	87.66U	32.66	3.16	6.68
40	57.0	-2.01	87.75U	30.75	2.98	6.82
41	53.0	-1.40	88.06U	35.06	3.39	5.25
42	50.0	-0.40	89.37U	39.37	3.81	1.58
43	51.0	-0.24	89.83U	38.83	3.76	0.43
44	70.0	-31.63U	86.50	16.50	1.60	17.50
105	45.0	44.00	101.27U	56.27	5.45	-25.03
106	57.0	0.0	87.75U	30.75	2.98	6.82
111	51.0	12.00	89.85U	38.85	3.76	0.37

APPENDIX IX-H

MANAGEMENT OF GROUNDWATER RESOURCES

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

All TWD 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.

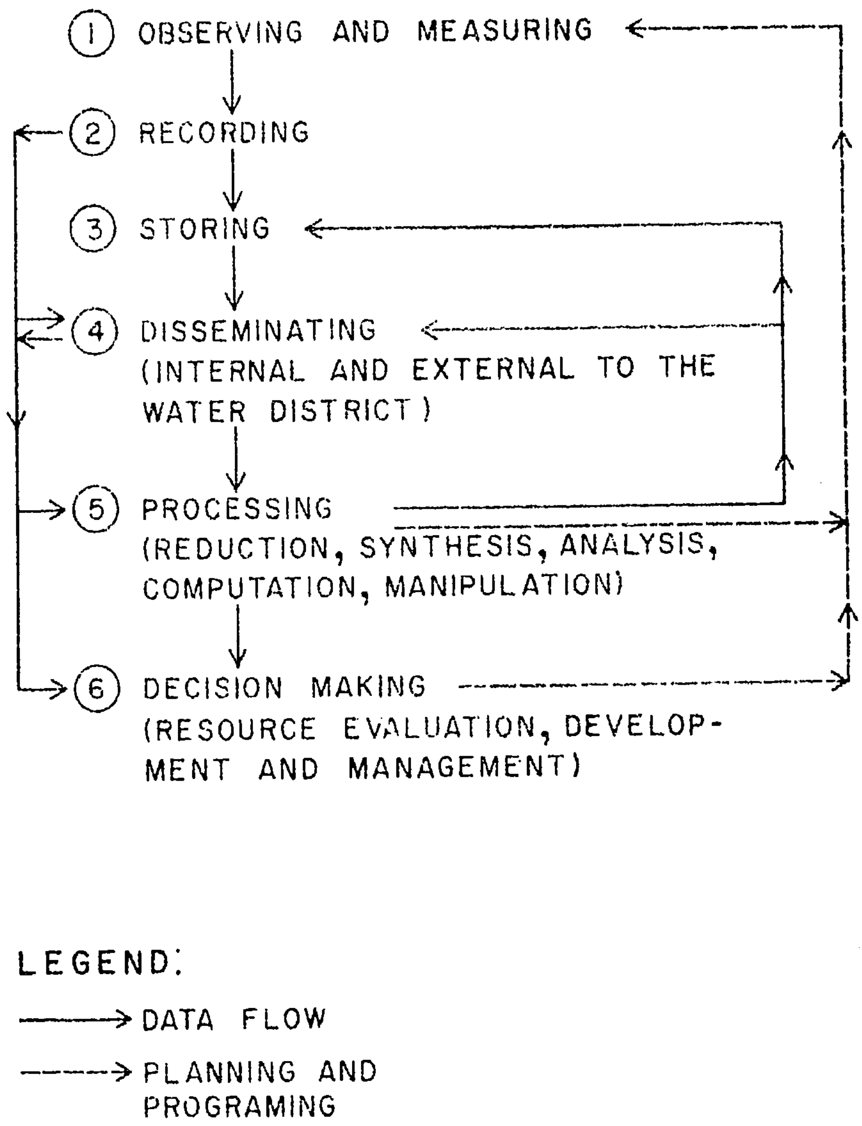
The management considerations with regard to wells should include a monitoring program of flow rates, pumping time, pumping water levels, static water levels and water quality. These records should be maintained on a daily basis. Water quality analyses, consisting of the parameters indicated in Chapter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

<u>Soil Erosion</u>

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

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area to its original state as soon as possible.

The erosion and dust problem can be minimized 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 gutters. In terms of being a burden to the existing surface drainage facilities, or causing flooding, wastewater is insignificant, by comparison with run-off from even a minor rainstorm, although 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 eutrophication 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 minimize 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.

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. 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 minimize 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 summarized 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 minimize 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 Z-B-1

PROJECT COST OF RECOMMENDED PROGRAM
TARLAC WATER DISTRICT
(WITHOUT ESCALATION)
P x 1000

Item	Service Life	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	Total
Source Development a) Structure	50	-	143	681	1303	651	-	68	581	1028	514	-	-	-	-	-	4969
b) Equipment	25	-	131	626	1198	598	-	63	540	955	477	-	-	-	-	-	4588
Storage Facilities a) Structure	50	-	222	1060	2029	1014	-	-	-	-	-	-	-	-	-	-	4325
b) Equipment	25	-	33	159	303	151	-	-	-	-	-	-	-	-	-	-	646
Distribution Mains	50	-	258	643	1179	1179	1309	1214	988	988	1050	791	470	470	470	235	11244
Internal Network	50	-	163	407	746	746	838	813	696	696	763	667	505	505	505	253	8303
Service Connections a) Meter	15	-	62	155	285	285	320	310	267	267	294	263	203	203	203	101	3218
b) Pipe	50	-	170	425	780	780	877	836	744	744	822	722	588	588	588	293	8957
Water District Buildings and Service Facilities	50	-	111	531	1016	508	-	-	-	-	-	-	-	-	-	-	2166
Early Action Works																	
a) Service Connections																	
(1) Meter	15	29	29	-	-	-	-	-	-	-	-	-	-	-	-	-	58
(2) Pipe	50	125	125	-	-	-	-	-	-	-	-	-	-	-	-	-	250
b) Vehicles	7	60	61	-	-	-	-	-	-	-	-	-	-	-	-	-	121
c) Miscellaneous System Improvements	50	175	176	-	-	-	-	-	-	-	-	-	-	-	-	-	351
d) Other Equipment	25	249	250	-	-	-	-	-	-	-	-	-	-	-	-	-	499
Sub-Total 1/		638	1934	4687	8839	5912	3344	3304	3816	4678	3920	2443	1766	1766	1766	882	49695
Land		297	-	-	-	-	-	-	-	-	-	-	-	-	-	-	297
TOTAL PROJECT COST 2/		935	1934	4687	8839	5912	3344	3304	3816	4678	3920	2443	1766	1766	1766	882	49992

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 Z-C-1.

APPENDIX TABLE X-B-2

PROJECT COST OF RECOMMENDED PROGRAM
TARLAC WATER DISTRICT
(WITH ESCALATION)
₱ x 1000

Item	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	Total
ESCALATION FACTOR	1.000	1.100	1.210	1.331	1.464	1.581	1.707	1.844	1.992	2.152	2.280	2.417	2.562	2.716	2.878	
Source Development a) Structure	-	157	824	1734	953	-	116	1071	2048	1106	-	-	-	-	-	8009
b) Equipment	-	144	757	1595	875	-	108	996	1902	1026	-	-	-	-	-	7403
Storage Facilities a) Structure	-	244	1283	2701	1484	-	-	-	-	-	-	-	-	-	-	5712
b) Equipment	-	36	192	403	221	-	-	-	-	-	-	-	-	-	-	852
Distribution Mains	-	284	778	1569	1726	2070	2072	1822	1968	2259	1803	1136	1204	1277	676	20644
Internal Network	-	179	492	993	1092	1325	1388	1283	1386	1641	1521	1221	1294	1372	728	15915
Service Connections a) Meters	-	68	188	379	417	506	529	492	532	632	600	491	520	551	291	6196
b) Pipes	-	187	514	1038	1142	1387	1427	1372	1482	1768	1646	1421	1506	1597	843	17330
Water District Buildings and Service Facilities	-	122	643	1352	744	-	-	-	-	-	-	-	-	-	-	2861
Early Action Works																
a) Service Connections																
(1) Meters	29	32	-	-	-	-	-	-	-	-	-	-	-	-	-	61
(2) Pipes	125	138	-	-	-	-	-	-	-	-	-	-	-	-	-	263
b) Vehicles	60	67	-	-	-	-	-	-	-	-	-	-	-	-	-	127
c) Miscellaneous System Improvements	175	193	-	-	-	-	-	-	-	-	-	-	-	-	-	368
d) Other Equipment	249	275	-	-	-	-	-	-	-	-	-	-	-	-	-	524
Sub-Total	638	2126	5671	11764	8654	5288	5640	7036	9318	8432	5570	4269	4524	4797	2538	86265
Land	297	-	-	-	-	-	-	-	-	-	-	-	-	-	-	297
TOTAL PROJECT COST	935	2126	5671	11764	8654	5288	5640	7036	9318	8432	5570	4269	4524	4797	2538	86562

APPENDIX TABLE [illegible]

ASSETS AND DEPRECIABLE VALUE FORECAST
TARLAC WATER DISTRICT
P x 1000

ITEM	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) Structure	-	157	981	2715	3668	-	116	1187	3235	4341	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Equipment	-	144	901	2496	3371	-	108	1104	3006	4032	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Storage Facilities a) Structure	-	244	1527	4228	5712	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Equipment	-	36	228	631	852	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Water District Building and Service Facilities	-	122	765	2117	2861	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total Work In Process	-	703	4402	12187	16464	-	224	2291	6241	8373	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
II. ASSETS ADDED BY YEAR END																									
Distribution Mains	-	284	778	1569	1726	2070	2072	1822	1968	2259	1803	1136	1204	1277	676	-	-	-	-	-	-	-	-	-	-
Internal Network	-	279	492	993	1092	1325	1388	1283	1386	1641	1521	1221	1294	1372	728	-	-	-	-	-	-	-	-	-	-
Service Connections a) Meters	-	68	188	379	417	506	529	492	532	632	600	491	520	551	291	-	-	-	-	-	-	-	-	-	-
b) Pipes	-	187	514	1038	1142	1387	1427	1372	1482	1768	1646	1421	1506	1597	843	-	-	-	-	-	-	-	-	-	-
Early Action Works a) Service Connections																									
1) Meters	29	32	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
2) Pipes	125	138	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
b) Vehicles	60	67	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
c) Miscellaneous System Improvements	175	193	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
d) Other Equipment	249	275	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
III. REPLACEMENT: a) Existing Facilities	-	-	2825	-	-	-	-	-	-	-	-	-	-	-	421	-	-	-	-	-	-	-	-	-	-
b) Meters	-	-	-	-	-	-	-	31	-	-	-	-	-	-	-	88	294	532	1032	1097	1306	1342	1304	1298	1514
c) Vehicles	-	-	-	-	-	-	-	111	121	-	-	-	-	-	173	186	-	-	-	-	-	260	280	-	-
Total Assets Added by Year End	638	1423	4797	3979	4377	5288	5416	5111	5489	6300	5570	4269	4524	4797	3132	274	294	532	1032	1097	1306	1602	1584	1298	1514
IV. DEPRECIABLE VALUES																									
a) 50 Years Service Life																									
Structures a) Source Development	-	-	-	-	-	3668	3668	3668	3668	3668	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009	8009
b) Storage Facilities	-	-	-	-	-	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712	5712
Existing Facilities	3276	3276	941	3766	3766	3766	3766	3766	3766	3766	3766	3766	3766	3766	3620	4041	4041	4041	4041	4041	4041	4041	4041	4041	4041
Distribution Mains	-	-	284	1062	2631	4357	6427	8427	10321	12289	14548	16351	17487	18691	19968	20644	20644	20644	20644	20644	20644	20644	20644	20644	20644
Internal Networks	-	-	179	671	1664	2756	4081	5469	6752	8138	9779	11300	12521	13815	15187	15915	15915	15915	15915	15915	15915	15915	15915	15915	15915
Service Connections (Pipes)	-	125	450	964	2002	3144	4531	5958	7330	8812	10580	12226	13647	15153	16750	17593	17593	17593	17593	17593	17593	17953	17593	17593	17593
Miscellaneous Improvements	-	175	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368	368
Water District Buildings and Service Facilities	-	-	-	-	-	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861	2861
Total 50 Years Service Life	3276	3576	2222	6831	10431	26632	31414	36301	40778	45614	55623	60593	64371	68375	72475	75143	75143	75143	75143	75143	75143	75143	75143	75163	75243
b) 25 Years Service Life																									
Equipment a) Source Development	-	-	-	-	-	3371	3371	3371	3371	3371	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403	7403
b) Storage Facilities	-	-	-	-	-	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852	852
Other Equipment	-	249	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524	524
Total 25 Years Service Life	-	249	524	524	524	4747	4747	4747	4747	4747	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779	8779
c) 15 Years Service Life																									
Existing and Replacement	17	17	17	17	17	17	17	-	31	31	31	31	31	31	31	31	119	413	945	1977	3074	4380	5722	7026	8324
Meter (Service Connection)	-	29	129	317	966	1113	1619	2148	2640	3172	3804	4404	4895	5415	5966	6228	6128	5940	5561	5144	4638	4109	3586	3054	2422
Total 15 Years Service Life	17	46	146	334	713	1130	1636	2148	[illegible]	3203	3835	4435	4926	5446	5997	6259	6247	6353	6506	7121	7712	8489	9308	10080	10746
d) 7 Years Service Life																									
Vehicles (Early Action and Development)	-	60	127	127	127	127	127	67	111	232	232	232	232	232	121	173	359	359	359	359	359	186	260	540	540
Total 7 Years Service Life	-	60	127	127	127	127	127	67	111	232	232	232	232	232	121	173	359	359	359	359	359	186	260	540	540
Total Depreciable Value	3293	3931	3019	7816	11795	32636	37924	43263	48307	53796	68469	74039	78308	82832	87372	90354	90528	90634	90787	91402	91993	92597	93499	94542	95208
Book Value of Assets Other Than Land	3931	6057	12218	23982	32636	37924	43564	50665	60037	68469	74039	78303	82832	87629	90504	90628	90822	91166	91819	92499	93299	94199	95074	95840	96722
Land	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297	297
Total Book Value of all Capital Assets	4228	6354	12515	24279	32933	38221	43861	50962	60334	68766	74336	78605	83129	87926	90801	90925	91119	91463	92116	92796	93596	94496	95371	96137	97019

APPENDIX TABLE X-E-2

SCHEDULE OF DEPRECIATION EXPENSES
TARLAC WATER DISTRICT
(₱ x 1000)

Year	Service Life Category				Total Annual Depreciation Expenses	Accumulated Depreciation Prior Year	Book Value of Assets Retired During the Year					Net Accumulated Depreciation Year End
	50 Years	25 Years	15 Years	7 Years			50 Years	25 Years	15 Years	7 Years	Total	
1976	66	-	1	-	67	2493	-	-	-	-	-	2560
1977	71	10	3	9	93	2560	-	-	-	-	-	2653
1978	44	21	10	18	93	2653	2335	-	-	-	2335	411
1979	137	21	22	18	198	411	-	-	-	-	-	609
1980	209	21	48	18	296	609	-	-	-	-	-	905
1981	533	190	75	18	816	905	-	-	-	-	-	1721
1982	628	190	109	18	945	1721	-	-	-	-	-	2666
1983	726	190	143	10	1069	2666	-	-	17	60	77	3658
1984	816	190	178	16	1200	3658	-	-	-	67	67	4791
1985	912	190	214	33	1349	4791	-	-	-	-	-	6140
1986	1112	351	256	33	1752	6140	-	-	-	-	-	7892
1987	1212	351	296	33	1892	7892	-	-	-	-	-	9784
1988	1287	351	328	33	1999	9784	-	-	-	-	-	11783
1989	1367	351	363	33	2114	11783	-	-	-	-	-	13897
1990	1450	351	400	17	2218	13897	146	-	-	111	257	15858
1991	1503	351	417	25	2296	15858	-	-	29	121	150	18004
1992	1503	351	416	51	2321	18004	-	-	100	-	100	20225
1993	1503	351	424	51	2329	20225	-	-	188	-	188	22366
1994	1503	351	434	51	2339	22366	-	-	379	-	379	24326
1995	1503	351	475	51	2380	24326	-	-	417	-	417	26289
1996	1503	351	514	51	2419	26289	-	-	506	-	506	28202
1997	1503	351	566	27	2447	28202	-	-	529	173	702	29947
1998	1503	351	621	37	2512	29947	-	-	523	186	709	31750
1999	1503	351	672	77	2603	31750	-	-	532	-	532	33821
2000	1503	351	716	77	2647	33821	-	-	632	-	632	35836

APPENDIX TABLE X-E-3

WORKING CAPITAL REQUIREMENTS
FOR REVOLVING FUND FOR NEW CONNECTIONS
TARLAC 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 3/	Increment Deducted 4/	Lump Sum Payments 5/ (Escalated)	Installment Payments (Cumulative)	Total Payments	Annual Construction Cost 6/	Working Capital Required	Cumulative Capital Requirement
1976	52	31	-	31	5.65	2	-	11	1	12	28	16	16
1977	52	31	-	62	6.21	2	-	12	3	15	31	16	32
1978	150	90	-	152	6.83	7	-	40	8	48	99	51	83
1979	1250	750	-	902	7.51	68	-	363	45	408	908	500	583
1980	1250	750	-	1652	8.26	74	-	399	116	515	997	482	1065
1981	1200	720	-	2372	8.92	77	-	414	192	606	1035	429	1494
1982	1200	720	-	3092	9.63	83	-	448	271	719	1120	401	1895
1983	1233	740	-	3832	10.40	92	-	498	359	857	1245	388	2283
1984	1233	740	-	4572	11.23	100	-	536	455	991	1339	348	2631
1985	1234	740	-	5312	12.13	108	-	580	559	1139	1449	310	2941
1986	900	540	16	5836	12.86	83	1	448	654	1102	1119	17	2958
1987	875	525	31	6330	13.63	86	2	463	736	1199	1157	(42)	2916
1988	875	525	60	6795	14.45	91	5	490	820	1310	1224	(86)	2830
1989	875	525	420	6900	15.31	96	37	518	876	1394	1295	(99)	2731
1990	875	525	750	6675	16.23	102	71	551	904	1455	1377	(78)	2653
1991	875	525	735	5940	17.20	102	76	551	879	879	1377	(879)	1774
1992	875	525	720	5220	18.23	102	80	551	799	799	1377	(799)	975
1993	875	525	730	4490	19.32	102	88	551	711	711	1377	(711)	264
1994	875	525	740	3750	20.48	102	96	551	615	615	1377	(615)	(351)
1995	875	525	740	3010	21.71	102	104	551	511	511	1377	(511)	(862)
1996	875	525	640	2370	23.01	102	96	551	415	415	1377	(415)	(1277)
1997	875	525	533	1837	24.39	102	84	551	331	331	1377	(331)	(1608)
1998	875	525	525	1312	25.85	102	88	551	243	243	1377	(243)	(1851)
1999	875	525	525	787	27.40	102	94	551	149	149	1377	(149)	(2000)
2000	875	525	525	262	29.04	102	99	551	50	50	1377	(50)	(2050)

3/ Accumulated installment payments are calculated on the basis of 100 per cent incremental additions during previous years and 50 per cent of the last year.

4/ Based on the assumption that installment plan will be paid back in 10 years.

5/ Assumed to be 40 per cent of construction cost.

6/ Amount to be shouldered by the customers, which is 2/3 of average cost of pipes plus meters.

APPENDIX T[illegible]LE [illegible]-4

REVENUE UNIT FORECAST
TARLAC WATER DISTRICT

Type of Connection By Meter Size	1976 Number of Connections 1/	1976 Proportion of Consumption 8/	1976 Estimated Consumption	1976 Use Factor	1976 Total EUs 9/	1980 Number of Connections	1980 Proportion of Consumption	1980 Estimated Consumption	1980 Use Factor	1980 Total EUs	1985 Number of Connections	1985 Estimated Consumption	1985 Total EUs	1990 Number of Connections	1990 Estimated Consumption	1990 Total EUs
1. Domestic/Government:																
1/2-inch	607	67	315	1	315	2788	67	1896	1	1896	7647	5561	5561	11152	8241	8241
3/4-inch	77	14	66	1	66	354	14	396	1	396	971	1162	1162	1416	1722	1722
1-inch	37	13	61	1	61	170	13	368	1	368	466	1079	1079	680	1599	1599
2-inches	3	6	28	1	28	13	6	170	1	170	36	498	498	52	738	738
Sub-Total	724	100%	470		470	3325	100%	2830		2830	9120	8300	8300	13300	1 300	12300
2. Commercial/Industrial:																
1/2-inch	14	38	101	2	202	110	32	342	2	684	302	544	1088	441	704	1408
3/4-inch	2	9	24	2	48	16	7	75	2	150	44	119	238	64	154	308
1-inch	6	53	140	2	280	47	44	471	2	942	129	748	1496	188	968	1936
2-inches	–	–	–	2	–	1	6	64	2	128	3	102	204	4	132	264
3-inches (wholesale)	–	–	–	3	–	1	11	118	3	354	2	187	561	3	242	726
Sub-Total	22	100%	265		530	175	100%	1070		2258	480	1700	3587	700	2200	4642
Total	746		735		1000	3500		3900		5088	9600	10000	11887	14000	14500	16942

1/ 1975 figures are actual; 1980, 1985 and 1990 are estimated with the proportion of connection in each size remaining constant.

8/ Proportion of consumption based on flow relationship.

9/ Includes both "Service EUs" and "Commodity EUs"; the effect of minimum monthly charges will be to increase total EUs 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
TARLAC WATER DISTRICT

Year	Rate/RU ₱	Estimated Number of RUs (Yearly in 000s)	₱ x 1000 Income From Sales	(Bad Debt)	Other Income 10/	Total Net Income
1976	1.00	365	365	7	7	365
1977	1.00	738	738	7	15	746
1978	1.00	1111	1111	11	22	1122
1979	1.95	1484	2894	29	29	2894
1980	1.95	1857	3621	36	72	3657
1981	1.95	2353	4588	46	92	4634
1982	2.00	2850	5700	114	114	5700
1983	2.00	3346	6692	67	134	6759
1984	2.00	3843	7686	77	154	7763
1985	2.30	4339	9980	200	200	9980
1986	2.30	4708	10828	108	217	10937
1987	2.30	5077	11677	117	234	11794
1988	2.40	5446	13070	261	261	13070
1989	2.40	5815	13956	140	279	14095
1990	2.40	6184	14842	148	297	14991
1991	2.60	6184	16078	322	322	16078
1992	2.60	6184	16078	161	322	16239
1993	2.60	6184	16078	161	322	16239
1994	3.00	6184	18552	371	371	18552
1995	3.00	6184	18552	186	371	18737
1996	3.00	6184	18552	186	371	18737
1997	3.30	6184	20407	408	408	20407
1998	3.30	6184	20407	204	408	20611
1999	3.30	6184	20407	204	408	20611
2000	3.30	6184	20407	204	408	20611

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
TARLAC 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	Amortized During Year	Outstanding Debt End of Year	Interest	Total Debt Service
1976	935	12	923	-	-	923	-	-
1977	2126	15	2111	923	-	3034	83	83
1978	5671	48	5623	3034	-	8657	273	273
1979	11764	408	11356	8657	-	20013	779	779
1980	8654	515	8139	20013	-	28152	1801	1801
1981	5288	606	4682	28152	-	32834	2534	2534
1982	5640	719	4921	32834	-	37755	2955	2955
1983	7036	857	6179	37755	283	43651	3398	3681
1984	9318	991	8327	43651	283	51695	3929	4212
1985	8432	1139	7293	51695	283	58705	4653	4936
1986	5570	1102	4468	58705	424	62749	5283	5707
1987	4269	1199	3070	62749	424	65395	5647	6071
1988	4524	1310	3214	65395	424	68185	5886	6310
1989	4797	1394	3403	68185	709	70879	6137	6846
1990	2538	1455	1083	70879	709	71253	6379	7088
1991	2538	1455	1083	71253	1270 11/	69983	6413	7683
1992	2538	1455	1083	69983	1270	68713	6298	7568
1993	2538	1455	1083	68713	1551	67162	6184	7735
1994	2538	1455	1083	67162	1689	65473	6045	7734
1995	2538	1455	1083	65473	1689	63784	5893	7582
1996	2538	1455	1083	63784	1689	62095	5741	7430
1997	2538	1455	1083	62095	2254	59841	5589	7843
1998	2538	1455	1083	59841	2254	57587	5386	7640
1999	2538	1455	1083	57587	2531	55056	5183	7714
2000	2538	1455	1083	55056	2531	52525	4955	7486

11/ Includes payment for second loan (1983-1990).

APPENDIX TABLE X-G-2

PROJECTED INCOME STATEMENT
TARLAC WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
Water Production (cumd)	1575	2120	2250	3970	6050	6050	9400	9400	9400	13300	13300	13300	13300
Water Sales (cumd)	735	1060	1240	2500	3900	3900	6600	6600	6600	10000	10000	10000	10000
Unaccounted-for-Water (%)	53	50	45	42	36	36	30	30	30	25	25	25	25
Connections: Metered	360	631	1000	2250	3500	4700	5900	7133	8366	9600	10500	11375	12250
Unmetered	438	219	–	–	–	–	–	–	–	–	–	–	–
Consumption (lpcd)	120.0	122.5	123.8	125.0	126.3	127.6	129.0	130.3	131.6	133.0	134.4	135.8	137.2
OPERATING REVENUE													
Water Sales	365	738	1111	2894	3621	4588	5700	6692	7686	9980	10828	11677	13070
Less: Uncollectibles	(7)	(7)	(11)	(29)	(36)	(46)	(114)	(67)	(77)	(200)	(108)	(117)	(261)
Other Revenue	7	15	22	29	72	92	114	134	154	200	217	234	261
Total Revenue	365	746	1122	2894	3657	4634	5700	6759	7763	9980	10937	11794	13070
OPERATING EXPENSES													
Administration and Personnel	110	149	171	258	427	461	498	591	638	690	745	804	1070
Source Facilities	105	119	187	264	354	476	614	747	898	1115	1291	1487	1707
Transmission and Distribution	12	13	16	29	67	87	108	135	148	202	242	277	320
Water Treatment Facilities	6	6	8	20	35	50	65	81	96	116	136	159	181
Miscellaneous	36	41	48	54	62	70	81	91	104	116	132	147	164
Depreciation	67	93	93	198	296	816	945	1069	1200	1349	1752	1892	1999
Total Operating Expenses	336	421	523	823	1241	1960	2311	2714	3084	3588	4298	4766	5441
Operating Income	29	325	599	2071	2416	2674	3389	4045	4679	6392	6639	7028	7629
Plus: Interest on Reserves	1	3	7	15	29	56	99	151	211	285	403	561	734
Net Income Before Interest	30	328	606	2086	2445	2730	3488	4196	4890	6677	7042	7589	8363
Interest On Debt	–	83	273	779	1801	2534	2955	3358	3929	4653	5283	5647	5886
Net Income (Loss)	30	245	333	1307	644	196	533	798	961	2024	1759	1942	2477
Cumulative Net Income (Loss)	30	275	608	1915	2559	2755	3288	4086	5047	7071	8830	10772	13249
Appropriation to Reserves	11	22	33	87	109	275	342	402	461	599	1083	1168	1307
Average Net Fixed Assets In Operation	1234	2333	5350	9593	13524	26032	38736	42992	47158	51778	60349	67633	70084
Rate of Return	2.4	13.9	11.2	21.6	17.9	10.3	8.7	9.4	9.9	12.3	11.0	10.4	10.9

	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Water Production (cumd)	13300	18600	18600	18600	18600	13600	18600	18600	18600	18600	18600	18600
Water Sales (cumd)	10000	14500	14500	14500	14500	14500	14500	14500	14500	14500	14500	14500
Unaccounted-for-Water (%)	25	22	22	22	22	22	22	22	22	22	22	22
Connections: Metered	13125	14000	14000	14000	14000	14000	14000	14000	14000	14000	14000	14000
Unmetered	–	–	–	–	–	–	–	–	–	–	–	–
Consumption (lpcd)	138.6	140	140	140	140	140	140	140	140	140	140	140
OPERATING REVENUE												
Water Sales	13956	14842	16078	16078	16078	18552	18552	18552	20407	20407	20407	20407
Less: Uncollectibles	(140)	(148)	(322)	(161)	(161)	(371)	(186)	(186)	(408)	(204)	(204)	(204)
Other Revenue	279	297	322	322	322	371	371	371	408	408	408	408
Total Revenue	14095	14991	16078	16239	16239	18552	18737	18737	20407	20611	20611	20611
OPERATING EXPENSES												
Administration and Personnel	1191	1286	1389	1500	1621	1750	1890	2041	2205	2381	2571	2777
Source Facilities	1952	2226	2404	2597	2805	3029	3271	3533	3816	4120	4450	4806
Transmission and Distribution	364	414	447	483	522	563	609	657	710	766	828	894
Water Treatment Facilities	209	241	260	281	303	328	354	382	413	446	481	520
Miscellaneous	185	205	222	240	259	280	302	326	352	380	411	444
Depreciation	2114	2218	2296	2321	2329	2339	2380	2419	2447	2512	2603	2647
Total Operating Expenses	6015	6590	7018	7422	7839	8289	8806	9358	9943	10605	11344	12068
Operating Income	8080	8401	9060	8817	8400	10263	9931	9379	10464	10006	9267	8523
Plus: Interest on Reserves	923	1125	1341	1566	1791	2033	2293	2553	2826	3112	3398	3684
Net Income Before Interest	9003	9525	10401	10383	10191	12296	12224	11932	13290	13118	12665	12207
Interest On Debt	6137	6379	6413	6298	6184	6045	5893	5741	5589	5386	5181	4955
Net Income (Loss)	2866	3147	3988	4085	4007	6251	6331	6191	7701	7732	7482	7252
Cumulative Net Income (Loss)	16115	19262	23250	27335	31342	37593	43924	50115	57816	65548	73030	80282
Appropriation to Reserves	1396	1484	1608	1608	1608	1855	1855	1855	2041	2041	2041	2041
Average Net Fixed Assets In Operation	72668	74486	73932	71908	69996	68444	67149	65951	64972	64085	62969	61750
Rate of Return	11.1	11.3	12.3	12.3	12.0	15.0	14.8	14.2	16.1	15.6	14.7	13.8

APPENDIX TABLE X-G-3

PROJECTED SOURCES AND APPLICATIONS OF FUNDS
TARLAC WATER DISTRICT
P x 1000

	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
SOURCES OF FUNDS													
Net Income Before Interest	30	328	606	2086	2445	2730	3488	4196	4890	6677	7042	7589	8363
Add: Depreciation	67	91	91	198	296	816	945	1069	1200	1349	1752	1892	1999
Total Internal Cash Generation	97	421	699	2284	2741	3546	4433	5265	6090	8026	8794	9481	10362
Long-Term Borrowing	923	2111	5623	11356	8139	4682	4921	6179	8327	7293	4468	3070	3214
Capital Contributions	12	15	48	408	515	606	719	857	991	1139	1102	1199	1310
Total External Cash Generation	935	2126	5671	11764	8654	5288	5640	7036	9318	8432	5570	4269	4524
Total Sources of Funds	1032	2547	6370	14048	11395	8834	10073	12301	15408	16458	14364	13750	14886
APPLICATIONS OF FUNDS													
Capital Expenditures	935	2126	5671	11764	8654	5288	5640	7036	9318	8432	5570	4269	4524
Debt Service: Interest	–	83	273	779	1801	2534	2955	3398	3929	4653	5282	5647	5886
Principal	–	–	–	–	–	–	–	283	283	283	424	424	424
Sub-Total	–	83	273	779	1801	2534	2955	3681	4212	4936	5707	6071	6310
Replacements	–	–	2825	–	–	–	–	142	121	–	–	–	–
Increase In Working Capital	98	89	233	665	185	253	242	276	251	543	135	132	262
Total Applications of Funds	1033	2298	9002	13208	10640	8075	8837	11135	13902	13911	11412	10472	11096
Increase (Decrease) In Cash Balance	(1)	249	(2632)	840	755	759	1236	1166	1506	2547	2952	3278	3790
Cash Balance Begin of Year	–	(1)	248	(2384)	(1544)	(789)	(30)	1206	2372	3878	6425	9377	12655
Cash Balance End of Year	(1)	248	(2384)	(1544)	(789)	(30)	1206	2372	3878	6425	9377	12655	16445
DEBT-SERVICE RATIO	–	5.07	2.56	2.93	1.52	1.40	1.50	1.43	1.45	1.63	1.54	1.56	1.64

	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
SOURCES OF FUNDS												
Net Income Before Interest	9003	9526	10401	10383	10191	12296	12224	11932	13290	13118	12665	12207
Add: Depreciation	2114	2218	2296	2321	2329	2329	2339	2419	2447	2512	2603	2647
Total Internal Cash Generation	11117	11744	12697	12704	12520	14635	14604	14351	15737	15630	15268	14854
Long-Term Borrowing	3403	1083	–	–	–	–	–	–	–	–	–	–
Capital Contributions	1394	1455	879	799	711	615	511	415	332	243	149	50
Total External Cash Generation	4797	2538	879	799	711	615	511	415	331	243	149	50
Total Sources of Funds	15914	14282	13576	13503	13231	15250	15115	14766	16068	15873	15417	14904
APPLICATIONS OF FUNDS												
Capital Expenditures	4797	2538	–	–	–	–	–	–	–	–	–	–
Debt Service: Interest	6137	6379	6413	6298	6184	6045	5893	5741	5589	5386	5183	4955
Principal	709	709	1270	1270	1551	1689	1689	1689	2254	2254	2531	2531
Sub-Total	6846	7088	7683	7568	7735	7734	7582	7430	7843	7640	7714	7486
Replacements	–	594	274	294	532	1032	1097	1306	1602	1584	1298	1514
Increase In Working Capital	226	(187)	(134)	53	18	667	(2)	(8)	339	(50)	(99)	(32)
Total Applications of Funds	11869	10033	7823	7915	8285	9433	8677	8728	9784	9174	8913	8968
Increase (Decrease) In Cash Balance	4045	4249	5753	5588	4946	5817	6438	6038	6284	6699	6504	5936
Cash Balance Begin of Year	16445	20490	24739	30492	36080	41026	46843	53281	59319	65603	72302	78806
Cash Balance End of Year	20490	24739	30492	36080	41026	46873	53281	59319	65603	72302	78806	84742
DEBT-SERVICE RATIO	1.62	1.66	1.65	1.68	1.62	1.89	1.93	1.93	2.01	2.05	1.98	1.98

APPENDIX TABLE X-G-4

PROJECTED BALANCE SHEET
TARLAC 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
ASSETS																									
Fixed Assets																									
Gross Value of Fixed Assets	4228	5651	8113	12092	16469	38221	43637	48671	54093	60393	74336	78605	83129	87926	90801	90925	91119	91463	92116	92796	93596	94496	95371	96137	97019
Less: Accumulated Depreciation	2560	2651	411	609	905	1721	2666	3658	4791	6140	7892	9784	11783	13897	15858	18004	20225	22166	24326	26289	28202	29947	31754	33821	35836
Net Value of Fixed Assets	1668	2998	7702	11483	15564	36500	40971	45013	49302	54253	66444	68821	71346	74029	74943	72921	70894	69097	67790	66507	65394	64549	63621	62316	62183
Work In Process	–	703	4402	12187	16464	–	224	2291	6241	8373	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
Total Fixed Assets	1668	3701	12104	23670	32028	36500	41195	47304	55543	66626	66444	68821	71346	74029	74943	72921	70894	69097	67790	66507	65394	64549	63621	62316	61183
Current Assets																									
Cash	(1)	248	(2384)	(1544)	(789)	(30)	1206	2372	3878	6425	9377	12655	16445	20490	24739	30492	36080	41026	46843	53281	59319	65603	72302	78806	84742
Accounts Receivable	91	185	278	724	905	1147	1425	1673	1922	2495	2702	2519	3268	3489	3711	4020	4020	4020	4638	4638	4638	5102	5102	5102	5102
Provision for Bad Debts	(2)	(2)	(3)	(14)	(9)	(11)	(29)	(17)	(19)	(50)	(27)	(29)	(65)	(35)	(37)	(80)	(40)	(40)	(93)	(46)	(45)	(102)	(51)	(51)	(51)
Inventories	54	59	217	479	532	578	597	659	703	763	714	691	735	786	458	116	192	278	454	484	562	585	584	593	678
Total Current Assets	142	490	(1892)	(355)	639	1684	3199	4687	6484	9633	12771	16236	20383	24730	28871	34548	40252	45284	51842	58357	64473	71185	77937	84450	90471
Total Assets	1810	4191	10212	23315	32667	38184	44394	51991	62027	76259	79215	85057	91729	98759	103814	107469	111146	114381	119632	124846	129867	135737	141558	146766	151654
EQUITY AND LIABILITIES																									
Current Liabilities																									
Accounts Payable	45	55	72	104	158	191	228	274	314	373	424	479	574	650	729	787	850	918	992	1071	1157	1249	1349	1457	1574
Current Maturities of Long-Term Debt	–	–	–	–	–	–	283	283	283	424	424	424	709	709	1270	1270	1551	1689	1689	1689	2254	2254	2531	2531	2531
Total Current Liabilities	45	55	72	104	158	191	511	557	597	797	848	903	1283	1359	1999	2057	2401	2607	2681	2760	3411	3503	3880	3988	4105
Long-Term Debt (Less: Current Maturities)	923	3034	8657	20013	28152	32834	37472	43368	51412	58281	62325	64971	67476	70170	69983	68713	67162	65473	63784	62095	59841	57587	55056	52525	49994
Equity																									
Government Contribution	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	800	700	700	800	800	800
Capital Contribution	12	27	75	483	998	1604	2323	3180	4171	5310	6412	7611	8921	10315	11770	12649	13448	14159	14774	15285	15700	16031	16274	16423	16473
Reserves	11	33	66	153	262	537	879	1281	1742	2341	3424	4592	5899	7295	8779	10387	11995	13603	15458	17313	19168	21209	23250	25291	27332
Unappropriated Retained Earnings	19	242	542	1762	2297	2218	2409	2805	3305	4730	5406	6180	7350	8820	10483	12863	15340	17739	22135	26611	30947	36607	42298	47739	52950
Total Equity	842	1102	1483	3198	4357	5159	6411	8066	10018	13181	16042	19183	22970	27230	31832	36699	41583	46301	53167	60009	66615	74647	82622	90253	97555
Total Equity and Liabilities	1810	4191	10212	23315	32667	38184	44394	51991	62027	72259	79215	85057	91729	98759	103814	107469	111146	114381	119632	124864	129867	135737	141558	146766	151654

APPENDIX TABLE [illegible]

RATE OF RETURN ON TOTAL INVESTMENT
(DISCOUNTED CASH FLOW METHOD)
TARLAC WATER DISTRICT
₱ x 1000

Year	Debt Service	Net Increase In Cash	Total Cash Inflow	Investments	Net Cash Inflow	First Trial Present Value: 10% Factor	First Trial Present Value: 10% Value	Second Trial Present Value: 12% Factor	Second Trial Present Value: 12% Value
1976	-	(1)	(1)	935	(936)	1.000	(936)	1.000	(936)
1977	83	249	332	2126	(1794)	.909	(1631)	.893	(1602)
1978	273	(2632)	(2359)	8496	(10855)	.826	(8966)	.797	(8651)
1979	779	840	1619	11764	(10145)	.751	(7619)	.712	(7223)
1980	1801	755	2556	8654	(6098)	.683	(4165)	.636	(3878)
1981	2534	759	3293	5288	(1995)	.621	(1239)	.567	(1131)
1982	2955	1236	4191	5640	(1449	.564	(817)	.507	(735)
1983	3681	1166	4847	7178	(2331)	.513	(1196)	.452	(1054)
1984	4212	1506	5718	9439	(3721)	.467	(1738)	.404	(1503)
1985	4936	2547	7483	8432	(949)	.424	(402)	.361	(343)
1986	5707	2952	[illegible]	5570	3089	.386	1192	.322	995
1987	6071	3278	9349	4269	5080	.350	1778	.287	1458
1988	6310	3790	10100	4524	5576	.319	1779	.257	1433
1989	6846	4045	10891	4797	6094	.290	1767	.229	1396
1990	7088	4249	11337	3132	8205	.263	2158	.205	1682
1991	7683	5753	13436	274	13162	.239	3146	.183	2409
1992	7568	5588	13156	294	12862	.218	2804	.163	2097
1993	7735	4946	12681	532	12149	.198	2406	.146	1774
1994	7734	5817	13551	1032	12519	.180	2253	.130	1627
1995	7582	6438	14020	1097	12923	.164	2119	.116	1499
1996	7430	6038	13468	1306	12162	.149	1812	.104	1265
1997	7843	6284	14127	1602	12525	.135	1691	.093	1165
1998	7640	6699	14339	1584	12755	.123	1569	.083	1059
1999	7714	6504	14218	1298	12920	.112	1447	.074	956
2000	7486	5936	13422	1514	24721 12/	.102	2522	.066	1632
							+1734		-4609

Rate of Return = 10.5%

12/ Includes Net Asset Value of ₱12813

Total Assets	= ₱151654
Total Liabilities	(54099)
Cash	(84742)
Net Asset Value	₱ 12813

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 40 ha from 1978 to 1982, by 56.2 ha from 1982 to 1986 and by 42.5 ha from 1986 to 2000.

2. The land use distribution of 36 per cent commercial, industrial and institutional; and 64 per cent residential, was used from 1979 to 1982. From 1982 to 1986, the land use was assumed to be 17 per cent commercial, industrial and institutional, and 83 per cent residential. From 1986 to 2000, land was classified into 15 per cent commercial, industrial and institutional and 85 per cent residential. This classification was based on the water demand projections in 1985 and 1990 by consumer category, as shown in Table VI-7, Chapter VI.

3. The 1975 costs of land are:

Residential	:	₱ 80 per sqm
Commercial/Industrial/Institutional	:	₱170 " "

   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
TARLAC WATER DISTRICT

Year	Land Use (sqm) Com./Ind./Res.	Land Use (sqm) Residential	Cost of Land in M ₱ Com./Ind./Ins.	Cost of Land in M ₱ Residential	Cost of Served Land	20% Benefit Due to Project	Discount Factor *	PV** of Benefit in M ₱
1979	144,000	256,000	20.160	10.240	30.400	6.080	0.712	4.329
1980	144,000	256,000	20.160	10.240	30.400	6.080	0.636	3.867
1981	144,000	256,000	20.160	10.240	30.400	6.080	0.567	3.447
1982	144,000	256,000	20.160	10.240	30.400	6.080	0.507	3.083
1983	95,540	466,460	13.376	18.658	32.034	6.407	0.452	2.896
1984	95,540	466,460	13.376	18.658	32.034	6.407	0.404	2.588
1985	95,540	466,460	13.376	18.658	32.034	6.407	0.361	2.313
1986	95,540	466,460	13.376	18.658	32.034	6.407	0.322	2.063
1987	63,750	361,250	8.925	14.450	23.375	4.675	0.287	1.342
1988	63,750	361,250	8.925	14.450	23.375	4.675	0.257	1.201
1989	63,750	361,250	8.925	14.450	23.375	4.675	0.229	1.071
1990	63,750	361,250	8.925	14.450	23.375	4.675	0.205	0.958
1991	63,750	361,250	8.925	14.450	23.375	4.675	0.183	0.856
1992							0.163	0.762
1993							0.147	0.687
1994							0.130	0.608
1995							0.116	0.542
1996							0.104	0.486
1997							0.093	0.435
1998							0.083	0.388
1999							0.074	0.346
2000	63,750	361,250	8.925	14.450	23.375	4.675	0.066	0.309
								₱34.577

*Discounted at 12 per cent
**PV = Present Value or Present Worth.

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 1,108 out of every 100,000 population in Tarlac province were afflicted with primary water-borne diseases every year, regardless of age, sex and income class. The same rate was used for the municipality of Tarlac in the absence of specific data. 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, 30 per cent of the city's population was economically active.[1/] It was assumed, therefore, that only 30 per cent of 1,108 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, the 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 30 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 30 per cent and then by ₱11,629. The projected

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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.

number of such deaths was based on the average of the 11-year mortality rate for primary water-borne diseases in Tarlac province, as gathered from the Department of Health. These figures indicated that 52 persons died of the 1,108 per 100,000 who were afflicted with water-borne diseases. This mortality rate was assumed to be constant over the projection period. The 30 per cent corresponds to the portion of the study area population who are income-earners. The ₱11,629, on the other hand, is 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 doctor's 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 the municipality of Tarlac. The total present value of said benefits after the adjustments amounts to ₱1.2 million.

2/ Refer to Methodology Manual, Chapter 20 for "Ability to Pay" studies.

APPENDIX TABLE XI-C-2

HEALTH BENEFITS
TARLAC WATER DISTRICT

Year	Study Area Population	Cost of Time Lost Due to Illness	Eco. Loss Due to Premature Death	Cost of Medical Expenses	Total	40% Reduction Due to Project	Discount Factor *	Present Value
1978	111,510	₱44,479	₱202,293	₱139,615	₱386,387	₱154,555	0.797	₱123,180
1979	114,315	45,598	207,382	143,127	396,107	158,443	0.712	112,811
1980	117,195	46,747	212,606	146,733	406,086	162,434	0.636	103,308
1981	130,145	47,923	217,958	150,426	416,307	166,523	0.567	94,419
1982	123,165	49,128	223,437	154,208	426,773	170,709	0.507	86,549
1983	126,265	50,365	229,060	158,089	437,514	175,006	0.452	79,103
1984	129,445	51,633	234,829	162,070	448,532	179,413	0.404	72,483
1985	132,700	52,931	240,734	166,146	459,811	183,924	0.361	66,397
1986	135,975	54,238	246,676	170,246	471,160	188,464	0.322	60,685
1987	139,330	55,576	252,762	174,447	482,785	193,114	0.288	55,617
1988	142,765	56,946	258,993	178,747	494,686	197,874	0.257	50,854
1989	146,290	58,352	265,388	183,161	506,901	202,760	0.229	46,432
1990	149,900	59,792	271,937	187,681	519,410	207,764	0.205	42,592
1991	149,900	59,792	271,937	187,681	519,410	207,764	0.183	38,021
1992							0.163	33,865
1993							0.145	30,334
1994							0.130	27,009
1995							0.116	24,101
1996							0.104	21,607
1997							0.093	19,322
1998							0.083	17,244
1999							0.074	15,375
2000	149,900	59,792	271,937	187,681	519,410	207,764	0.066	13,712
								₱1,235,021

*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 21,297 dwelling units in the municipality of Tarlac of which 14,536 units were made of concrete and galvanized iron.

2. Since not all of the 21,297 units are within the service area which will be provided with fire hydrants, it was assumed that 90 per cent of the 14,536 units made of concrete and galvanized iron (equivalent to 13,082 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 municipality where there is usually a heavy concentration of dwelling units.

3. These 13,082 dwelling units are all made of concrete and galvanized iron and may be considered insurable. However, only 20 per cent (2,616 units) were assumed to be actually insured. These were classified into 36 per cent commercial, industrial and institutional and 64 per cent residential from 1978 to 1985. From 1985 to 1989, they were classified into 17 per cent commercial, industrial and 83 per cent residential. Thereafter up to 2000, they were classified into 15 per cent commercial, industrial and institutional and 85 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-7, 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 2.76 per cent from 1970 to 1975, by 2.52 per cent from 1975 to 1985 and by 2.47 per cent from 1985 to 2000. These are the rates by which the number of households in TWD service area has 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 number 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.

APPENDIX TABLE XI-C-3

REDUCTION IN FIRE INSURANCE COST
TARLAC WATER DISTRICT

Year	Total Insured Units	No. of Insured Units		Cost of Insurance		Level of Insurance Cost	20% Reduction Due to Project (Benefit)	Discount Factor *	PV of Benefit (1976 Prices)
		Com./Inst./Ind.	Residential	Com./Inst./Ind.	Residential				
1979	3,311	1,192	2,119	₱1,490,000	₱ 894,748	₱2,384,748	₱476,950	0.712	₱339,588
1980	3,395	1,222	2,173	1,527,500	917,549	2,445,049	489,010	0.636	311,010
1981	3,480	1,253	2,227	1,566,250	940,351	2,506,601	501,320	0.567	284,248
1982	3,568	1,284	2,284	1,605,000	964,419	2,569,419	513,884	0.507	260,539
1983	3,658	1,317	2,341	1,646,250	988,487	2,634,737	526,947	0.452	238,180
1984	3,750	1,350	2,400	1,687,500	1,013,400	2,700,900	540,180	0.404	218,233
1985	3,845	654	3,191	817,500	1,347,400	2,164,900	432,980	0.361	156,306
1986	3,940	670	3,270	837,500	1,380,758	2,218,258	443,652	0.322	142,856
1987	4,037	686	3,351	857,500	1,414,960	2,272,460	454,490	0.287	130,439
1988	4,137	703	3,434	878,750	1,450,006	2,328,756	465,751	0.257	119,698
1989	4,239	721	3,518	901,250	1,485,476	2,386,725	477,345	0.229	109,312
1990	4,344	652	3,692	815,000	1,558,947	2,373,947	474,789	0.205	97,332
1991	4,344	652	3,692	815,000	1,558,947	2,373,947	474,789	0.183	86,886
1992								0.163	77,391
1993								0.146	69,319
1994								0.130	61,723
1995								0.116	55,076
1996								0.104	49,378
1997								0.093	44,155
1998								0.083	39,407
1999								0.074	35,134
2000	4,344	652	3,692	815,000	1,558,947	2,373,947	474,789	0.066	31,336
								Total =	₱2,957,546

*Discounted at 12 per cent

Reduction in Fire Damage

Based on the records of the municipality's fire department, from 1973 to 1975, the average annual loss due to fire in the area was estimated to be ₱565,000. 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 37.5 ha of high-value and residential areas in Tarlac, or a total of 150 ha during the four-year period. From 1982 to 1986, another 45 ha every year in both high-value and residential areas will be extended fire protection. From 1986 to 1990, additional 57.5 ha every year will be covered. Hence, by the end of Stage I Construction, a total of 560 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, 660 additional hectares will be extended fire protection. However, this area was 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 (560 ha) was maintained up to 2000 for purposes of this study.

According to the records of Tarlac's fire department, the average annual fire damage from 1973 to 1975 in the municipality is ₱565,000. Since ₱565,000 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:

$$\frac{\text{No. of hectares with installed fire hydrants}}{\text{No. of hectares in study area}} \times ₱565{,}000$$

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 hydrants and rehabilitated old fire hydrants with adequate water pressure and in sufficient quantity will go a long way in controlling fires. The existing 47 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 TWD. In the first approach where 1976 prices were used and then discounted, reduction in fire damage amounts to ₱161,917. In the second approach where inflation was considered, the same benefit amounts to ₱400,013.

APPENDIX TABLE XI-C-4

REDUCTION IN FIRE DAMAGE
TARLAC WATER DISTRICT

Year	Annual Fire Damage	75% Reduction due to Project (First Approach)	75% Reduction due to Project (Second Approach)	Discount Factor*	PV of Benefit (First Approach)	PV of Benefit (Second Approach)
1979	₱ 4,726	₱ 3,545	₱ 4,718	0.712	₱ 2,524	₱ 3,359
1980	9,452	7,089	10,378	0.636	4,509	6,600
1981	14,179	10,634	16,812	0.567	6,030	9,532
1982	18,905	14,179	24,218	0.507	7,189	12,278
1983	24,576	18,432	33,989	0.452	8,331	15,363
1984	30,248	22,686	45,910	0.404	9,165	18,548
1985	35,919	26,939	57,946	0.361	9,725	20,918
1986	41,590	31,192	71,118	0.322	10,044	22,900
1987	48,837	36,628	88,530	0.287	10,512	25,408
1988	56,084	42,063	107,765	0.257	10,810	27,696
1989	63,331	47,498	129,005	0.229	10,877	29,542
1990	70,578	52,934	152,397	0.205	10,851	31,241
1991	70,578	52,934	152,397	0.183	9,687	27,889
1992				0.163	8,628	24,841
1993				0.147	7,781	22,402
1994				0.130	6,881	19,812
1995				0.116	6,140	17,678
1996				0.104	5,505	15,849
1997				0.093	4,923	14,173
1998				0.083	4,394	12,649
1999				0.074	3,917	11,277
2000	70,578	52,934	152,397	0.066	3,494	10,058
			T O T A L		₱ 161,917	₱ 400,013

*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 TWD are paying an average of ₱0.39 per cubic meter, with a present aggregate consumption of 270,000 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 TWD from 1977 to 2000 was obtained from Table VI-7, 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 270,000 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 in 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

INCR[illegible]TAL REVENUE BENEFIT

[illegible]ST APPROACH

TARL[illegible] WATER DISTRICT

Year	Projected Delivered Water (cum x 1,000)	Incremental Delivered Water* (cum x 1,000)	Price per Cum Domestic (cum x 1,000)	Price per Cum Com/Ind (cum x 1,000)	Pr[illegible] per Cum Dom[illegible]ic	Pr[illegible] per Cum Com/Ind	Revenues From Domestic (P x 1,000)	Revenues From Com/Ind (P x 1,000)	Total Revenues (P x 1,000)	Discount Factor**	PV of Total Revenues (P x 1,000)
1976	270	–	–	–	1.00	2.00	–	–	–	1.000	–
1977	387	117	80.0	37.0	.91	1.82	72.8	67.3	140.1	0.893	125.1
1978	453	183	124.0	59.0	.83	1.65	102.9	97.9	200.8	0.797	160.0
1979	913	643	488.7	154.3	1.47	2.94	718.4	453.6	1,172.0	0.712	834.5
1980	1,424	1,154	837.3	316.6	1.33	2.66	1,113.6	842.2	1,955.8	0.636	1,243.9
1981	1,953	1,683	1,248.1	434.9	1.23	2.46	1,535.2	1,069.9	2,605.1	0.567	1,477.1
1982	2,409	2,139	1,620.5	518.5	1.17	2.34	1,896.0	1,213.3	3,109.3	0.507	1,576.4
1983	2,847	2,577	2,014.4	562.6	1.08	2.16	2,175.6	1,215.2	3,390.8	0.452	1,532.6
1984	3,267	2,997	2,415.3	581.7	1.00	2.00	2,415.3	1,163.4	3,578.7	0.404	1,445.8
1985	3,650	3,380	2,805.4	574.6	1.07	2.14	3,001.8	1,229.5	4,231.4	0.361	1,527.5
1986	3,997	3,727	3,106.8	620.2	1.01	2.02	3,137.9	1,252.8	4,390.7	0.322	1,413.8
1987	4,307	4,037	3,379.8	657.2	.95	1.90	3,210.8	1,248.7	4,459.5	0.288	1,248.3
1988	4,636	4,366	3,670.9	695.1	.94	1.88	3,450.6	1,306.8	4,757.4	0.257	1,222.7
1989	4,982	4,712	3,978.8	733.2	.88	1.76	3,501.3	1,290.4	4,791.8	0.229	1,097.3
1990	5,293	5,023	4,259.5	763.5	.83	1.66	3,535.4	1,267.4	4,802.8	0.205	984.6
1991										0.183	878.9
1992										0.163	782.9
1993										0.146	701.2
1994										0.130	624.4
1995										0.116	557.1
1996										0.104	499.3
1997										0.093	446.7
1998										0.083	398.6
1999										0.074	355.4
2000	5,293	5,023	4,259.5	763.5	.83	1.66	3,535.4	1,267.4	4,802.8	0.066	317.0
						Total	65,221.6	26,392.5	91,614.2		21,451.3

*The present volume of delivered water amounting to 270,000 cum was deducted from the projected delivered water throughout the projection period to obtain the annual incremental volume.

**Discounted at 12 per cent.

APPENDIX TABLE XI-C-6

INCREMENTAL REVENUE BENEFIT
SECOND APPROACH
TARLAC WATER DISTRICT

Year	Projected Delivered Water (cum x 1,000)	Domestic (cum x 1,000)	Com/Ind (cum x 1,000)	Eco. Value per Cum: Domestic	Eco. Value per Cum: Com/Ind	Economic Revenues: Domestic (P x 1,000)	Economic Revenues: Com/Ind (P x 1,000)	Total Eco. Revenues (P x 1,000)	Net Revenue (Benefit)* (P x 1,000)	Discount Factor**	PV of Net Revenue (P x 1,000)
1976	270	181	89	P 1.50	P 2.40	271.5	213.6	485.1	372.7	1.000	372.7
1977	387	265	122	1.50	2.40	397.5	292.8	690.3	577.9	0.893	516.1
1978	453	307	146	1.50	2.40	460.5	350.4	810.9	698.5	0.797	556.7
1979	913	694	219	2.93	4.68	2,033.4	1,024.9	3,058.3	2,945.9	0.712	2,097.5
1980	1,424	1,033	391	2.93	4.68	3,026.7	1,829.9	4,856.6	4,744.2	0.636	3,017.3
1981	1,953	1,448	505	2.93	4.68	4,242.6	2,363.4	6,606.0	6,493.6	0.567	3,681.9
1982	2,409	1,825	584	3.00	4.80	5,475.0	2,803.2	8,278.2	8,165.8	0.507	4,140.1
1983	2,847	2,225	622	3.00	4.80	6,675.0	2,985.6	9,660.6	9,548.2	0.452	4,315.8
1984	3,267	2,633	634	3.00	4.80	7,899.0	3,043.2	10,942.2	10,829.8	0.404	4,375.2
1985	3,650	3,030	620	3.45	5.52	10,453.5	3,422.4	13,875.9	13,763.5	0.361	4,968.5
1986	3,997	3,332	665	3.45	5.52	11,495.4	3,670.8	15,166.2	15,053.8	0.322	4,847.3
1987	4,307	3,606	701	3.45	5.52	12,440.7	3,869.5	16,310.2	16,197.8	0.287	4,648.8
1988	4,636	3,898	738	3.60	5.76	14,032.8	4,250.9	18,283.7	18,171.3	0.257	4,670.0
1989	4,982	4,207	775	3.60	5.76	15,145.2	4,464.0	19,609.8	19,496.8	0.229	4,464.8
1990	5,293	4,488	804	3.60	5.76	16,156.8	4,631.0	20,787.8	20,675.4	0.205	4,238.5
1991										0.183	3,783.6
1992										0.163	3,370.1
1993										0.146	3,018.6
1994										0.130	2,687.8
1995										0.116	2,398.3
1996										0.104	2,150.2
1997										0.093	1,922.8
1998										0.083	1,716.1
1999										0.074	1,530.0
2000	5,293	4,488	804	3.60	5.76	16,156.8	4,631.0	20,787.8	20,675.4	0.066	1,364.6
				T O T A L		271,773.5	85,525.6	357,299.2	354,489.2		74,853.3

*The present economic revenue amounting to P112,400 (.35 x 181 + .55 x 89) 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
TARLAC WATER DISTRICT
1976 PRICES
P x 1,000

	Financial Project Cost	Foreign Component	Domestic Component	Unskilled Labor	Balance of Domestic	Taxes 5%	Others 95%	Shadow Pricing: Foreign Component x 1.2	Shadow Pricing: Unskilled Labor x .5	Shadow Pricing: Others x 1.00	Economic Project Cost	Economic Construct. Cost
Source Development												
a) Structure	6,868.0	1,792.3	5,075.7	594.7	4,481.0	224.1	4,257.0	2,150.8	297.4	4,257.0	6,705.2	5,146.2
b) Equipment	2,689.0	2,140.6	548.4	64.3	484.1	24.2	459.8	2,568.7	32.1	459.8	3,060.6	2,348.9
Storage Facilities												
a) Structure	4,325.0	869.1	3,455.9	496.4	2,959.5	148.0	2,811.5	1,042.9	248.2	2,811.5	4,102.6	3,148.7
b) Equipment	646.0	435.0	211.0	30.3	180.7	9.0	171.7	522.0	15.2	171.7	708.9	544.1
Distribution Mains	11,244.0	5,985.7	5,258.3	924.6	4,333.7	216.7	4,117.0	7,182.8	462.3	4,117.0	11,762.1	9,027.3
Internal Network	8,303.0	3,625.2	4,477.8	856.8	3,621.0	181.0	3,440.0	4,590.2	428.4	3,440.0	8,456.6	6,491.9
Service Connections												
Meters	3,354.3	2,683.2	671.1	181.8	489.3	24.5	464.8	3,219.8	90.9	464.8	3,775.5	2,897.1
Pipes	7,057.4	4,099.1	2,958.3	801.8	2,156.5	107.8	2,048.7	4,918.9	400.9	2,048.7	7,368.3	5,655.2
Others	1,763.3	101.1	1,662.2	450.4	1,211.8	60.6	1,151.2	121.3	225.2	1,151.2	1,497.7	1,149.5
Water District Building	2,166.0	1,009.8	1,156.2	239.2	917.0	45.8	871.2	1,211.8	119.6	871.2	2,202.6	1,690.9
Early Action Works												
Service Connections												
Meters	117.1	99.0	18.1	-	18.1	0.9	17.2	118.8	-	17.2	136.0	112.6
Pipes	160.7	91.8	68.9	4.8	64.1	3.2	60.9	110.2	2.4	60.9	173.5	143.7
Others	30.2	-	30.2	3.7	26.5	1.3	25.2	-	1.9	25.2	27.1	22.4
Vehicles	121.0	72.5	48.5	-	48.5	2.4	46.1	87.0	-	46.1	133.1	110.2
Miscellaneous System												
Improvements	351.0	199.3	151.7	7.2	144.5	7.2	137.3	239.2	3.6	137.3	380.1	314.8
Other Equipment	499.0	423.8	75.2	2.4	72.8	3.6	69.2	508.6	1.2	69.2	579.0	473.5
SUB-TOTAL	49,695.0	23,828.3	25,866.7	4,658.4	21,209.1	1,060.3	20,148.8	28,593.0	2,329.3	20,148.8	51,071.1	39,233.2
Land	297.0	-	297.0	-	297.0	14.8	282.2	-	-	282.2	282.2	233.7
TOTAL	49,992.0	23,828.3	26,163.7	4,658.4	21,506.1	1,075.1	20,431.0	28,593.0	2,329.3	20,431.0	51,353.3	39,516.9

APPENDIX TABLE XI-E-2

REPLACEMENT COST (1976 PRICES)
TARLAC WATER DISTRICT
₱ x 1,000

Year	Vehicles	Meters	Total
1976			
1977			
1978			
1979			
1980			
1981			
1982			
1983			
1984	66.0		66.0
1985	67.1		67.1
1986			
1987			
1988			
1989			
1990			
1991			
1992	66.0	68.0	134.0
1993	67.1	147.3	214.4
1994		181.2	181.2
1995		328.5	328.5
1996		328.5	328.5
1997		370.0	370.0
1998		354.9	354.9
1999		313.4	313.4
2000	66.0	313.4	379.4
TOTAL	332.2	2,405.2	2,737.4

APPENDIX TABLE XI-E-3

SALVAGE VALUE IN 2001 (1976 PRICES)
TARLAC WATER DISTRICT
P x 1,000

	50 Years			25 Years			15 Years			7 Years			Infinite (Land)		
	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value	Eco. Value	%	Salvage Value
1976	240.2	52.0	124.9	239.3	4	9.6									
1977	1,109.5	54.0	599.1	336.0	8	26.9							233.7	100	233.7
1978	3,049.9	56.0	1,707.9	455.6	12	54.7									
1979	5,741.1	58.0	3,329.8	870.6	16	139.3									
1980	3,935.6	60.0	2,361.4	435.0	20	87.0									
1981	2,369.6	62.0	1,469.2	-	-	-									
1982	2,322.6	64.0	1,486.5	32.9	28	9.2									
1983	2,505.4	66.0	1,654.2	274.8	32	87.9									
1984	2,969.6	68.0	2,019.3	486.2	36	175.0									
1985	2,585.9	70.0	1,810.1	241.9	40	96.8									
1986	1,703.5	72.0	1,226.5												
1987	1,217.3	74.0	900.8				188.4	6.6	12.4						
1988	1,217.3	76.0	925.1				188.4	13.3	25.1						
1989	1,217.3	78.0	949.5				188.4	20.0	37.7						
1990	602.1	80.0	481.7				92.7	26.7	24.8						
1991															
1992							56.3	40.0	22.5						
1993							117.2	47.0	55.1						
1994							139.1	53.0	73.7						
1995							252.1	60.0	151.3						
1996							252.1	67.0	168.9						
1997							284.0	73.0	207.3						
1998							274.4	80.0	219.5						
1999							240.5	87.0	209.2						
2000							240.5	93.0	223.7	54.7	86.0	47.0			
TOTAL	32,787.9		21,046.0	3,372.3		686.4	2,514.1		1,431.2	54.7		47.0	233.7		233.7

Total Economic Value: 38,962.7

Total Salvage Value: 23,444.3

APPENDIX TABLE XI-B-4

SUMMARY OF ECONOMIC COST
FIRST APPROACH
TARLAC WATER DISTRICT
₱ x 1,000

	Project Cost	Replacement Cost	Salvage Value	O & M Cost	Total Cost	Discount Factor*	PV of Project Cost	PV of Replacement Cost	PV of Salvage Value	PV of O & M Cost	PV of Total Cost
1976	995.2			12.0	1,007.2	1.000	995.2			12.0	1,007.2
1977	2,052.1			47.0	2,099.1	0.893	1,832.5			42.0	1,874.5
1978	4,743.8			112.0	4,855.8	0.797	3,780.8			89.3	3,870.1
1979	8,939.6			240.0	9,179.6	0.712	6,365.0			170.9	6,535.9
1980	6,018.2			438.0	6,456.2	0.636	3,827.6			278.6	4,106.1
1981	3,457.6			522.0	3,979.6	0.567	1,960.5			296.0	2,256.4
1982	3,424.2			604.0	4,028.2	0.507	1,736.1			306.2	2,042.3
1983	3,942.4			703.0	4,645.4	0.452	1,782.0			317.8	2,099.7
1984	4,821.3	66.0		761.0	5,648.3	0.404	1,947.8	26.7		307.4	2,281.9
1985	4,033.3	67.1		863.0	4,963.4	0.361	1,456.0	24.2		311.5	1,791.8
1986	2,525.4			922.0	3,447.4	0.322	813.2			296.9	1,110.1
1987	1,831.8			976.0	2,807.8	0.287	525.7			280.1	805.8
1988	1,831.8			1,110.0	2,941.8	0.257	470.8			285.3	756.0
1989	1,831.8			1,178.0	3,009.8	0.229	419.5			269.8	689.2
1990	905.3			1,232.0	2,137.3	0.205	185.6			252.6	438.1
1991				1,232.0	1,232.0	0.183				225.4	225.5
1992		134.0			1,366.0	0.163		21.8		200.8	222.7
1993		214.4			1,446.4	0.146		31.3		179.9	211.2
1994		181.2			1,413.2	0.130		23.6		160.2	183.7
1995		328.5			1,560.5	0.116		38.1		142.9	181.0
1996		328.5			1,560.5	0.104		34.2		128.1	162.3
1997		370.0			1,602.0	0.093		34.4		114.6	149.0
1998		354.9			1,586.9	0.083		29.5		102.2	131.7
1999		313.4			1,545.4	0.074		23.2		91.2	114.4
2000		379.4		1,232.0	1,611.4	0.066		25.0		81.3	106.3
2001			23,444.3			0.059			1,383.2		
TOTAL	51,353.8	2,737.4		22,040.0	76,131.2		28,098.3	312.0		4,943.0	33,353.3
											− 1,383.2
											31,970.1

*Discounted at 12 per cent

APPENDIX TABLE II-3-5

SUMMARY OF ECONOMIC COST
SECOND APPROACH
TARLAC WATER DISTRICT
₱ 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	995.2			12.0	1,007.2	1.000	995.2			12.0	1,007.2
1977	2,257.3			50.8	2,308.1	0.893	2,015.8			45.4	2,061.2
1978	5,740.0			130.6	5,870.6	0.797	4,574.8			104.1	4,678.9
1979	11,898.6			302.4	12,201.0	0.712	8,471.8			215.3	8,687.1
1980	8,810.6			595.7	9,406.3	0.636	5,603.5			378.9	5,982.4
1981	5,466.5			766.8	6,233.3	0.567	3,099.5			434.8	3,534.3
1982	5,848.5			958.5	6,807.0	0.507	2,965.2			486.0	3,451.2
1983	7,269.8			1,204.9	8,474.7	0.452	3,285.9			544.6	3,830.5
1984	9,604.0	131.5		1,408.6	11,144.1	0.404	3,880.0	53.1		569.1	4,502.2
1985	8,675.6	144.3		1,725.1	10,545.0	0.361	3,131.9	52.1		622.8	3,806.8
1986	5,757.9			1,990.6	7,748.5	0.322	1,854.2			641.0	2,495.0
1987	4,427.5			2,276.0	6,703.5	0.287	1,270.7			653.2	1,923.9
1988	4,693.1			2,795.0	7,488.1	0.257	1,206.1			718.3	1,924.4
1989	4,975.2			3,204.2	8,179.4	0.229	1,139.3			733.8	1,873.1
1990	2,606.4			3,618.4	6,224.8	0.205	534.3			741.8	1,276.1
1991				3,618.4	3,618.4	0.183				662.2	662.2
1992		385.8			4,004.2	0.163		62.9		589.8	652.7
1993		617.3			4,235.7	0.146		90.1		528.3	618.4
1994		521.7			4,140.1	0.130		67.8		470.4	538.2
1995		945.8			4,564.2	0.116		109.7		419.7	529.4
1996		945.8			4,564.2	0.104		98.4		376.3	474.7
1997		1,065.2			4,683.6	0.093		99.1		336.5	435.6
1998		1,021.8			4,640.2	0.083		84.8		300.3	385.1
1999		902.3			4,520.7	0.074		66.8		267.6	334.6
2000		1,092.3		3,618.4	4,710.7	0.066		72.1		238.8	310.9
2001			67,496.1			0.059			3,982.3		
	89,026.2	7,773.4		57,223.6	154,023.2		44,028.0	856.9		11,091.2	55,976.1
					- 67,496.1						- 3,982.3
					86,527.1						51,993.8

*Escalated annually by 10 per cent from 1976 to 1980, by eight per cent from 1980 to 1985 and by six per cent from 1985 to 1990. Thereafter up to 2000, the escalation factor was maintained at its 1990 level.

**Escalated annually by eight 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 II-E-6

INTERNAL ECONOMIC RATE OF RETURN
TARLAC WATER DISTRICT
P x 1,000

Year	FIRST APPROACH: Benefits (1976 Prices)	FIRST APPROACH: Costs (1976 Prices)	Present Value at 37%: Discount Factor	Present Value at 37%: Benefits (in 1976 Prices)	Present Value at 37%: Costs (in 1976 Prices)	SECOND APPROACH: Escalated Benefits	SECOND APPROACH: Escalated Costs	Present Value at 20%: Discount Factor	Present Value at 20%: Benefits	Present Value at 20%: Costs
1976		1,007.2	1.000		1,007.2	372.7	1,007.2	1.000	372.7	1,007.2
1977	140.1	2,099.1	0.730	102.3	1,522.3	577.9	2,308.1	0.833	481.4	1,922.6
1978	355.4	4,855.8	0.533	189.4	2,588.1	698.5	5,870.6	0.694	484.6	4,074.2
1979	7,890.9	9,179.6	0.389	3,069.6	3,570.8	2,950.6	12,201.0	0.579	1,708.4	7,064.4
1980	8,694.3	6,456.2	0.284	2,469.2	1,833.6	4,754.6	9,406.3	0.482	2,291.7	4,533.8
1981	9,363.6	3,979.6	0.207	1,938.2	823.8	6,510.9	6,233.3	0.402	2,617.4	2,505.8
1982	9,888.1	4,028.2	0.151	1,493.1	608.3	8,190.0	6,807.0	0.335	2,743.6	2,280.3
1983	10,518.1	4,645.4	0.110	1,157.0	507.7	9,582.2	8,474.7	0.279	2,673.4	2,364.4
1984	10,728.0	5,648.3	0.086	922.6	485.8	10,875.7	11,144.1	0.233	2,534.0	2,596.6
1985	11,282.2	4,963.4	0.059	665.6	292.8	13,821.4	10,545.0	0.194	2,681.4	2,045.7
1986	11,461.0	3,447.4	0.043	492.8	148.2	15,124.9	7,748.5	0.162	2,450.2	1,255.2
1987	9,818.7	2,807.8	0.031	304.4	87.0	16,286.3	6,703.5	0.135	2,198.6	905.0
1988	10,138.2	2,941.8	0.023	233.2	67.7	18,279.1	7,488.1	0.112	2,047.2	836.7
1989	10,194.4	3,009.8	0.017	173.3	51.2	19,625.8	8,179.4	0.093	1,825.2	760.7
1990	10,213.3	2,137.3	0.012	122.6	25.6	20,827.8	6,224.8	0.078	1,624.6	485.5
1991		1,232.0	0.009	91.9	11.1		3,618.4	0.065	1,353.8	235.2
1992		1,366.0	0.006	61.3	8.2		4,004.2	0.054	1,124.7	216.2
1993		1,446.4	0.005	51.1	7.2		4,235.7	0.045	937.2	190.6
1994		1,413.2	0.003	30.6	4.2		4,140.1	0.038	791.4	157.3
1995		1,560.5	0.0025	25.5	3.9		4,564.2	0.031	645.7	141.5
1996		1,560.5	0.0018	18.4	2.8		4,564.2	0.026	541.5	118.7
1997		1,602.0	0.0012	13.3	2.1		4,683.6	0.022	458.2	103.0
1998		1,586.9	0.0010	10.2	1.6		4,640.2	0.018	374.9	83.5
1999		1,545.4	0.0007	7.1	1.1		4,520.7	0.015	312.4	67.8
2000	10,213.3	1,611.4	0.0005	5.1	0.8	20,827.8	4,520.7	0.013	270.8	58.8
		76,131.2			13,673.1		153,833.6			36,010.7
2001		- 23,444.3	0.0004		- 9.4		- 67,496.1	0.010		- 675.0
	222,819.3	52,686.9		13,647.9	13,663.7	356,756.4	86,337.5		35,545.2	35,335.7
	(4.229)	(1.000)		(0.999)	(1.000)	(4.132)	(1.000)		(1.006)	(1.000)