BIBLIOGRAPHIC DATA SHEET	1. CONTROL NUMBER PN-AAK-217	2. SUBJECT CLASSIFICATION (695) AP10-0000-G792

3. TITLE AND SUBTITLE (240) Water supply: Misamis Occidental Water District; feasibility study, technical final report; volume II (appendices)

4. PERSONAL AUTHORS (100)

5. CORPORATE AUTHORS (101)

Camp Dresser + McKee Intl., Inc.

6. DOCUMENT DATE (110)	7. NUMBER OF PAGES (120)	8. ARC NUMBER (170)
1976	217p.	

9. REFERENCE ORGANIZATION (130) CDM

10. SUPPLEMENTARY NOTES (500)

(Volume I, text, 193p.: PN-AAK-216; Summary Final Report, 45p.: PN-AAK-218)

11. ABSTRACT (950)

public utilities

water distribution

12. DESCRIPTORS (920)		13. PROJECT NUMBER (150)	
Philippines	design criteria	492026400	
water resources	financing	14. CONTRACT NO.(140)	15. CONTRACT TYPE (140)
water supply	economic aspects	unknown	
water management	cost analysis	16. TYPE OF DOCUMENT (160)	
feasibility	engineering		

AID 590-7 (10-79)

LOCAL WATER UTILITIES ADMINISTRATION

REPUBLIC OF THE PHILIPPINES

FEASIBILITY STUDY TECHNICAL FINAL REPORT

VOLUME II (APPENDICES)

WATER SUPPLY

MISAMIS OCCIDENTAL WATER DISTRICT

JANUARY 1976

CAMP DRESSER & McKEE INTERNATIONAL INC.

CONSULTING ENVIRONMENTAL ENGINEERS

FOREWORD

Volume II (Appendices) of the Technical Final Report for the Misamis Occidental Water District Water Supply Feasibility Study contains detailed information relating to several chapters of Volume I. The appendices may either pertain to more than one chapter of Volume I or to specific sections of a chapter.

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.

The other appendices refer to specific chapter sections in Volume I. The appendices are numbered according to the 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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A P P E N D I X A

DESIGN CRITERIA

APPENDIX A DESIGN CRITERIA

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APPENDIX A

DESIGN CRITERIA

General

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

Study Area

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

Population Projections

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

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

Land Use Projections

Residential, institutional, industrial, commercial and public 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 customers and borrowers would be considered separately. For borrowers, an average unit consumption of 20 lpcd shall be assumed. However, the final estimates will be based on actual field measurements. Field measurement will be done by direct meter reading by isolating certain service area sections which will represent different economic classes of customers. This measurement will be conducted in areas which have adequate supply.

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

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

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

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

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

Unaccountable Water

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

Total Supply

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

Demand Variation

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

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

The present and future projected water demands will be tabulated.

Population and Demand Distribution

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

Existing Water System Analysis

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

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

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

a) Asbestos Cement Pipe

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

b) Cast Iron Pipe

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

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

Computer Studies for Future System

The proposed system will be studied for the design year 2000

[1/] Subject to field verification.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Hydrants	:	inlet-outlet characteristics, locations
Valves	:	check valves, closed or throttled valves

Pipes

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

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

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

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

d. Operation and maintenance problems; and

e. Economic Consideration

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

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

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

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

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

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

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

Recommended Pipe Materials

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

Pipe Material	Diameter (mm) 10-100	150-400	450-600	700-1200	Service 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 lolyethylene 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

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[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.

<u>Water Quality Criteria</u>

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

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

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

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

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

Surface Water Sources

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

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

Hydrological Studies

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

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

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

Raw Water Pump Stations

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

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

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

Staging of Source Development

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

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

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

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

Surveys

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

Groundwater-Springs

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

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

Groundwater Wells

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

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

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

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

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

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

Water Treatment Works

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

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

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

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

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

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

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

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

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

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

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

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

(7) Unessential instrumentation will be avoided.

<u>Types of Water Treatment Plants</u>

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

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

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

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

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

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

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

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

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

APPENDIX TABLE A-1

APPLICATION OF TREATMENT METHODS[4]

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

[4] E—essential; O—optional; S—special justification required.

[5] Superchlorination shall be followed by dechlorination.

[6] As alternate, dilute with low-chloride water.

[7] Double settling shall be provided for coliform exceeding 20,000 M.B.N.

[8] For extreme muddy water, presedimentation by plain settling may be provided.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX TABLE A-2

DESIGN PARAMETERS OF SETTLING BASINS

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

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

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

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

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[9] With subsequent filtration.

APPENDIX TABLE A-3

TYPICAL WEIR OVERFLOW RATES

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX TABLE A-4

RECOMMENDED VELOCITIES FOR FILTRATION UNITS

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

Cost Estimates

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

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

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

Economic Cost Comparison

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

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

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

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

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

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

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

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

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

A P P E N D I X B

BASIS OF COST ESTIMATES

APPENDIX B BASIS OF COST ESTIMATES

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APPENDIX B

BASIS OF COST ESTIMATES

General

Cost data 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 United States, 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 (unskilled and skilled), 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 cost, administrative and legal costs, and interest during construction. The project cost is the sum of construction cost, 25% engineering and contingencies, land cost, 3% administrative and legal costs, and 12 per cent interest during construction.

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-1. 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

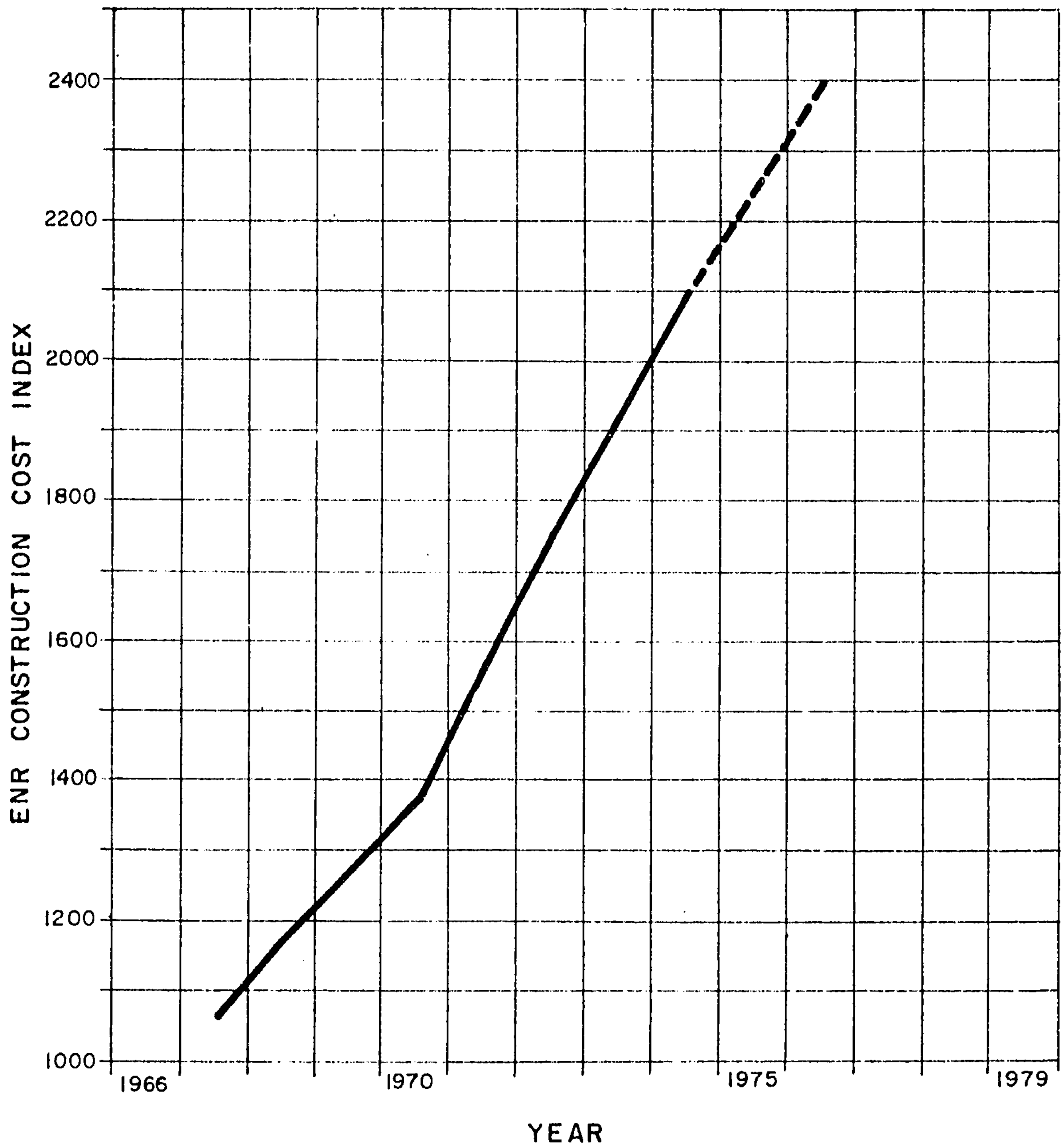
The construction costs of tunnels are heavily dependent on a large number of variables. Among these are the types of rock or other material encountered, the physical or structural defects of the rocks, the extent to which water is present in the formations along the route of the proposed tunnel, length 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 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-2. 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 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

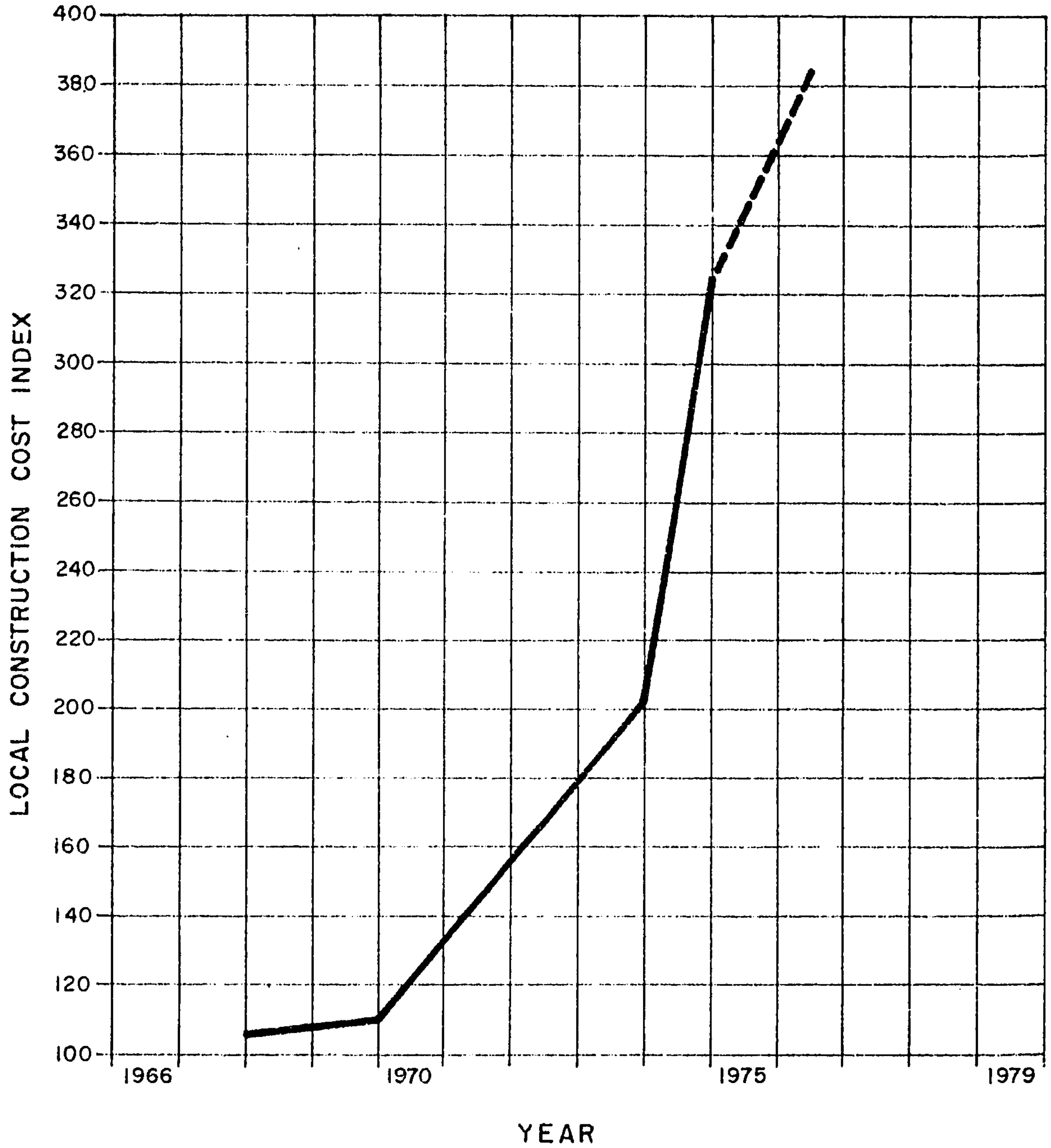


NOTE :

BASE YEAR IS 1913, WITH
CONSTRUCTION COST INDEX = 100

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

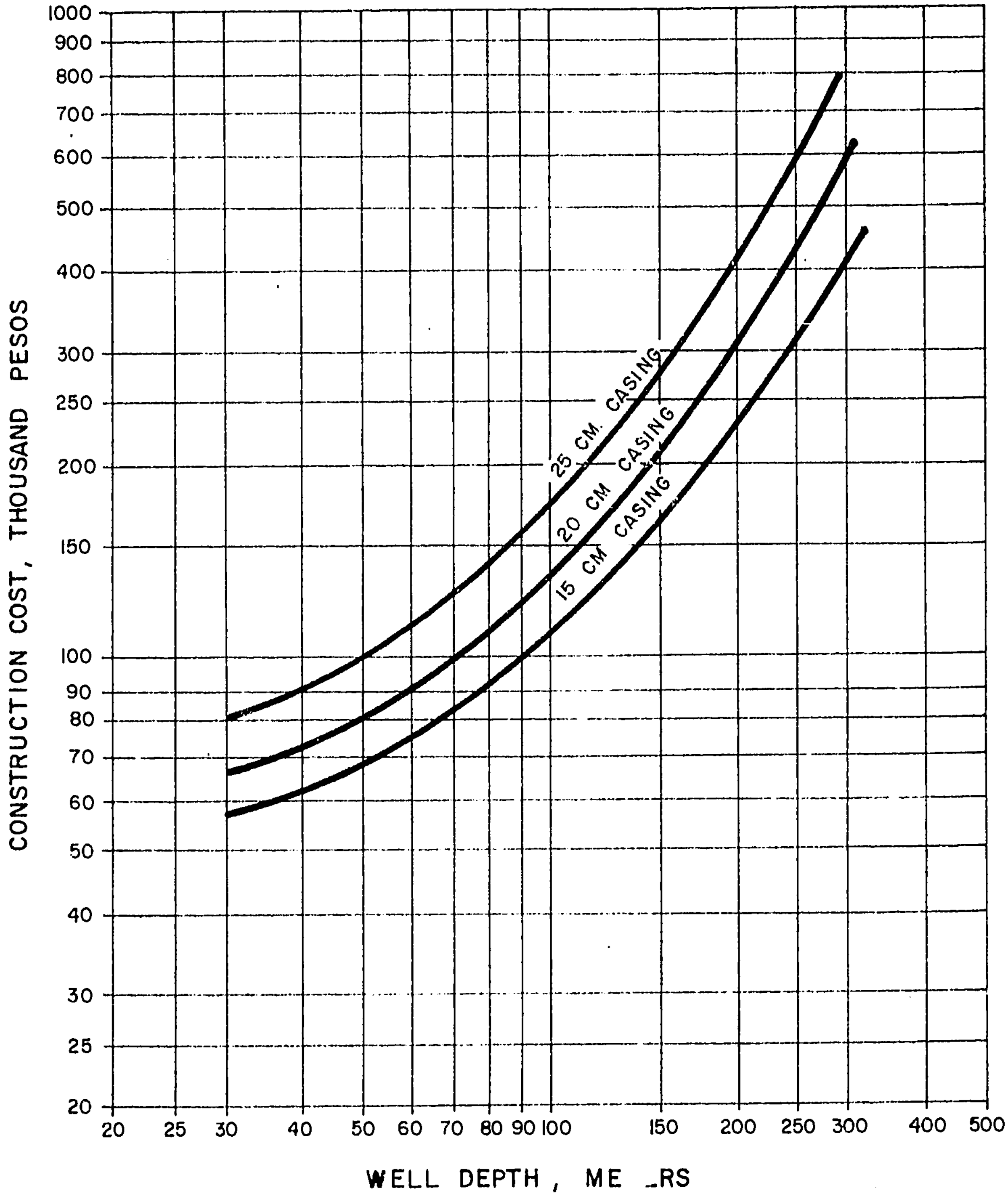
APPENDIX FIGURE B-1
ENGINEERING NEWS RECORD CONSTRUCTION COST INDEX



NOTE :

BASE YEAR IS 1965, WITH
CONSTRUCTION COST INDEX = 100

APPENDIX FIGURE B-2
LOCAL CONSTRUCTION COST INDEX



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)

APPENDIX TABLE B-1

UNIT COSTS[1] FOR DAM AND APPURTENANCES

A. Dam Embankment

Item	Unit	Unit Cost (July 1976) (₱)	Remarks
Mobilization	L.S.[2]	300,000	
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	
Placement of fine aggregate	cum	12	
Impervious earth core			
hauling	cum/km	8	
placement	cum	7	
Backfill			
dump	cum	8	
compacted (machine)	cum	60	
Crushed rock	cum	50	material
Riprap (placement)	sqm	30	
Steel sheet pile in place	ton	10,000	

B. Roads

Common Excavation for Roads	cum	4	
Road Embankment	cum	6.5	placement and compaction

[1]Foreign exchange component of dams and appurtenances is 30% of total construction cost.

[2]Lump sum.

APPENDIX TABLE B-1 (Continued)

Macadam Pavement	sqm	20
Bituminous Surface	sqm	27
C. Spillway		
Excavation		(see previous unit costs)
Concrete (Plain)	cum	500
Reinforced Concrete	cum	900
D. Miscellaneous		
Generators		
25 kw	L.S.	3,000
50 kw	L.S.	10,000
100 kw	L.S.	15,000
250 kw	L.S.	20,000
Structural Steel	ton	8,000
Sluice Gates	L.S.	50,000
Miscellaneous	L.S.	10% of Total

APPENDIX TABLE B-2

TUNNEL CONSTRUCTION COST[3] ESTIMATES

(July 1976 prices)

Item Number	Work Description	Foreign Exchange Component (% 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	100/cum
3	Tunnel-Concrete Lining	10	600/cum
4	Tunnel-Steel Supports	30	(See page 6)
5	Rock Bolts	20	(See page 6)
6	Grouting	10	(See page 6)
7	Drainage and Ventilation		(See page 6)
8	Miscellaneous	20	(See page 6)
9	Cofferdam and General Dewatering	17	(See page 6)

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

APPENDIX TABLE B-2 (Continued)

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

Item Number	Work Description	Tunnel diameter (m) 2.5	3.0	4.0	5.0	7.0
4	Steel Supports[4]	800	900	1,100	1,300	1,550
5	Rock Bolts[4]	350	400	450	500	550
6	Grouting[4]	400	500	650	800	900
7	Drainage and Ventilation	500	550	600	650	650
8	Miscellaneous	500	600	750	900	1,000

C. Estimated Lump Sum Cost for Cofferdam, Channels and General Dewatering.

Drainage Area of River (sqkm)	Estimated Cost ₱ x 10^6
40-50	2.0
50-100	3.0
100-200	4.0
200-500	4.5
500-800	5.0

D. Mobilization and Demobilization = ₱200,000

[4] For required length only.

Schedule 40 black iron pipe casing. Labor costs include mobilization and demobilization charges, drilling, installation of casing, perforating, developing the well, test pumping, well disinfection, and grouting the upper 15 to 30 m of the well.

Deep Well Pumps 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 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 is also included. Cost of land, power transmission and substation, and 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 to use 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 pipes of various materials including cast iron, asbestos cement, steel, ductile iron and pre-stressed concrete. The unit cost of pipelines is based on the assumption that all pipes will be locally manufactured. The estimated unit in-place costs based on lower limit of cost envelope, are presented in Appendix Tables B-3a and B-3b. The costs include pipe, fittings, jointing materials, excavation, pipe 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 stations are shown in Appendix Figure B-7. Development of these curves is based on available local information and US 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

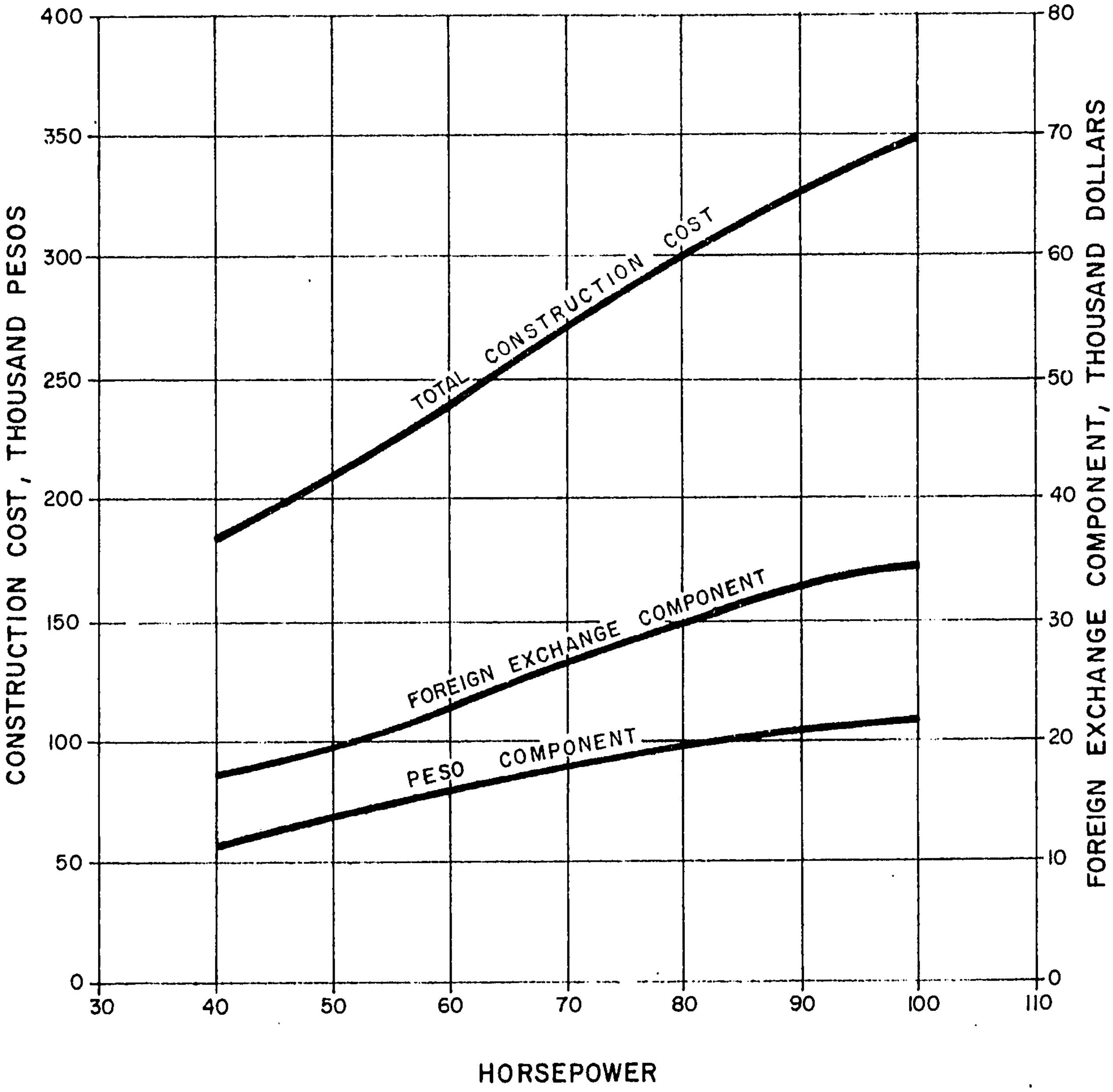
Construction cost estimates of ground storage reservoirs are presented in Appendix Figure B-8, including steel, reinforced concrete and prestressed concrete tanks. The costs for steel and reinforced concrete tanks are based on updated costs of actual construction in the past in the Philippines and in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

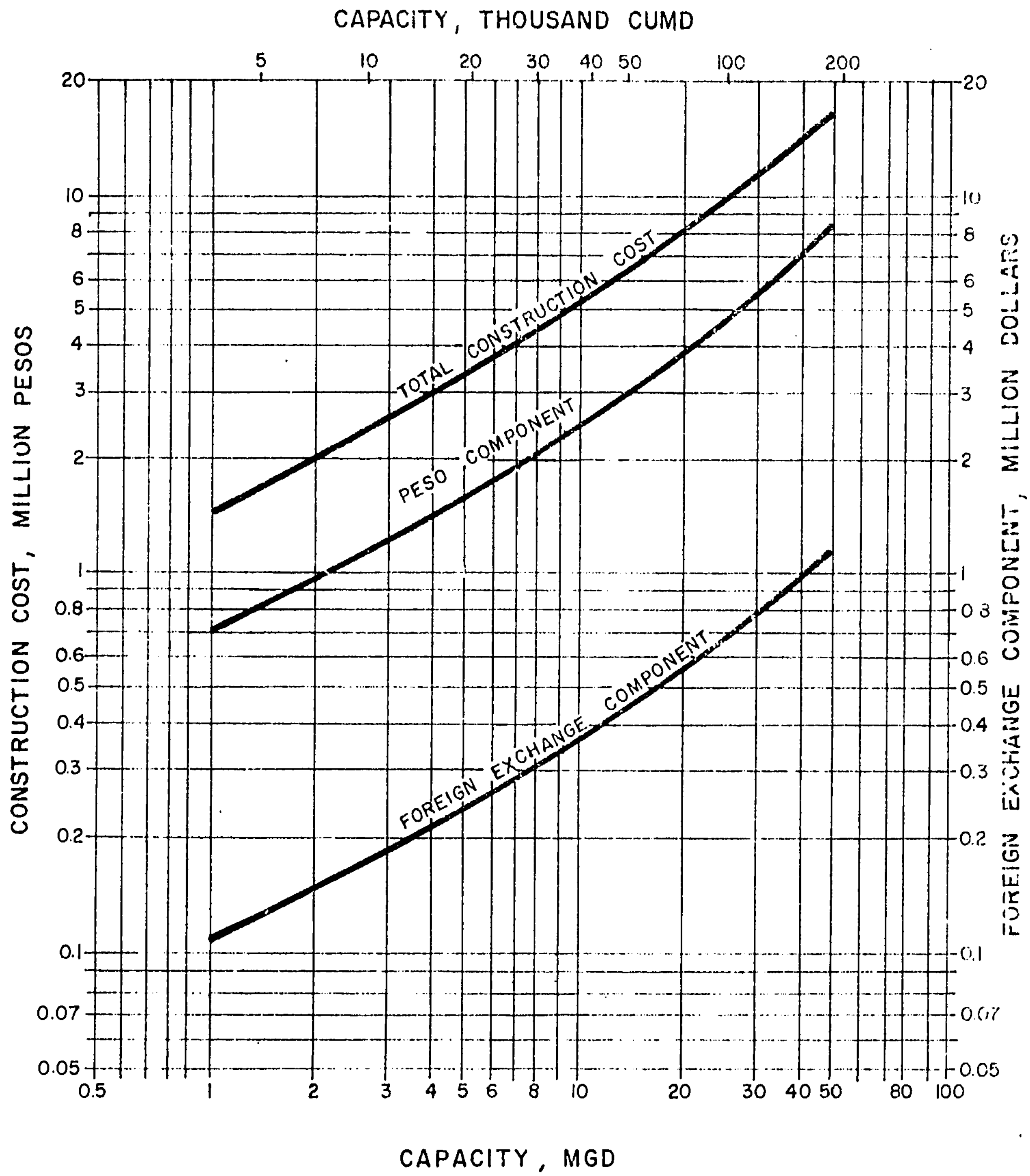
Gate valves up to 600 mm diameter can be manufactured in the Philippines. Unit costs for gate valves are based on the prices of locally manufactured valves. However, studies indicate that the prices of imported (U.S.) gate valves conforming to AWWA Standard C500 are only slightly higher than the locally manufactured valves. The in-place estimating prices for gate valves up to 300 mm diameter are shown in Appendix Table B-4. The unit prices include a locally manufactured cast iron valve box and cover.



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)

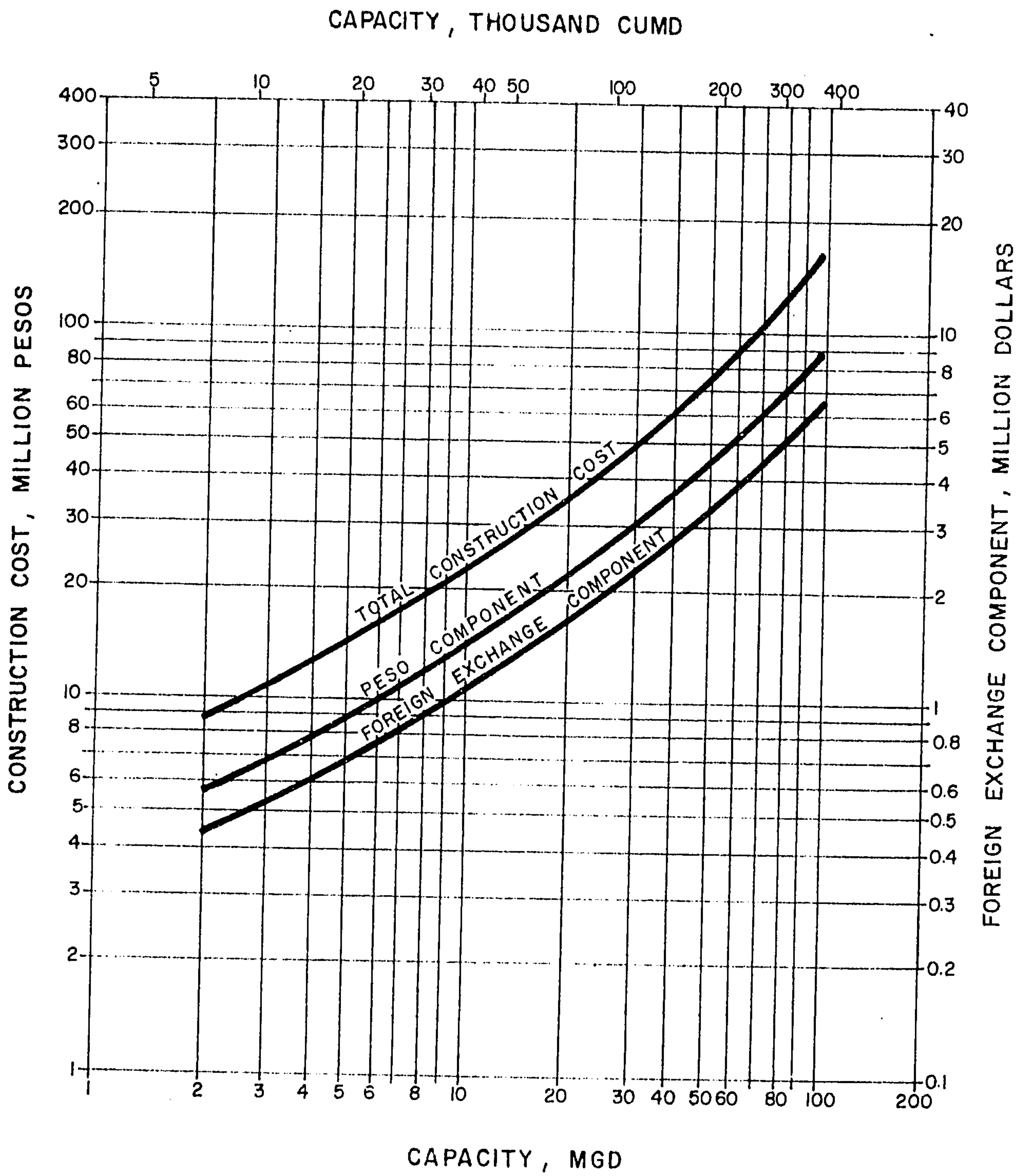


FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS CDM-LWUA

APPENDIX FIGURE B-3

WATER PUMP STATION CONSTRUCTION COSTS

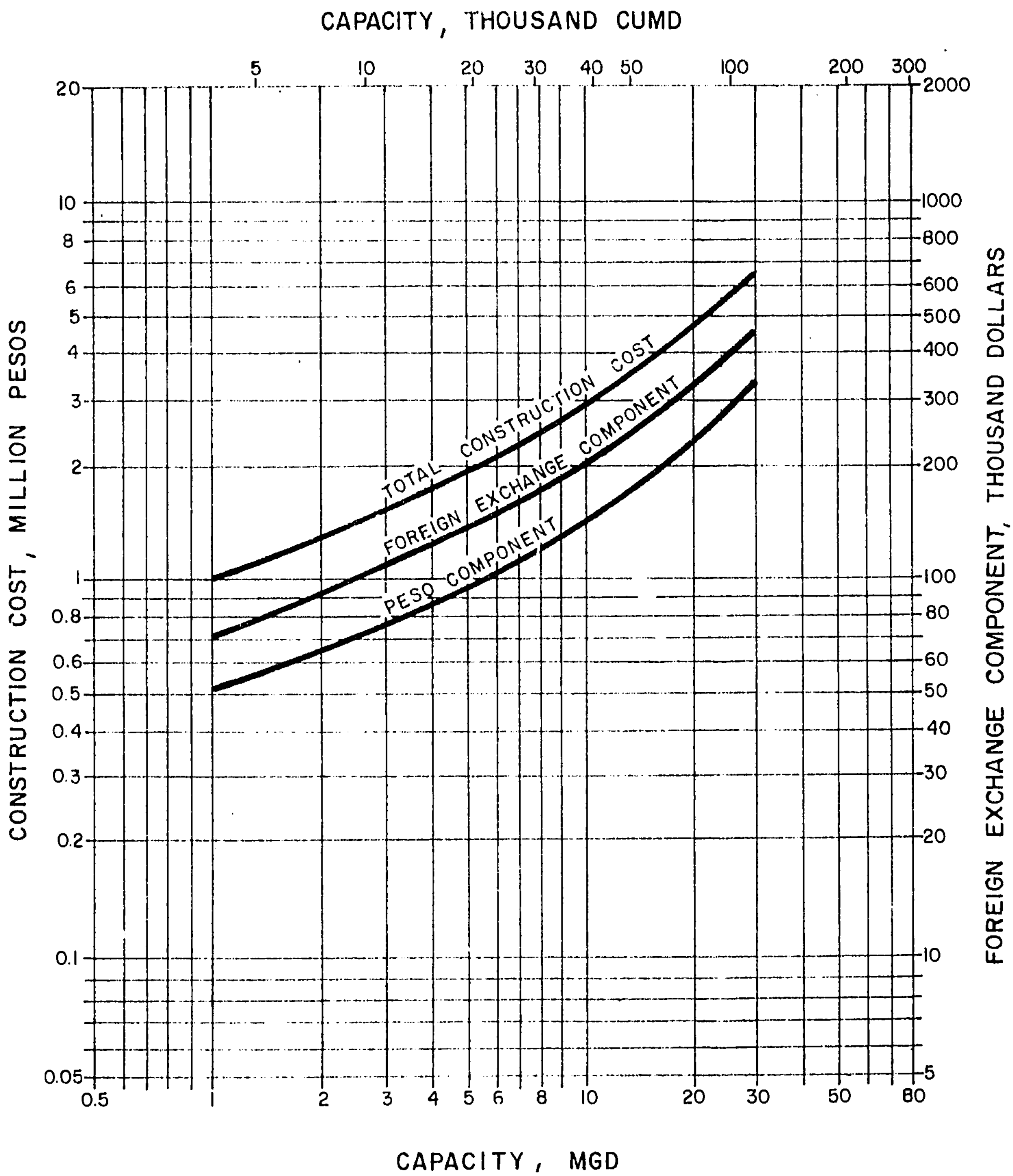
(JULY 1976 PRICES)



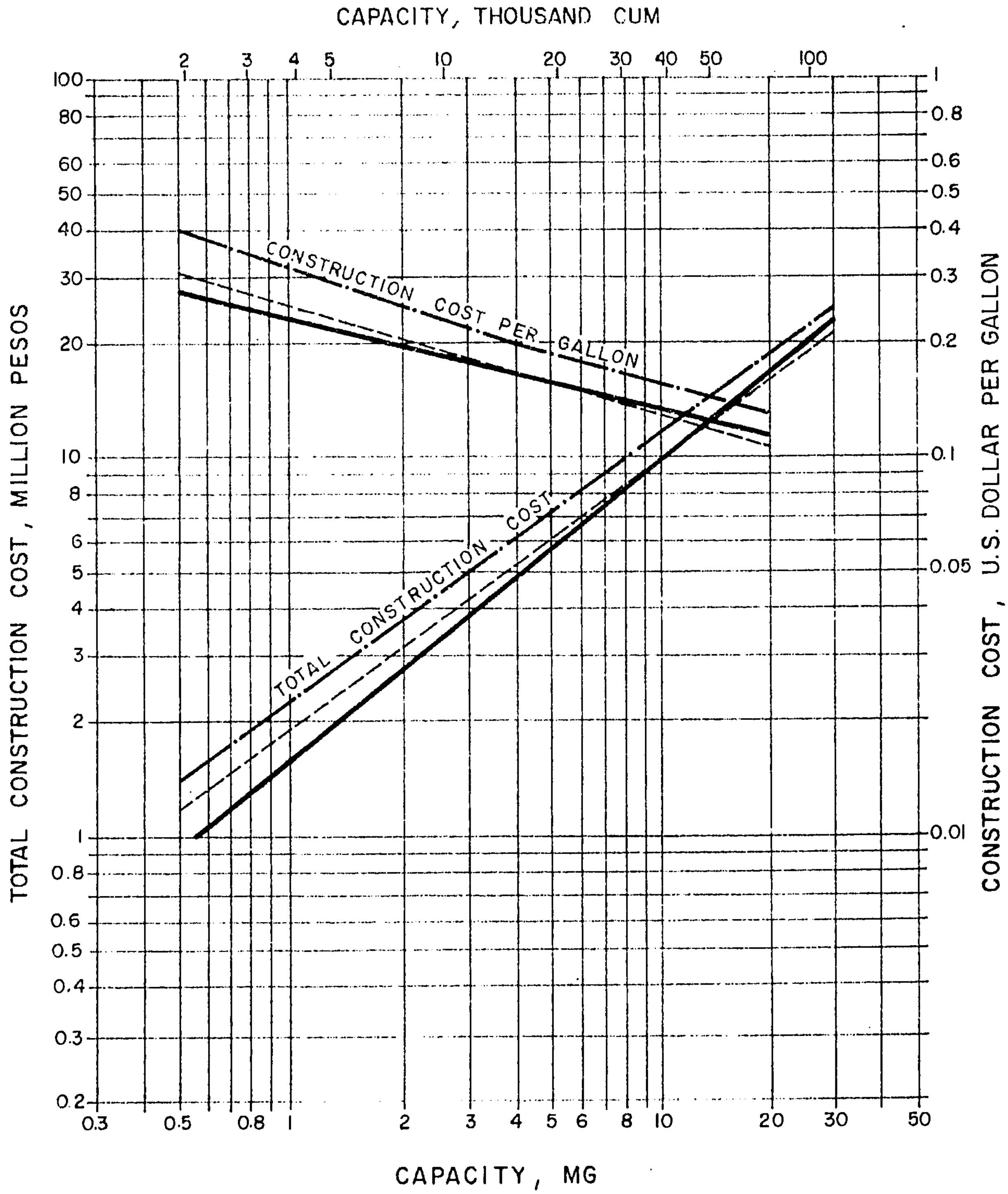
NOTE:

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 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)

APPENDIX TABLE B-3a

ESTIMATED UNIT COSTS FOR PIPELINES IN PLACE[5]

JULY 1976

Diameter (mm)	Peso Component (₱/m)	Foreign Exchange Component ($/m)	Total (₱/m)
100	94	3.7	120
150	140	5.7	180
200	190	10.0	260
250	330	14.3	430
300	400	21.4	550
400	530	35.7	780
450	640	65.7	1,100
500	740	80.0	1,300
600	900	90.0	1,530
800	1,090	111.4	1,870
900	1,605	108.0	2,360
1,200	2,000	183.0	3,280
1,350	2,660	295.7	4,590
1,500	2,890	337.0	5,250

[5] The above unit costs are used in the alternative studies.

APPENDIX TABLE B-3b

ESTIMATED UNIT COSTS FOR PIPELINES IN PLACE[6]
July 1976

Pipe Diameter (mm)	Peso Component (₱/m)	Foreign Component ($/m)	Total (₱/m)
100	113	4.5	145
150	190	7.8	245
200	263	13.8	360
250	350	17.9	475
300	436	23.4	600
350	429	44.4	740
400	505	52.1	870
450	586	60.6	1,010
500	650	67.1	1,120
600	802	79.7	1,360
700	976	89.1	1,600
750	1,103	92.4	1,750
800	1,160	97.1	1,840
900	1,336	112.0	2,120
1,000	1,468	123.1	2,330
1,050	1,575	132.1	2,500
1,100	1,657	139.0	2,630
1,200	1,757	160.4	2,880

[6]The above unit costs are used in the computation of capital costs.

APPENDIX TABLE B-4

ESTIMATED IN-PLACE COSTS[7] OF GATE VALVES, BUTTERFLY VALVES

Inside Diameter (mm)	Item	In-Place Cost ₱/unit
100	Gate Valve	₱ 2,040
150	Gate Valve	2,800
200	Gate Valve	4,310
250	Gate Valve	5,780
300	Gate Valve	7,240
300	Butterfly Valve	7,400
350	Butterfly Valve	9,400
400	Butterfly Valve	12,000
450	Butterfly Valve	16,300
500	Butterfly Valve	19,300
600	Butterfly Valve	45,200
750	Butterfly Valve	75,600
900	Butterfly Valve	113,000

---

[7] a) Include valve box and cover of each item given.
b) Costs are for July 1976.
c) Gate valves are locally manufactured items.
d) Butterfly valves are imported items.

Butterfly Valves

Current local practice uses butterfly valves instead of gate valves for sizes 400 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-4.

Miscellaneous Valves

Miscellaneous valves employed in water supply systems include check valves, pressure reducing valves, altitude valves and surge relief valves. Except for small size check valves, none of these valves is manufactured locally.

The unit costs are based on the assumption that all materials to be used, except the valves themselves, are locally manufactured.

The costs of altitude and pressure reducing valves include an isolation valve (gate of butterfly depending on size) before and after the main valve and a valved by-pass line of the same diameter as the main line. The costs of surge relief valves include a shut-off valve (gate or butterfly) preceding the relief valve. All unit costs include all fittings, connections, and miscellaneous materials.

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-5 and include fire hydrant, gate valve, tee filling, jointing materials, concrete thrust blocks, miscellaneous materials, and installation. All materials are assumed to be locally manufactured.

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 for service connections from ½ in to 2 in are shown in Appendix Table B-6. The cost figures

APPENDIX TABLE B-5

FIRE HYDRANTS

Size (inlet connection)	In Place Cost[9] (₱)
100 mm	7,900
150 mm	8,800

APPENDIX TABLE B-6

ESTIMATED UNIT CONSTRUCTION COSTS[10] FOR HOUSE SERVICE CONNECTIONS

Diameter (in)	PE Pipe Service Connection (₱)	GI Pipe Service Connection (₱)
½	366	414
¾	399	494
1	509	606
1¼	706	855
1½	872	1,068
2	1,260	1,462

---

[9] a) Costs are for July 1976.
b) Hydrants are locally manufactured.

[10] a) Above costs are for July 1976.
b) Costs do not include curb stop, curb box and cover, water meter, and surface or pavement replacement.
c) All materials are locally manufactured.

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.

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.

Cast Iron and Ductile Iron Pipe

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

Gray cast iron 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.

<u>Cast Iron Pipe</u>. 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 maufactured 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.

<u>Ductile Iron Pipe</u>. 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 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 1 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 tons per year but was undergoing 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 polyvinyledine chloride (Saran). Volume production of pipe began in 1948, when polythylene (PE) was accepted for various water uses.

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

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

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

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

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

To date, plastic pipe has not been used extensively in the Philippines for water mains. Limited experience with PVC pipe water mains used in a high pressure distribution system has not been satisfactory because of frequent failures and leakage particularly at the joints. One problem that has discouraged or deterred some engineers to specify PVC pipe is the non-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 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 aligmments which are subject to traffic loads is 90 cm. For the areas with no traffic loads a minimum cover of 60 cm may be used. Trenches may be dug manually or with excavation machinery.

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

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

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

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

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

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

Excavation. Whether excavation is done manually or by machine, the excavated material is piled on one side of the trench at a distance away from the trench 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 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 backhoe. 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 method, 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, and 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 is 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 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. <u>Reinforced Lining</u>. 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. <u>The Tate Process</u>. The mandrel process, commonly known as the Tate process after its Australian inventor, cleans and scours out encrustation from the pipe, then lines the pipe with cement mortar. An advantage of the Tate process is that road opening is kept to a minimum. Only two major digging operations take place at both ends of a 90 m section of main, and only small openings are required to disconnect and temporarily bypass service connections. The exact location of each service connection is obtained by electrifying the household system and sweeping the "live" area with a detector which tells the operator through headphones where the connection is located. Customers suffer only little inconvenience, with full service restorable in 24 hours.

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

Pipe Cleaning in the Philippines

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

Tunnel Construction Methods

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

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

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

Pumping Stations

General. Water supply pumping stations may be classified into raw water pumping stations, deep well pumping stations, and booster pumping stations. The latter 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 weather-proof 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 reinforeced 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 Plant

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

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

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

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

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

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

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

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

A P P E N D I X   D

OUTLINE SPECIFICATIONS

APPENDIX D OUTLINE SPECIFICATIONS

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APPENDIX D

OUTLINE SPECIFICATIONS

Spring Intake Structure

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

Hydraulic Control Structure

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

Dams and Appurtenances

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

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

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

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

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

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

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

Diversion Dams

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

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

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

Access and Service Roads

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

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

Water Transmission Pipelines

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

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

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

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

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

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

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

Water Treatment Plant

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

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

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

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

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

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

Administration Building

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

Well Construction

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

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

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

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

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

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

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

Flow Meters (Mainline Meters)

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

The venturi meter tube shall be of standard or long form design, the included angle of the outlet cone being approximately 8^{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.

Submersible Deep Well Pump

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 oable, surface plate, strainer, discharge pipe, pump bowls, impellers shall oomply 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 manufaoturer or supplier shall furnish the services of competent personnel to check the installation and testing of the equipment. Spare parts, as specified, and operation and maintenance manuals shall be be furnished.

Diesel Engine

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

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

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

Diesel Generator Unit

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

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

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

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

Chlorination System

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

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

Chlorinators may be directly mounted on 150-lb cylinder or ton container, wall - or floor-mounted units. Models of a design that permit enlarging the capacity by replacement of a component such as 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.)

<u>Installation of Equipment - General</u>

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 IV

TABLE IV-E-1

SYSTEM DATA
OZAMIZ CITY EXISTING SYSTEM PK-HR(RESERVOIR AND PUMP FLOWS UNKNOWN)

INPUT AND OUTPUT IN	LPS
NO OF NODES	63
NO OF PIPES	92
MAX NO OF ITERATIONS	20
PEAKING FACTOR	1.00000
ALLOW P-DROP FR/STATIC - PCT	50.0
STATIC HGL FOR P-DROP CALC	62.3
MAX UNBAL - LPS	0.10000
MAX ALLOW VEL - MPS	3.000
MIN ALLOW VEL - MPS	0.400
MAX ALLOW HL - MT/1000 MT	10.00
MIN ALLOW HL - MT/1000 MT	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	36.52
BANDWIDTH	10

ITER 1 UNBAL 11.67 LPS
ITER 2 UNBAL 4.99 LPS
ITER 3 UNBAL 4.01 LPS
ITER 4 UNBAL 0.99 LPS
ITER 5 UNBAL 0.08 LPS

SOLUTION NO. 1 REACHED IN 5 ITERATIONS
0.0786 IMBALANCE

TABLE IV-E-2

PIPE DATA

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS--- MT	MT/1000	CK
1	2	1	102	100.	100	0.399E-01	0.74	0.09	LO	0.02	0.23	LO
2	2	3	152	90.	100	0.498E-02	1.35	0.07	LO	0.01	0.10	LO
3	3	4	152	190.	100	0.105E-01	0.63	0.03	LO	0.00	0.02	LO
4	7	6	102	194.	100	0.774E-01	0.42	0.05	LO	0.02	0.08	LO
5	8	7	102	120.	100	0.479E-01	0.94	0.11	LO	0.04	0.35	LO
6	9	5	102	212.	100	0.845E-01	0.97	0.12	LO	0.08	0.38	LO
7	9	8	102	90.	100	0.359E-01	2.71	0.33	LO	0.23	2.53	
8	7	10	38	200.	80	0.143E 02	0.08	0.07	LO	0.14	0.72	
9	8	10	102	248.	100	0.989E-01	1.41	0.17	LO	0.19	0.75	
10	11	10	38	234.	80	0.167E 02	0.10	0.09	LO	0.25	1.09	
11	13	9	152	240.	100	0.133E-01	4.65	0.26	LO	0.23	0.95	
12	15	2	152	366.	100	0.203E-01	3.29	0.18	LO	0.18	0.50	
13	16	17	152	465.	100	0.257E-01	2.61	0.14	LO	0.15	0.33	LO
14	15	16	102	80.	100	0.319E-01	4.94	0.61		0.62	7.69	
15	14	15	102	110.	100	0.439E-01	4.07	0.50		0.59	5.36	
16	13	14	102	163.	100	0.650E-01	2.54	0.31	LO	0.36	2.24	
17	12	13	102	94.	100	0.375E-01	1.08	0.13	LO	0.04	0.46	LO
18	12	65	102	190.	100	0.758E-01	1.43	0.17	LO	0.15	0.77	
19	65	11	76	114.	80	0.279E 00	1.01	0.22	LO	0.28	2.50	
20	11	23	38	55.	80	0.394E 01	0.19	0.16	LO	0.18	3.21	
21	22	12	102	45.	100	0.179E-01	2.76	0.34	LO	0.12	2.62	
22	20	13	152	50.	100	0.277E-02	6.85	0.38	LO	0.10	1.95	
23	19	14	102	55.	100	0.219E-01	2.04	0.25	LO	0.08	1.49	
24	18	15	102	55.	100	0.219E-01	5.15	0.63		0.46	8.31	
25	19	18	102	110.	100	0.439E-01	2.36	0.29	LO	0.22	1.96	
26	20	19	102	160.	100	0.638E-01	2.62	0.32	LO	0.38	2.38	
27	21	20	102	54.	100	0.215E-01	1.84	0.23	LO	0.07	1.23	
28	21	22	102	46.	100	0.183E-01	0.40	0.05	LO	0.00	0.07	LO
29	35	23	38	220.	90	0.127E 02	0.25	0.22	LO	0.96	4.36	
30	25	22	102	100.	100	0.399E-01	2.55	0.31	LO	0.23	2.26	
31	26	21	102	100.	100	0.399E-01	2.43	0.30	LO	0.21	2.07	
32	27	20	152	100.	100	0.554E-02	8.02	0.44		0.26	2.61	
33	29	19	102	150.	100	0.598E-01	2.45	0.30	LO	0.31	2.09	
34	28	18	102	150.	100	0.598E-01	3.22	0.39	LO	0.52	3.48	
35	26	27	102	54.	100	0.215E-01	0.74	0.09	LO	0.01	0.23	LO
36	25	26	102	47.	100	0.187E-01	0.90	0.11	LO	0.02	0.33	LO
37	24	25	102	55.	100	0.219E-01	1.36	0.17	LO	0.04	0.70	
38	33	24	102	50.	100	0.199E-01	1.70	0.21	LO	0.05	1.06	
39	32	25	102	50.	100	0.199E-01	2.28	0.28	LO	0.09	1.84	
40	31	26	102	50.	100	0.199E-01	2.48	0.30	LO	0.11	2.14	
41	30	27	152	50.	100	0.277E-02	7.64	0.42		0.12	2.39	
42	29	28	102	110.	100	0.439E-01	0.38	0.05	LO	0.01	0.07	LO
43	31	30	152	54.	100	0.299E-02	0.00	0.00	LO	0.00	0.00	LO
44	31	32	152	47.	100	0.260E-02	0.03	0.00	LO	0.00	0.00	LO
45	32	33	152	56.	100	0.310E-02	0.25	0.01	LO	0.00	0.00	LO
46	33	34	152	105.	100	0.581E-02	0.97	0.05	LO	0.01	0.05	LO
47	34	35	102	20.	100	0.798E-02	3.42	0.42		0.08	3.89	
48	35	36	102	520.	100	0.207E 00	1.77	0.22	LO	0.60	1.15	
49	37	34	102	50.	100	0.199E-01	2.68	0.33	LO	0.12	2.48	
50	38	33	102	50.	100	0.199E-01	2.62	0.32	LO	0.12	2.37	

TABLE IV-E-2 (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
51	39	32	102	50.	100	0.199E-01	2.62	0.32	LO	0.12	2.38	
52	40	31	102	50.	100	0.199E-01	2.63	0.32	LO	0.12	2.40	
53	41	30	152	50.	100	0.277E-02	7.88	0.43		0.13	2.53	
54	41	40	102	54.	100	0.215E-01	0.53	0.06	LO	0.01	0.12	LO
55	40	39	102	47.	100	0.187E-01	0.21	0.03	LO	0.00	0.02	LO
56	39	38	102	57.	100	0.227E-01	0.15	0.02	LO	0.00	0.01	LO
57	38	37	102	95.	100	0.379E-01	0.03	0.00	LO	0.00	0.00	LO
58	48	37	102	52.	100	0.207E-01	2.79	0.34	LO	0.14	2.66	
59	47	38	102	55.	100	0.219E-01	2.70	0.33	LO	0.14	2.51	
60	46	39	102	55.	100	0.219E-01	2.69	0.33	LO	0.14	2.49	
61	45	40	102	55.	100	0.219E-01	2.46	0.30	LO	0.12	2.10	
62	44	41	152	55.	100	0.304E-02	8.68	0.48		0.17	3.03	
63	43	29	102	105.	100	0.419E-01	3.70	0.45		0.47	4.51	
64	42	28	102	105.	100	0.419E-01	3.41	0.42		0.41	3.87	
65	43	42	102	110.	100	0.439E-01	1.32	0.16	LO	0.07	0.67	
66	44	43	102	150.	100	0.598E-01	2.24	0.27	LO	0.27	1.78	
67	44	45	102	54.	100	0.215E-01	1.70	0.21	LO	0.06	1.06	
68	46	45	102	47.	100	0.187E-01	1.04	0.13	LO	0.02	0.43	LO
69	47	46	102	58.	100	0.231E-01	0.11	0.01	LO	0.00	0.01	LO
70	47	48	102	85.	100	0.339E-01	0.00	0.00	LO	0.00	0.00	LO
71	49	48	102	160.	100	0.638E-01	3.15	0.38	LO	0.53	3.33	
72	50	47	102	156.	100	0.622E-01	3.19	0.39	LO	0.53	3.42	
73	51	46	102	158.	100	0.630E-01	3.91	0.48		0.79	4.99	
74	52	44	152	160.	100	0.886E-02	13.19	0.73		1.05	6.57	
75	53	43	102	164.	100	0.654E-01	3.69	0.45		0.73	4.47	
76	54	42	102	170.	100	0.678E-01	2.68	0.33	LO	0.42	2.47	
77	53	54	102	110.	100	0.439E-01	3.24	0.40	LO	0.39	3.52	
78	52	53	102	155.	100	0.618E-01	3.37	0.41		0.59	3.77	
79	52	51	102	100.	100	0.399E-01	2.98	0.36	LO	0.30	3.00	
80	51	50	102	55.	100	0.219E-01	3.75	0.46		0.25	4.61	
81	50	49	102	55.	100	0.219E-01	0.18	0.02	LO	0.00	0.02	LO
82	60	49	102	350.	100	0.140E 00	3.76	0.46		1.62	4.64	
83	58	51	102	110.	100	0.439E-01	5.25	0.64		0.94	8.58	
84	57	52	152	110.	100	0.609E-02	20.38	1.12		1.62	14.71	HI
85	56	53	102	110.	100	0.439E-01	4.49	0.55		0.71	6.44	
86	55	54	102	110.	100	0.439E-01	0.04	0.00	LO	0.00	0.00	LO
87	56	55	38	108.	90	0.622E 01	0.40	0.35	LO	1.13	10.46	HI
88	57	56	102	160.	100	0.638E-01	5.49	0.67		1.50	9.34	
89	57	58	102	100.	100	0.399E-01	5.62	0.69		0.97	9.75	
90	59	57	152	60.	100	0.332E-02	32.08	1.77		2.04	34.06	HI
92	61	60	102	1100.	100	0.439E 00	4.45	0.54		6.96	6.32	
93	64	61	152	1100.	100	0.609E-01	4.45	0.25	LO	0.97	0.88	

TABLE IV-E-3
NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	- - - - PRESSURE ATM	- - - CK	PCT DROP	- - - CK
1	1.5	-0.74	2.28U	0.78	0.08	LO	98.71	HI
2	1.5	-1.20	2.31U	0.81	0.08	LO	98.67	HI
3	1.5	-0.72	2.30U	0.80	0.08	LO	98.69	HI
4	1.5	-0.63	2.29U	0.79	0.08	LO	98.69	HI
5	1.5	-0.97	3.14U	1.64	0.16	LO	97.31	HI
6	1.5	-0.42	2.93U	1.43	0.14	LO	97.65	HI
7	1.5	-0.44	2.95U	1.45	0.14	LO	97.62	HI
8	1.5	-0.36	2.99U	1.49	0.14	LO	97.55	HI
9	1.5	-0.96	3.22U	1.72	0.17	LO	97.18	HI
10	1.5	-1.60	2.80U	1.30	0.13	LO	97.86	HI
11	1.5	-0.72	3.06U	1.56	0.15	LO	97.46	HI
12	1.5	-0.26	3.49U	1.99	0.19	LO	96.73	HI
13	1.5	-0.74	3.44U	1.94	0.19	LO	96.80	HI
14	1.5	-0.51	3.08U	1.58	0.15	LO	97.40	HI
15	1.5	-0.99	2.49U	0.99	0.10	LO	98.37	HI
16	1.5	-2.33	1.88U	0.38	0.04	LO	99.38	HI
17	1.5	-2.61	1.72U	0.22	0.02	LO	99.63	HI
18	1.5	-0.42	2.95U	1.45	0.14	LO	97.62	HI
19	1.5	-0.67	3.16U	1.66	0.16	LO	97.27	HI
20	1.5	-0.39	3.54U	2.04	0.20	LO	96.64	HI
21	1.5	-0.19	3.61U	2.11	0.20	LO	96.53	HI
22	1.5	-0.19	3.61U	2.11	0.20	LO	96.54	HI
23	1.5	-0.44	2.88U	1.38	0.13	LO	97.73	HI
24	1.5	-0.34	3.87U	2.37	0.23	LO	96.10	HI
25	1.5	-0.19	3.83U	2.33	0.23	LO	96.17	HI
26	1.5	-0.20	3.82U	2.32	0.22	LO	96.19	HI
27	1.5	-0.37	3.80U	2.30	0.22	LO	96.21	HI
28	1.5	-0.57	3.47U	1.97	0.19	LO	96.76	HI
29	1.5	-0.88	3.48U	1.98	0.19	LO	96.75	HI
30	1.5	-0.24	3.92U	2.42	0.23	LO	96.01	HI
31	1.5	-0.12	3.92U	2.42	0.23	LO	96.01	HI
32	1.5	-0.12	3.92U	2.42	0.23	LO	96.01	HI
33	1.5	-0.20	3.92U	2.42	0.23	LO	96.01	HI
34	1.5	-0.23	3.92U	2.42	0.23	LO	96.02	HI
35	1.5	-1.40	3.84U	2.34	0.23	LO	96.15	HI
36	1.5	-1.77	3.24U	1.74	0.17	LO	97.14	HI
37	1.5	-0.13	4.04U	2.54	0.25	LO	95.82	HI
38	1.5	-0.20	4.04U	2.54	0.25	LO	95.82	HI
39	1.5	-0.13	4.04U	2.54	0.25	LO	95.82	HI
40	1.5	-0.13	4.04U	2.54	0.25	LO	95.82	HI
41	1.5	-0.27	4.05U	2.55	0.25	LO	95.81	HI
42	1.5	-0.59	3.88U	2.38	0.23	LO	96.09	HI
43	1.5	-0.91	3.95U	2.45	0.24	LO	95.97	HI
44	1.5	-0.57	4.22U	2.72	0.26	LO	95.53	HI
45	1.5	-0.28	4.16U	2.66	0.26	LO	95.63	HI
46	1.5	-0.28	4.18U	2.68	0.26	LO	95.59	HI
47	1.5	-0.38	4.18U	2.68	0.26	LO	95.59	HI
48	1.5	-0.36	4.18U	2.68	0.26	LO	95.59	HI
49	1.5	-0.80	4.71U	3.21	0.31	LO	94.72	HI
50	1.5	-0.38	4.71U	3.21	0.31	LO	94.71	HI

TABLE IV-E-3 (Continued)

NODE DATA

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM	CK	PCT DROP	CK
51	1.5	-0.56	4.97U	3.47	0.34	LO	94.30	HI
52	1.5	-0.85	5.27U	3.77	0.36	LO	93.80	HI
53	1.5	-0.93	4.68U	3.18	0.31	LO	94.77	HI
54	1.5	-0.61	4.30U	2.80	0.27	LO	95.40	HI
55	1.5	-0.36	4.30U	2.80	0.27	LO	95.40	HI
56	1.5	-0.60	5.39U	3.89	0.38	LO	93.60	HI
57	1.5	-0.58	6.89U	5.39	0.52	LO	91.14	HI
58	1.5	-0.37	5.91U	4.41	0.43	LO	92.74	HI
59	3.0	32.08U	8.93	5.93	0.57	LO	90.00	HI
60	1.5	-0.68	6.34U	4.84	0.47	LO	92.04	HI
61	3.0	-0.0	13.29U	10.29	1.00		82.64	HI
64	3.0	4.45U	14.26	11.26	1.09		81.01	HI
65	1.5	-0.41	3.34U	1.84	0.18	LO	96.97	HI

APPENDIX TO CHAPTER VII

APPENDIX TABLE VII-A-1
WATER WELL DATA SUMMARY
OZAMIZ CITY-CLARIN AREA

Number	Location	Nominal Diameter (mm)	Depth (m) from Ground Surface: Total	Cased	SWL[1]	Test PWL[2]	Test Yield (lps)	Sp. Cap. (lps/m)	Date Completed	Remarks
MSO-1	Catadman, National Road, Ozamiz City	200	45	43	+1.2	-9.1	7.6		1967	Abandoned - flowing 0.41ps
MSO-2	Bo. Lapasan, Clarin	150	59	53	-	-	1.6	-	1954	Flowing
MSO-3	Catadman, Ozamiz City	100	40	34	+2.1	-			1961	Flowing rate decreased
MSO-4	Rizal St., Gango (1 km) from airport)	100	46	30	-	-	0.2		1959	Flowing
MSO-5	Near Cotta, Panguil Bay	100	38	34	+0.3	-3.0	2.7		1960	Abandoned
MSO-6	Bo. Malaubang	100	46	23	-4.0	-5.5	0.5		1961	Abandoned
MSO-7	Bo. Lipusong, Ozamiz City	100	117	49	-108.2?	-112.2?	0.8		1955	
MSO-8	Bo. Hinagdong, Ozamiz City	100	67	57	-56.7?	-57.3?	0.8		1963	
MSO-9	Opel Beach, Ozamiz City	100	46	38	+1.8	-	-	-	1961	Flowing
MSO-10	Misamis Annex, Ozamiz City	60	40	24	-	-	-	-	1971	Flowing
MSO-11	Bo. Maningcol, Ozamiz City	100	48	34	-	-	-	-	1960	Flowing
MSO-12	Bo. Cogon, Ozamiz City	100-150	56	26-10	-51.5?	-	-	-	1962	Abandoned
MSO-13	Bo. Gata, Clarin	150	48	25	+2.4	-	-	-	1956	No longer flowing
MSO-14	Lapasan District, Clarin	100	29	23	-	-	-	-	1958	
MSO-15	Public Elem. Sch., Clarin	100	46	29	-1.5	-	-	-	1961	

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

APPENDIX TABLE VII-A-1
WATER WELL DATA SUMMARY
OZAMIZ CITY-CLARIN AREA (continued)

Number	Location	Nominal Diameter (mm)	Depth (m) from Ground Surface Total	Cased	SWL	Test PWL	Test Yield (lps)	Sp. Cap. (lps/m)	Date Completed	Remarks
MSO-16	Poblacion, Clarin	100	47	45	-1.2	-	-	-	1960	Poor present yield
MSO-17	Gango Elem. Sch., Bo. Gango	100	39	32	-	-	-	-	1958	Flowing
MSO-18	Rizal St., Gango (150 m from National Road	100	43	23	-	-	-	-	1958	Flowing
MSO-19	Las Agudas Ext., Ozamiz City	100	35	34	-	-	-	-	1953	Flowing at high tide
MSO-20	Bo. Balintawak Elem. School	150	78	53	-	-	-	-	1954	Abandoned 1954
MSO-21	Baybay, Clarin Poblacion	100	46	35	+0.9	+0.3	0.3		1962	Flowing
MSO-22	High School Compound Clarin Poblacion	100	-	-	-	-	-	-	-	
MSO-23	Baybay, Clarin Poblacion	50	-	-	-	-	-	-	-	Flowing
MSO-24	Rizal St. at Pilar St., Clarin Poblacion	50	-	-	-	-	-	-	1954	Flowing
MSO-25	National Rd., South edge Clarin Poblacion	50	-	-	-	-	-	-	1971	Flowing
MSO-26	Gango, on road to airport, 500 m west Nat'l road	100	-	-	-	-	-	-	-	-
MSO-27	Mingrino Law Office, Catadman, Ozamiz City	50	-	-	-	-	-	-	-	-
MSO-28	Road to Gata, 250 m west National Road	50	-	-	-	-	-	-	-	Flowing

Note: All wells except MSO-23, 24, 25, 27 and 28 were drilled by NWASA.

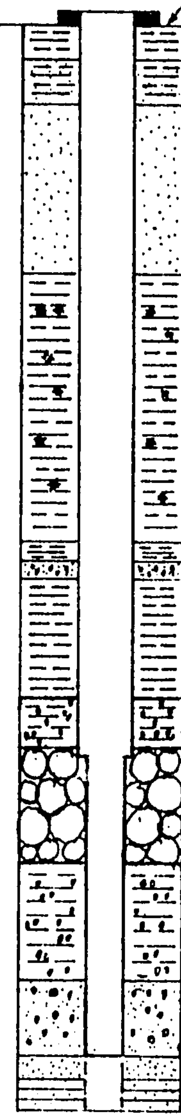
DESCRIPTIVE DATA

WELL NO. (CDM) MSO-1
(OTHER) NWASA 35-67-1
LOCATION CATADMAN, NORTH SIDE OF DON M. MARCOS AVE. 70M S.W. OF PROSPERIA PARK.
CITY OZAMIZ CITY
PROVINCE MISAMIS OCCIDENTAL
CONST. BY NWASA
DRILLER G.T. CHAVEZ
STARTED 30 AUGUST 1967
COMPLETED 29 SEPTEMBER 1967
OWNER MISAMIS OCCIDENTAL WATER DISTRICT
STATUS OUT OF SERVICE; PUMP REMOVED
CASING DIAMETER 200 MM; 150 MM

DRILLER'S TEST DATA:
DATE 29 SEPTEMBER 1967
STATIC WATER LEVEL 1.2 M ABOVE GROUND SURFACE
PUMPING WATER LEVEL 9.2 M BELOW GROUND SURFACE
TEST PUMP YIELD 7.6 LPS

REMARKS:
FLOWING ARTESIAN, DISCHARGING 6 GPM WHEN COMPLETED, 2 GPM ON 16 APRIL 1975.

GRAPHIC LOG



DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
1.8	6	BLUE STICKY CLAY
3.7	12	SANDY CLAY
10.7	35	COARSE SAND
22.0	72	BLUE STICKY CLAY WITH SHELLS
22.6	74	YELLOW STICKY CLAY
23.2	76	SAND AND GRAVEL
28.0	92	YELLOW STICKY CLAY
30.2	98	YELLOW STICKY CLAY WITH GRAVEL
34.8	114	BOULDER
39.6	130	YELLOW STICKY CLAY WITH GRAVEL
43.0	141	SAND & GRAVEL
43.6	143	FINE SAND
45.1	148	SANDSTONE

APPENDIX FIGURE VII-A-1
WELL DATA SHEET
WELL MSO-1

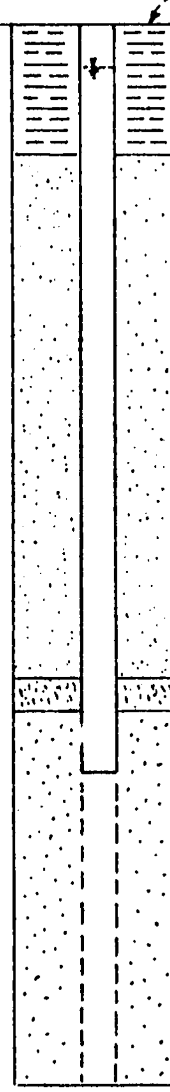
DESCRIPTIVE DATA

WELL NO. (CDM) MSO - 15
(OTHER) NWASA 34-61-27
LOCATION ELEMENTARY SCHOOL, ZULUETA ST. AT RIZAL ST., CLARIN POBLACION
CITY CLARIN
PROVINCE MISAMIS OCCIDENTAL
CONST. BY NWASA
DRILLER RIG. NO.236
STARTED
COMPLETED 22 MARCH 1961
OWNER
STATUS
CASING DIAMETER 100 MM

DRILLER'S TEST DATA:
DATE 22 MARCH 1961
STATIC WATER LEVEL 1.5 M
PUMPING WATER LEVEL
TEST PUMP YIELD 1.6 LPS

REMARKS:
DATA FROM NWASA RECORDS.

GRAPHIC LOG



DEPTH (M)	DEPTH (FT.)	STRATIFICATION
		GROUND SURFACE
		BROWN CLAY
4.9	16	
		SAND
25.3	83	
		QUICKSAND
26.5	87	
29.0	95	
		COARSE SAND
41.5	136	
		BOULDER

APPENDIX FIGURE VII-A-2
WELL DATA SHEET
WELL MSO - 15

DESCRIPTIVE DATA

WELL NO. (CDM) MSO-16
(OTHER) NWASA 34-60-55
LOCATION TEJUDU ST. AT NATIONAL ROAD
CLARIN POBLACION
CITY CLARIN
PROVINCE MISAMIS OCCIDENTAL
CONST. BY
DRILLER
STARTED
COMPLETED 6 JUNE 1960
OWNER

STATUS

CASING DIAMETER 100 MM

DRILLER'S TEST DATA:
DATE 6 JUNE 1960
STATIC WATER LEVEL 1.2 M

PUMPING WATER LEVEL

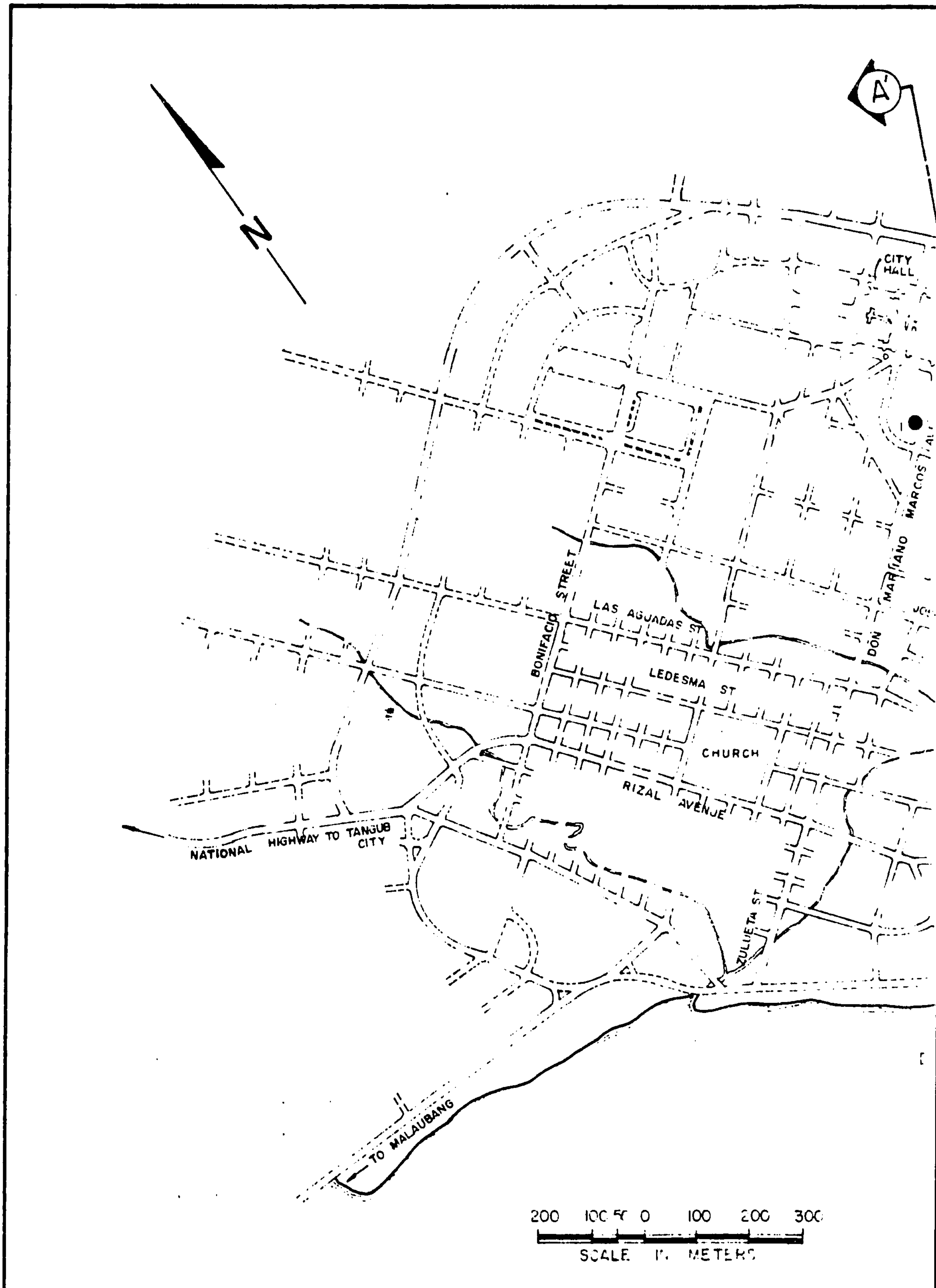
TEST PUMP YIELD

REMARKS:
DATA FROM NWASA RECORDS.

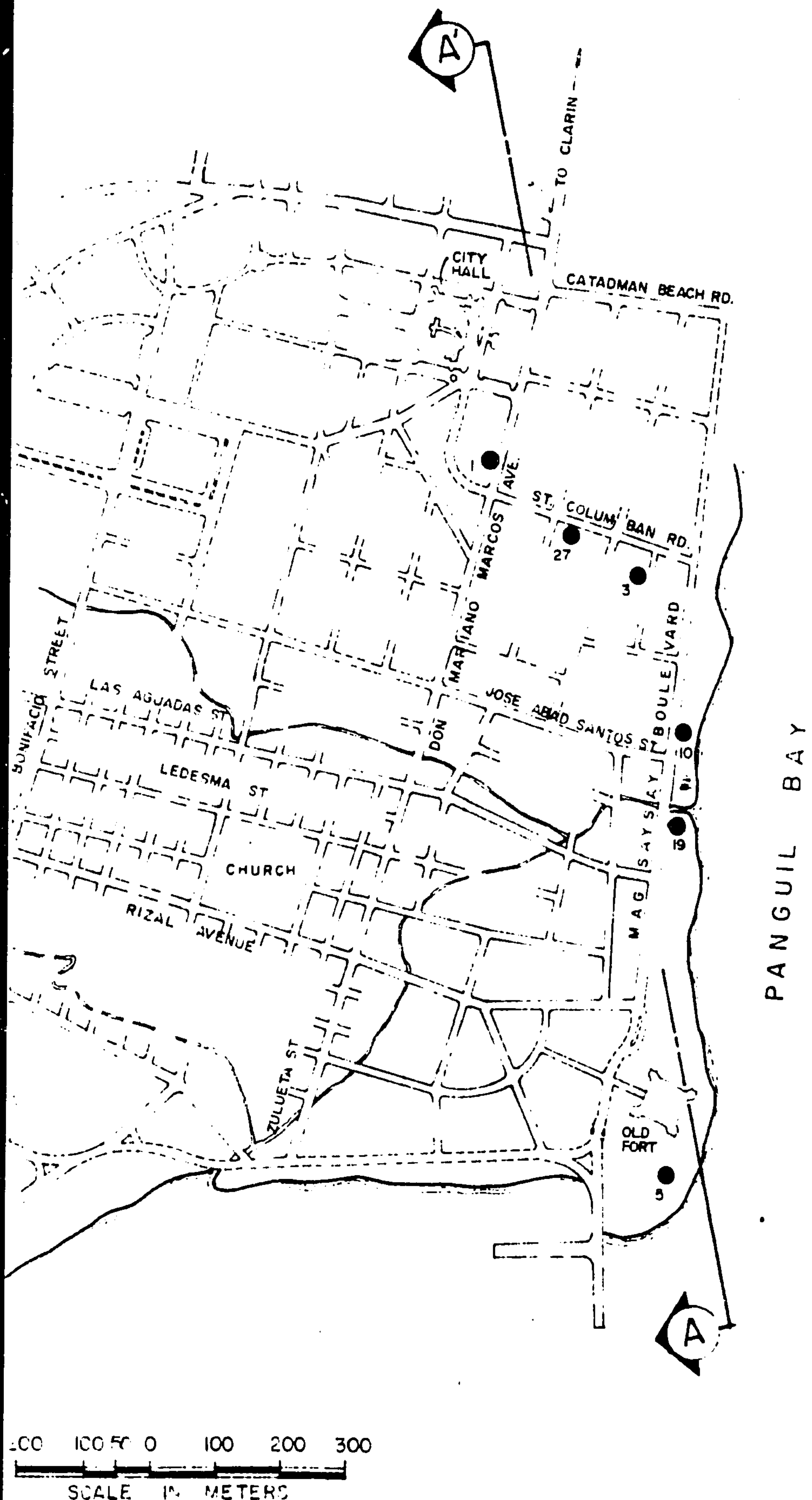
GRAPHIC LOG

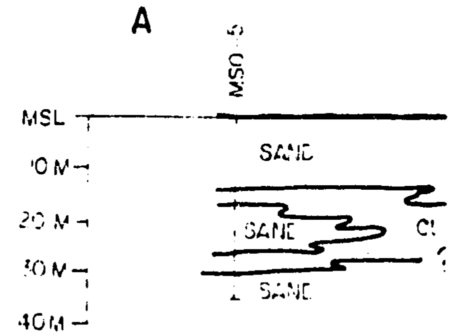
DEPTH (M)	DEPTH (FT.)	CASING	STRATIFICATION
			GROUND SURFACE
			SANDY CLAY
5.5	18		GRAVELLY CLAY
7.9	26		SAND
17.1	56		SAND WITH SHELLS
22.9	75		FINE SAND
27.4	90		SHALE CLAY WITH DECAYED WOOD
33.5	110		COARSE SAND AND GRAVEL WATER BEARING
42.1	138		POORLY CEMENTED SANDSTONE
44.8	147		
47.3	155		

APPENDIX FIGURE VII-A-3
WELL DATA SHEET
WELL MSO - 16


N
A'
CITY HALL
MARIANO MARCOS AVE
DON
BONIFACIO STREET
LAS AGUADAS ST
LEDESMA ST
CHURCH
RIZAL AVENUE
ZULUETA ST
NATIONAL HIGHWAY TO TANGUB CITY
TO MALAUBANG
200 100 50 0 100 200 300
SCALE IN METERS

a





GEO

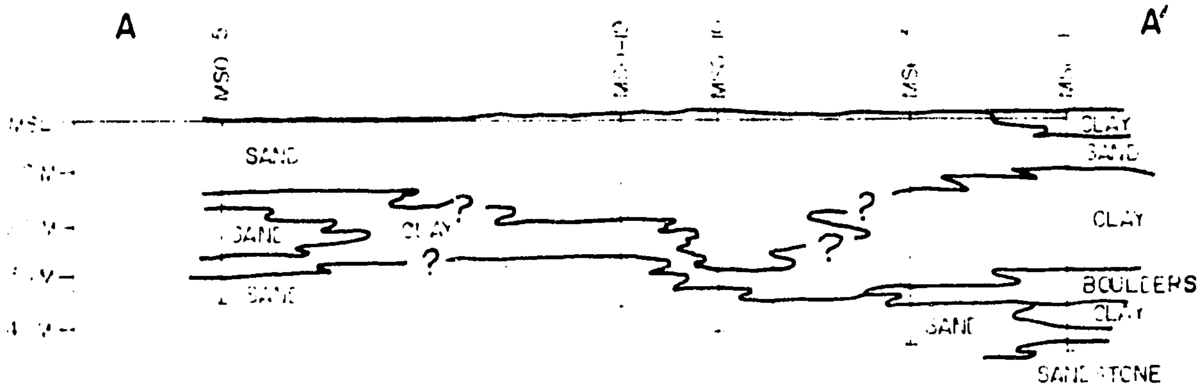
200

LEGEND:

5 SELECTED WE

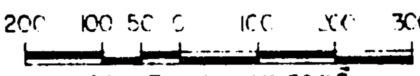
A' A GEOLOGIC CRO

b



GEOLOGIC SECTION A-A'

HORIZONTAL SCALE



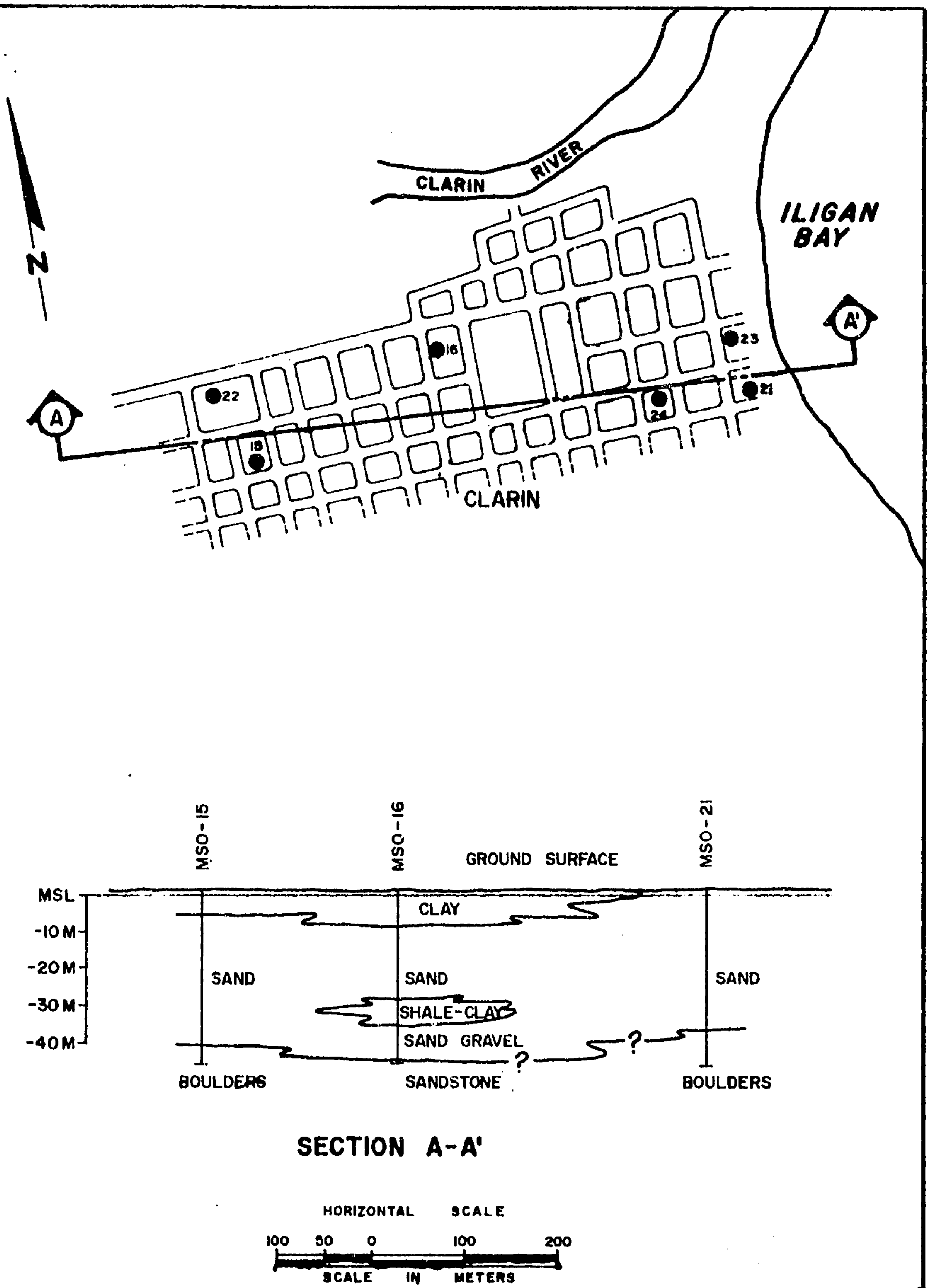
SCALE IN METERS

LEGEND:

5● SELECTED WELL, MSO-5

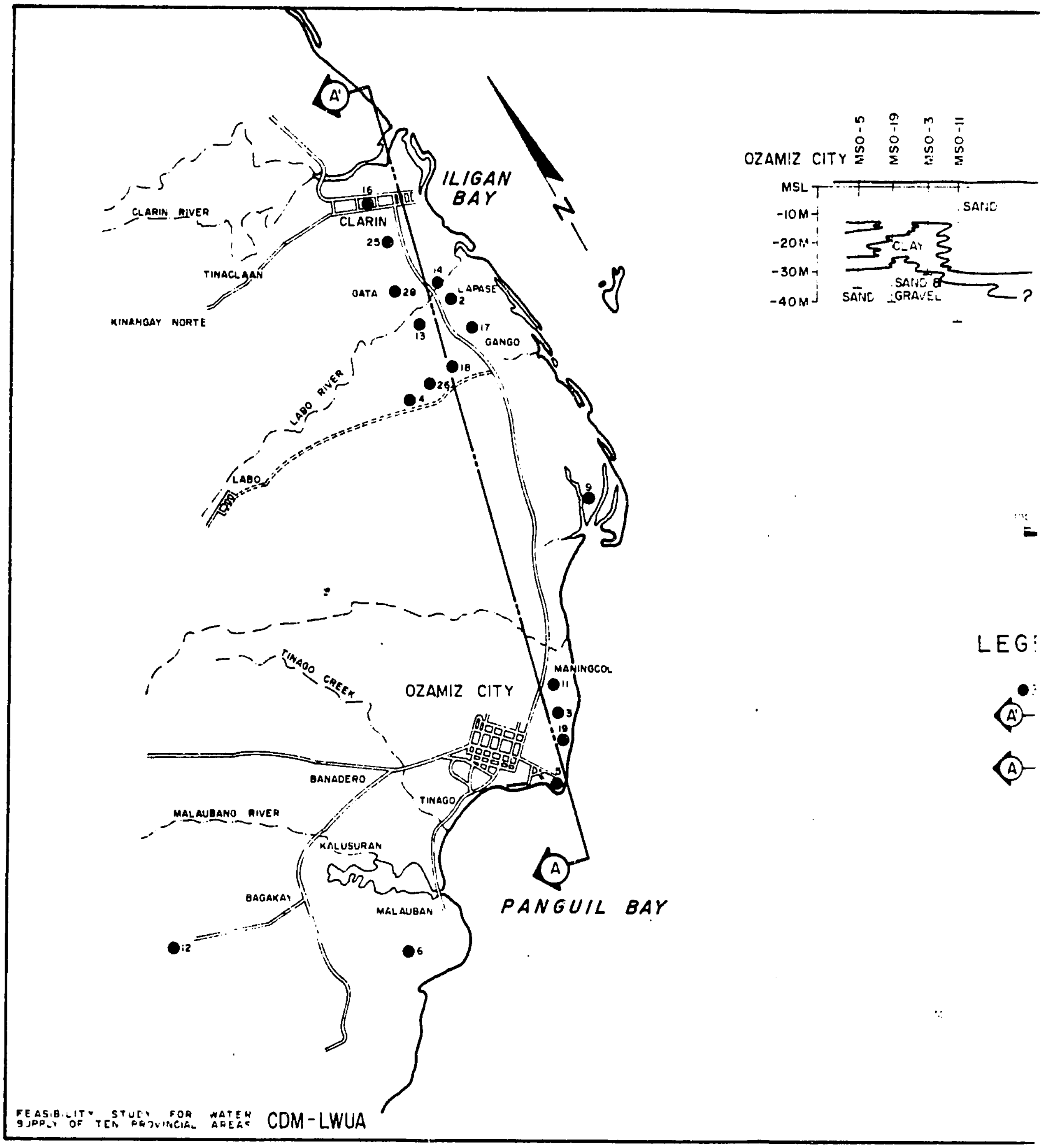
GEOLOGIC CROSS SECTION A-A'

APPENDIX FIGURE VII-A-4
WELL LOCATION MAP AND GEOLOGIC SECTION
OZAMIZ CITY



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

APPENDIX FIGURE VII-A-5
WELL LOCATION MAP AND GEOLOGIC SECTION
CLARIN


ILIGAN BAY
N
CLARIN RIVER
CLARIN
16
25
TINACLAAN
KINANGAY NORTE
GATA
28
14
LAPASE
2
17
GANGO
13
18
26
4
LABO RIVER
LABO
9
TINAGO CREEK
OZAMIZ CITY
MANINGCOL
11
3
19
5
BANADERO
TINAGO
MALAUBANG RIVER
KALUSURAN
BAGAKAY
MALAUBAN
12
6
A'
A
PANGUIL BAY
OZAMIZ CITY
MSO-5
MSO-19
MSO-3
MSO-11
MSL
-10M
-20M
-30M
-40M
SAND
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FEASIBILITY STUDY FOR WATER SUPPLY OF TEN PROVINCIAL AREAS
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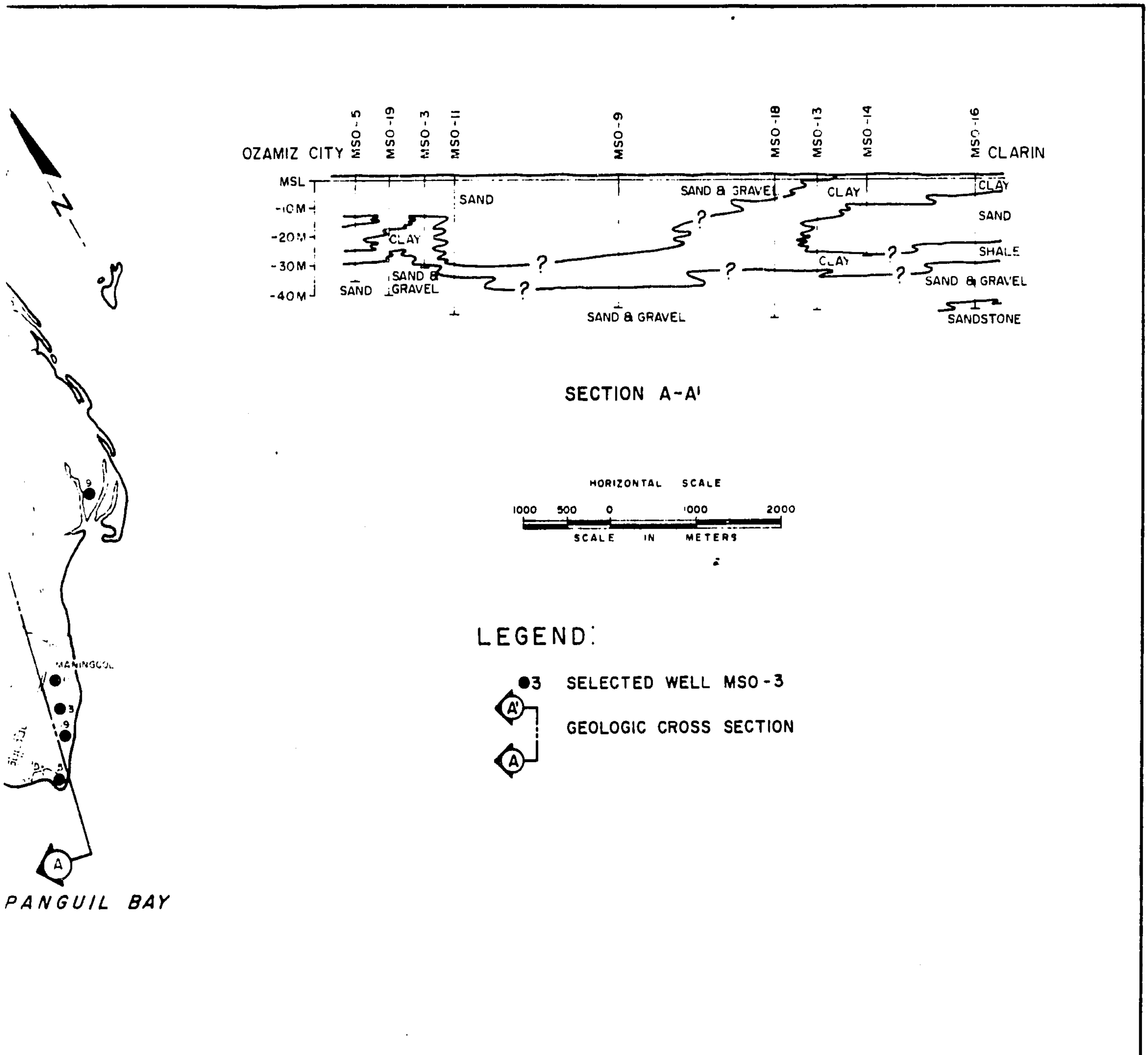


APPENDIX FIGURE VII A-6
WELL LOCATION MAP AND GEOLOGIC SECTION

APPENDIX TO CHAPTER VIII

APPENDIX VIII-C

WATER TREATMENT ALTERNATIVES

Analysis of water samples taken from the proposed spring sources indicates that all the physical and chemical quality parameters are within the acceptable limits, with the exception of slightly excessive iron (0.55 mg/l) in the Coook Spring water. Water from some of the existing wells also contains slightly excessive (0.85 mg/l) iron. Official records show the presence of water-borne diseases in the area.

Surface waters, in general, are of good quality. However, color, turbidity and bacterial removal will be necessary, particularly during rainy periods.

Feasibility of iron removal from the groundwater sources will be studied. Otherwise, disinfection of water by chlorination would be the only treatment required for the spring or well supplies. Alternative application points will be compared to obtain the most effective disinfection results.

Surface water would normally require provision of complete treatment facilities regardless of frequency of rainy periods. When water does not contain a high degree of color and turbidity, direct filtration followed by disinfection may be practiced.

<u>Disinfection Alternatives</u>

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 bacteriocidal 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 scarce. 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 serves 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 system conveys the water to the consumer. 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 that 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.

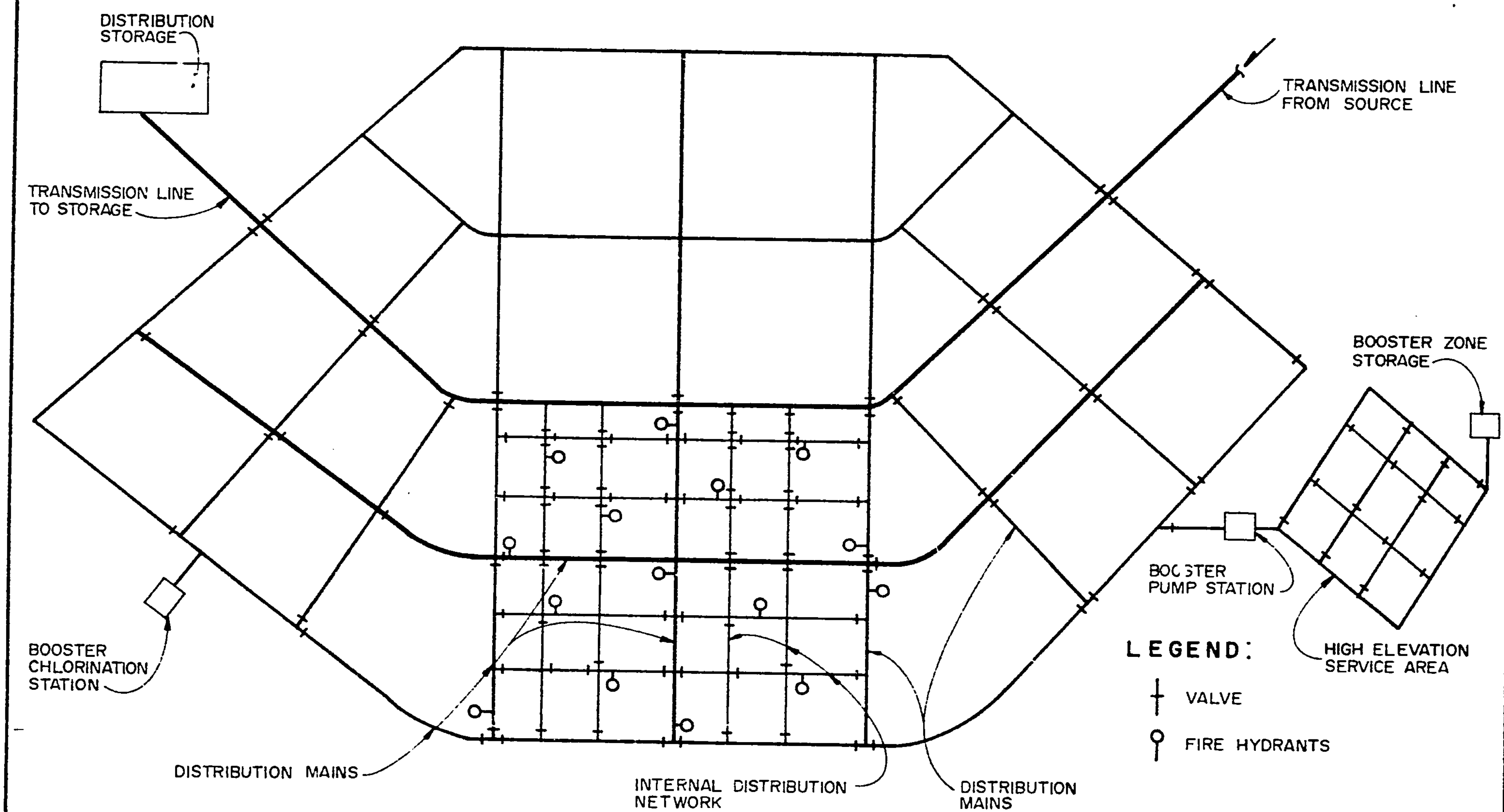
Each component of the distribution system and its respective alternatives are discussed in subsequent sections.

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 for



APPENDIX FIGURE VIII-D-1
COMPONENTS OF THE DISTRIBUTION SYSTEM
NOT TO SCALE

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

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

EVALUATION OF DISTRIBUTION MAIN STAGING

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

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

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

[2] Discount rate is 12 per cent.

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 when the tank is half-full. Because of this small head differential, care must be taken in choosing location and size of the supply lines.

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

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

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

Internal Network System

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

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

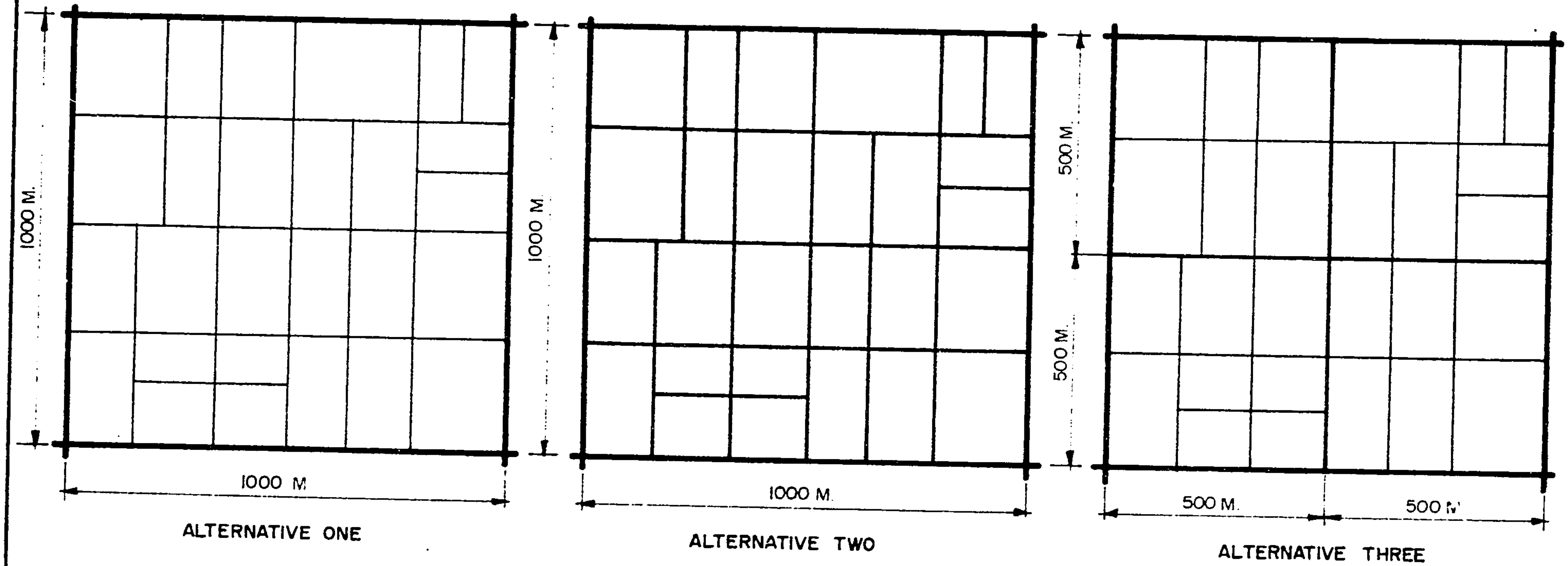
The internal network design is controlled by either of two conditions: peak hour demands with minimum main pressure of 10 m or fire flow demand coincident with maximum-day demands with a minimum hydrant pressure of 7 meters.

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

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

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

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



SIZE OF AREA = 100 HA
TOTAL LENGTH OF INTERNAL NETWORK 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

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

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 lpod, 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	Minimum Internal Network Pressure (m)[3] Population Density	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 subalternatives 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	11,600
100 mm Pipe	100	100:0	14,500
	120	120:0	17,400
2 - All	80	0:80	19,600
150 mm Pipe	100	0:100	24,500
	120	0:120	29,400
3 - Mixed	80	60:20	13,600
100-150 mm Pipe	100	80:20	16,500
	120	100:20	19,400

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

Alternative 3 is 10-15 per cent more than Alternative 1 and 45-50 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.

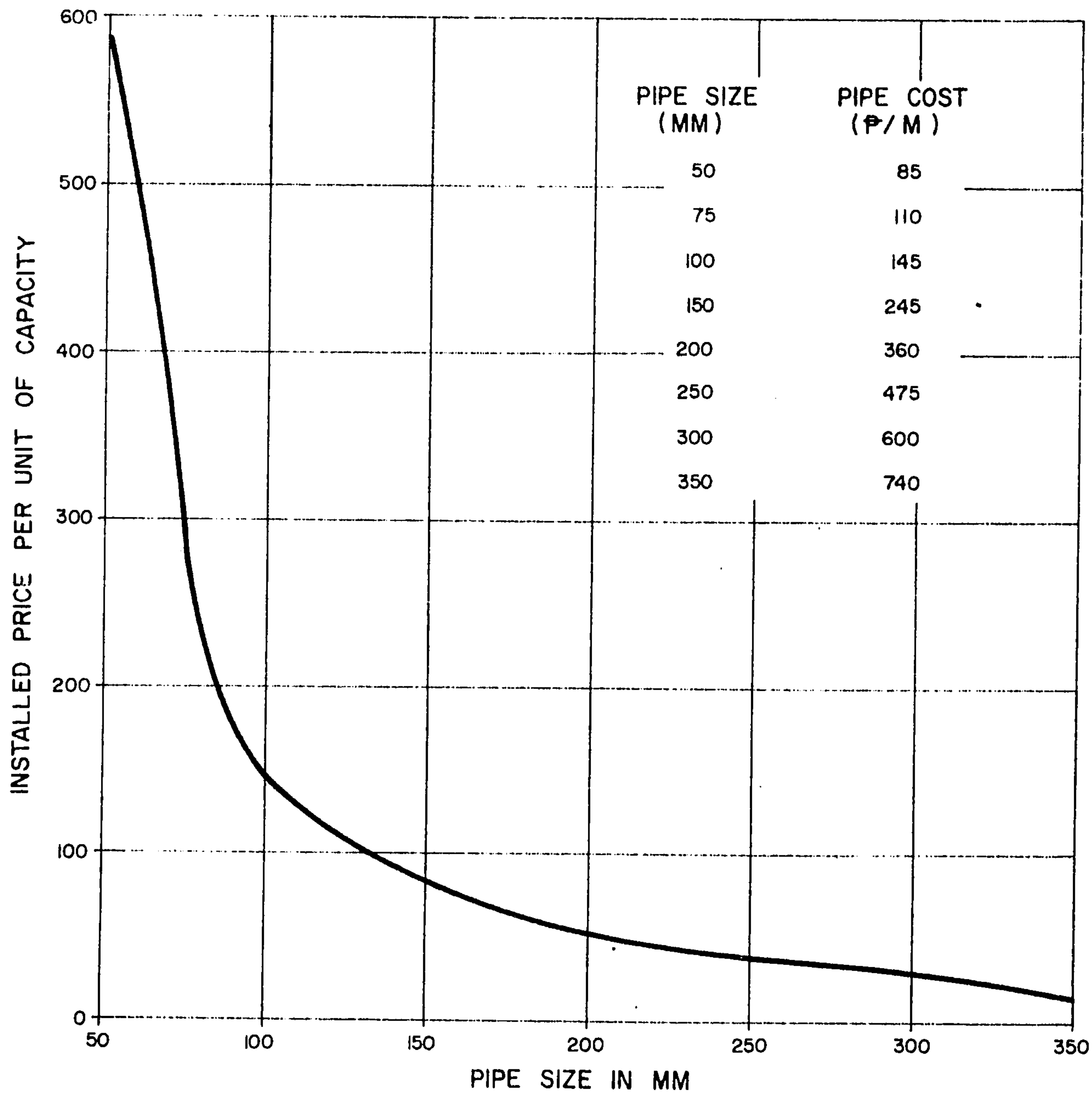
Fire hydrants will be placed at street intersections to permit quick location and enable fire-fighting in several directions. In high value areas, hydrants will be spaced a maximum of 150 m apart with two outlets - 1 x 60 mm hose outlet and a 1 x 100 mm pump outlet. The hydrant will be connected to a 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 powered to permit a minimum level of service during power shortages.



PIPE SIZE (MM)	PIPE COST (₱/M)
50	85
75	110
100	145
150	245
200	360
250	475
300	600
350	740

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

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

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 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.

Types and Costs of HPW

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.

High Water Level HPW

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 (2½ 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).

Deep Water Level HPW

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

The essential materials of such a well would include:

1. A pump, or more properly, a pump cylinder, which should be brass lined steel, of 57 mm (2¼ in) or 54 mm (2 3/16 in)

diameter. The cylinder should include the piston, of three-cup type, and the bottom valve assembly. The cylinder should connect on the top to 62 mm (2½ in) diameter pipe and on the bottom to a 62 mm (2½ in) diameter pipe.

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

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

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

5. A pump rod to connect the pump piston through the discharge head to the pump handle. If the rod is not more than about 12 m long it may be of 11 m (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

[7]Where groundwater conditions are favorable for HPW.

[8]Based on 7 persons per house.

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.

Additional Analyses of Storage Alternatives

In order to reduce transmission line costs associated with storage tanks described in Chapter VIII, additional storage tank alternative schemes were analyzed. These schemes are described below.

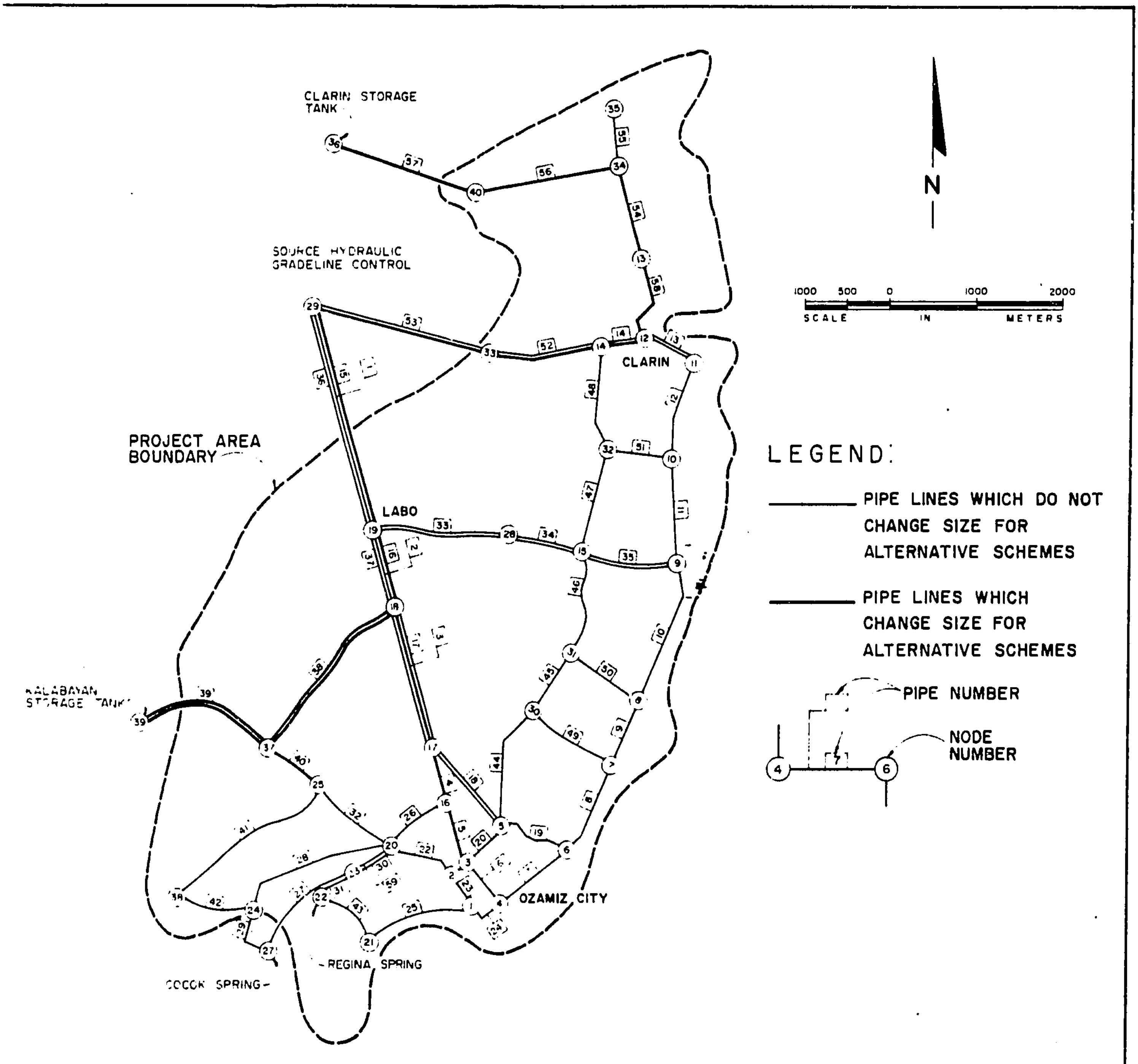
ST - 1. All storage in the distribution system. This is the alternative associated with source/transmission alternative 2 discussed in Section VIII-B. It consists of storage sites located in Barrio Kalabayan and northeast of Clarin (sites 1 and 3).

ST - 2. Stage I storage (1,900 cum) would be built along the source transmission line coming from Talibaksan and Bitoon Springs in Barrio Segatic. In 1989 the Stage II storage (1,600 cum) would be built at the Kalabayan Site and no storage would be built at the Clarin site. This alternative corresponds to 54 per cent of the storage being located at the source hydraulic gradeline control.

ST - 3. The Stage I storage plan is identical to ST-2 with 1,900 cum located at the source HGL control. In Stage 2,700 cum of storage would be constructed at the Kalabayan site in 1989 and 900 cum additional storage at the source HGL control in 1994-95. For ST-3 80 per cent of the storage would be located at the source HGL control.

ST - 4. For this alternative, no storage would be located in the distribution system. Consequently, 100 per cent would be located at the source HGL control.

The pipelines and storage tanks which were considered in the analysis of the storage alternatives are shown in Appendix Figure VIII-D-4. For each alternative, several computer runs were made to check peak-hour pressures and the storage filling condition. The required pipe sizes and construction period for each alternative are listed in Appendix Table VIII-D-2. The pipelines listed are only those which change with respect to the alternatives. Those lines which are the same for all alternatives are not listed in the table, though included in the analysis. The storage volumes and construction periods are listed as follows:



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

APPENDIX FIGURE VIII-D-4
EVALUATION OF ALTERNATIVE STORAGE SCHEMES
SCHEMATIC PLAN

Site	Construction Period	Storage Volume (cum) for Alternative ST-1	ST-2	ST-3	ST-4
Source HGL (Segatic)	1981-82		1,900	1,900	1,900
	1989-90				1,600
	1994-95			900	
Site 1 (Kalabayan)	1981-82	1,900	1,600	700	
	1989-90	900			
Site 3 (Clarin)	1994-95	700			

Each construction phase was assumed to be a 2-year period beginning in January of the first year and ending in December of the second year. Thus, all interest during construction was computed at 12 per cent for the 2-year period. Annual expenses were assumed to start at the end of the construction period.

The summary of present worth analysis of each alternative is listed in Appendix Table VIII-D-3. The tabulation of present worth cost includes the pipelines which are different in the alternative schemes, the storage tanks, and those pipelines and distribution mains which are identical for all alternatives.

From Appendix Table VIII-D-3, it can be seen that the more storage which is located along the source transmission line, the cheaper the alternative. An explanation for this is that the Ozamiz peak-hour supply comes from two directions - Coook and Regina Springs as well as Talibaksan, Bitoon, and Lower Dalingap Springs. Thus, Coook and Regina serve the same function as a storage tank by supplying peak-hour flow from a second direction. Though ST-4 is the least costly alternative, one significant drawback occurs because of no distribution storage, that is, peak-hour service along the Kalabayan Road cannot be maintained. The service area along the road to Barrio Kalabayan is at higher ground elevations (15-50 m) than Ozamis City, causing peak-hour pressures to be low. Thus, having no distribution storage in Kalabayan prevents the water district from supplying concessionaires in this area without booster pumpage.

Alternative ST-3, which costs 2 per cent more than ST-4 in the present worth analysis, would enable the Water District to serve the Kalabayan area without booster pumpage. At the same time the 700 cum of storage at the Kalabayan Site would make the service to Ozamiz City more reliable especially if a break occurs in one of the major transmission lines. The staging of storage in scheme ST-3 provides considerable flexibility to the water district in the period 1995-2000. Since 900 cum of storage would be constructed at that time, the Water District can place all of the 900 cum or part of it in any of three locations - Kalabayan site, source site, or Clarin site. The choice of the location for the storage should be based on demand patterns and urban growth projections which will be more readily discernible in 1995. Therefore, alternative storage scheme ST-3 is recommended. The Water District can supply a wider area with gravity service, and the storage staging provides more flexibility in the period 1995-2000.

- -

APPENDIX TABLE VIII-D-2

PIPE SIZES AND STORAGE VOLUMES
FOR ALTERNATIVE STORAGE SCHEMES

Pipe Number/ Storage Site	Stage/Construction Period	Length (m)	Pipe Size (mm) for Alternative ST-1	ST-2	ST-3	ST-4
	Stage I					
15	1980-81	2,650	450	250	250	250
16	1980-81	900	450	250	250	250
38	1982-83	2,250	450			
39	1982-83	1,850	450			
40	1982-83	750	250			
58	1986-87	1,100	150	150	150	150
15	1986-87	2,650		350	350	350
16	1986-87	900		350	350	350
	Stage II					
3	1989-90	1,650				250
4	1989-90	650				250
5	1989-90	750				250
15	1989-90	2,650	250			
16	1989-90	900	250			
38	1989-90	2,250	250	350	200	
39	1989-90	1,850	250	350	250	
40	1989-90	750		250	200	
53	1994-95	2,150	350	200	200	200
52	1994-95	1,350	350	200	200	200
14	1994-95	500	350	150	150	150
58	1994-95	1,100	350			
54	1994-95	1,100	350	150	150	150
56	1994-95	1,800	350			
57	1994-95	1,700	350			
38	1994-95	1,850	300	300	200	
39	1994-95	2,250	300	300	200	

APPENDIX TABLE VIII-D-3

SUMMARY OF PRESENT WORTH COSTS FOR ALTERNATIVE STORAGE SCHEMES

Alternative	Per Cent of Year 2000 Storage Located at Source Hydraulic Gradeline Control	Present Worth of Net Disbursement (₱)			Total
		Transmission Lines in Appendix Table VIII-D-2	Storage Tanks	Transmission Lines and Distribution Mains[9]	
ST-1	0	₱ 8,831,000	₱ 832,000	₱ 9,601,000	₱ 19,264,000
ST-2	54	5,073,000	870,000	9,601,000	15,544,000
ST-3	80	4,534,000	826,000	9,601,000	14,961,000
ST-4	100	4,193,000	856,000	9,601,000	14,650,000

[9] Pipelines which are the same for all alternatives.

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 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. 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 sewage 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 surface 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 in operation throughout the world, with a combined capacity of 227,260 cumd. Kuwait has the largest plant with a 113,700 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 5 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. These particles used in the method are called silver iodide crystals.

The costs of cloud seeding in 1971 ranged from $1.00 to $2.30 per acre-foot 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 than can reduce usable water yield.

Land Management

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

Two potential techniques of land management that apply to Philippine conditions are forest management and control of streambank vegetation. Forests impact upon water supply in a number of ways. They intercept rain from 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 deforestation, will affect water supply.

Phreatophytes or deep-rooted vegetation along the banks of canals and rivers consumes much water in their growth. Especially in cases as when 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 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 effectivity. For example, in Hongkong during the severe drought of summer 1963, water service was rationed into the various city sectors 4 hours every 4 days. Extraneous uses of water such as toilet flushing were therefore severely curtailed. In some of the high-rise government housing in Hongkong, dual plumbing system has been used, with sea water for toilet flushing.

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

If an existing system is to be replaced entirely by a dual system, the cost may well be unreasonable. However, if the existing water supply piping is retained 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 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 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 pressure otherwise required in a system with high fire demand.

APPENDIX TO CHAPTER IX

APPENDIX IX-C

DISTRIBUTION SYSTEM GROWTH

General

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

Population Served

The population served projections given in Chapter VI are divided into that of the present service area, 1990 service extension, and year 2000 service area extension. These areas were further divided into sub-sectors for apportioning the population served projections. The population served estimates for each sub-sector are given in Appendix Table IX-C-1. The present service area population in Appendix Table IX-C-1 was taken directly from Table VI-3. The method of apportioning the population served in Item B was first project the total population of the Clarin poblacion and then apply the same percentage served factor which applies to the present service area (Ozamiz City Poblacion). The Clarin Poblacion population was assumed to be 11% of the 1990 service area population in 1980, 10% in 1990 and 9% in year 2000. This reflects a trend of decreasing rate of growth in Clarin as compared to the increasing population growth of the area outside the poblacion in Ozamiz City. After determining the population served for Clarin, Items B-1 and B-3 (Table IX-C-1) were obtained by assigning approximately 75% of the remaining population served in the 1990 service area extension to Item B-1. The method is shown for 1980 below:

Population of Clarin Poblacion = .11 x 38,500 = 4,200

B - 2. Population Served in Clarin Poblacion = .80 x 4,200 = 3,400

Remaining Population Served = 7,700 - 3,400 = 4,300

B - 1. Population Served in Ozamiz City outside the Poblacion = .75 x 4,300 = 3,300

APPENDIX TABLE IX-C-1
POPULATION SERVED APPORTIONMENT
TO SUB-SECTORS

Service Area/Sub-Sector	Population Served in Year 1975	1980	1990	2000
A. Present Service Area				
1. Ozamiz City-Poblacion	17,500	19,200	27,200	35,100
B. 1990 Service Area Extension				
1. Ozamiz City-Outside Poblacion		3,300	13,400	30,100
2. Clarin-Poblacion		3,400	4,900	6,400
3. Clarin-Outside Poblacion		1,000	4,500	10,000
C. Year 2000 Service Area Extension				
1. Ozamiz City-Outside Poblacion			3,000	7,900
2. Clarin-Outside Poblacion				3,500
Total Population Served	17,500	26,900	53,000	93,000

B - 3. Population Served in Clarin

Outside the Poblacion = 4,300 - 3,300 = 1,000

For Item C.1 the 1990 population served was assumed to be entirely in the Ozamiz City area. For year 2000, 70% of the served population in the year 2000 service area extension was assumed to be in Ozamiz City.

Number of Consumers Served per Connection

The present average number of consumers per connection in Ozamiz City Poblacion is 10.8 according to interviews with concessionaires. Over the next 25 years, this number is assumed to decrease gradually because of (1) decreasing population growth which will reduce the number of people 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 projected average number of consumers served per connection is shown in Appendix Table IX-C-2.

APPENDIX TABLE IX-C-2

PROJECTION OF NUMBER OF CONSUMERS SERVED PER CONNECTION

Service Area/Sub-Sector	Number of Consumers Served per Connection 1975	1980	1990	2000
A. Present Service Area				
1. Ozamiz City-Poblacion	10.8	10	9	8
B. 1990 Service Area Extension				
1. Ozamiz City-Outside Poblacion		8	7.5	7
2. Clarin-Poblacion		10	9	8
3. Clarin-Outside Poblacion		8	7.5	7
C. Year 2000 Service Area Extension				
1. Ozamiz City-Outside Poblacion			7.5	7
2. Clarin-Outside Poblacion				7

The number of consumers per connection in the poblacion of Clarin is assumed the same as that for Ozamiz City Poblacion. The concessionaires outside the poblaciones are assumed to have slightly less consumers per connection than those in the poblaciones.

Number of Connections Per Hectare

Projecting the number of concessionaires to be connected per hectare of area served enables the determination of the total number of hectares served. At present the MOWD serves approximately 165 hectares in and around the poblacion of Ozamiz City. There are 1,624 concessionaires or approximately an average of 10 connections per hectare. In the projections of number of hectares served in the poblacion of Ozamiz for the future, the total area remains constant and the number of connections per hectare increases. By year 2000 the number of connections per hectare will be approximately 27. A large portion of the increasing connections per hectare can be attributed to the likely construction of multi-family dwellings.

To determine the area to be served in the Clarin Poblacion the number of connections per hectare is assumed. For example, it is assumed that initial service to the poblacion of Clarin will obtain an average of 8 connections per hectare. Thus the number of hectares in 1980 is calculated as follows:

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

$$= \frac{3400}{10 \times 8} = 42.5 \text{ ha}$$

The 42.5 ha represents the net area served. This area should be increased by 20% to reflect the land which will be used for schools, churches, and other institutions. Thus the total area served in the poblacion of Clarin is approximately 50 hectares. As in the case of Ozamiz City, the area of the poblacion of Clarin is kept constant and the number of connections increases as the population increases. Thus in 1990 there will be 12.7 connections per hectare (4,900/9 x 43) and in year 2000, 18.6 connections per hectare (6,400/8 x 43).

For the areas outside the poblaciones, it is assumed that the connections per hectare increase as the fringe areas around the poblacion of Ozamiz City and the poblacion of Clarin become more densely settled. In Ozamiz City the connections per hectare in the 1990 service area extension are assumed to be less than that for the present service area in the poblacion and more than that for year 2000 service area extension. The same trend is maintained for the sub-sectors in the Municipality of Clarin though the number of connections per hectare is slightly less than those in the corresponding sub-sector of Ozamiz City.

The projection of area served is listed in Appendix Table IX-C-3. The service area is projected to increase from the present 165 ha to 290 ha by 1980, 510 ha by 1990, and 750 ha by year 2000.

Area Served by Internal Network System

The present area being served by the internal network system in Ozamiz City is approximately 70 ha which is less than the total service area of 165 ha. The reason for this difference is that many concessionaires are connected directly to the transmission lines and distribution mains. This practice enables the Water District to serve more concessionaires than if no connections were permitted to the transmission and distribution mains. Thus, in Ozamiz approximately 95 ha is served in this manner. It is assumed that this practice will continue, and, therefore, the area served by distribution mains and transmission mains is subtracted from the total service area to determine the area which will receive internal network piping. It is estimated that 25 m can be served on each side of transmission lines and distribution mains. No service area is attributed to those transmission lines passing through areas of very low density population. The areas to be served by transmission lines and distribution mains are listed below:

	1980	1985	1990	2000
Length (m) of transmission and distribution mains likely to support concessionaires	10,000	6,000	4,000	12,000
Corresponding area (ha)	50	30	20	60

All of the present area of 95 ha served by transmission lines and distribution mains is not served adequately. It is assumed that as improvements are made, the 95 ha will be reduced to 30 ha and that 65 ha will be served with internal network. It is estimated that 60 ha currently served by internal network in the poblacion will require reinforcment to improve the level of service in the high-value commercial area.

The expansion of the service area and the area served by internal network system is listed in Appendix Table IX-C-4. By 1990, the MOWD (Ozamiz and Clarin) will have extended the internal network system to an additional 220 ha and reinforced the 60 ha of the existing system. By year 2000, 490 ha will have internal network piping. 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.

APPENDIX TABLE IX-C-3
PROJECTION OF AREA SERVED

Service Area/Sub-Sector	1980			1990			2000		
	No. Connections Per Hectare	Area Served (ha) Net	Gross	No. Connections Per Hectare	Area Served (ha) Net	Gross	No. Connections Per Hectare	Area Served (ha) Net	Gross
A. Present Service Area									
1. Ozamiz City - Poblacion	11.6		165	18.3		165	26.6		165
B. 1990 Service Area Extension									
1. Ozamiz City - Outside Poblacion	9	46	55	12	150	180	18	220	260
2. Clarin - Poblacion	8	43	50	12.7	43	50	18.6	43	50
3. Clarin - Outside Poblacion	7	18	20	10	60	70	15	90	110
C. Year 2000 Service Area Extension									
1. Ozamiz City - Outside Poblacion				11	36	45	12	90	110
2. Clarin - Outside Poblacion			—			—	10	45	55
Total =			290			510			750

APPENDIX TABLE IX-C-4
PROJECTED AREA SERVED BY INTERNAL NETWORK SYSTEM

	Area (ha) Served in 1975	1980	1985	1990	2000
A. Area Served by Transmission and Distribution Mains					
1. Existing Mains	95	60	30	30	30
2. New Mains		50	80	100	160
B. Area Served by Internal Network System					
1. Existing System	70	70	70	70	70
2. New System		110	220	310	490
C. Total Service Area	165	290	400	510	750

The first three periods are 4-year intervals, the last of which ends in 1990. The final two periods are 5-year intervals with the last one ending in year 2000. The construction program for the internal network is modified slightly from that shown in Appendix Table IX-C-4 in order to have contant level of growth. The recommended program is listed below:

Construction Period	Area of Internal Network Installed (ha) Reinforcement	New Service Area
I. First Stage		
A. 1978-82	30	100
B. 1982-86	30	100
C. 1986-90	—	100
Sub-total	60	300
II. Second Stage		
A. 1990-95		95
B. 1995-2000		95
Sub-total		190
Grand Total	60	490

Since the Interim Report, the cost of internal network piping has been re-evaluated. The reinforcement of the internal network in the poblacion has been reduced in part because of field observations which found larger pipe sizes than were recorded earlier. For internal network in new areas, the ratio of total length of 100 mm pipe to 150 mm pipe per hectare served was increased from 1.85 to 4.0 per hectare. Computer studies of internal network indicate that this ratio of 100 mm to 150 mm pipe per hectare can maintain both peak-hour and fire-flow requirements. The data listed in Table VIII-3 are used to compute the construction cost of the internal network.

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 Phases A and B of Stage I (1978-86) only the poblacion area be provided fire hydrants. This will correspond to the 60 ha of the existing internal network which will be reinforced. Since this is a commercial and high-value area, the hydrants will have 150 mm risers and be spaced at 150 m intervals. This can be accomplished at a construction cost of ₱5,000 per hectare.

The areas outside the poblacion will receive fire protection at later stages. The extension of fire protection will gradually increase, so that by Stage II-B the installation of hydrants will correspond with the construction of the internal network. The construction cost of hydrants in the less dense and lower valued residential areas is ₱1,600/hectare. Also, in Stage II - Phase B allowance is made for upgrading fire protection in 50 ha of formerly low-value residential area to higher-value area. The schedule for fire hydrant construction is listed below:

	Area to Receive Fire Protection (ha)	
Construction Period	High-Value Area	Residential Area
I. First Stage		
A. 1978-82	30	
B. 1982-86	30	
C. 1986-1990		100
II. Second Stage		
A. 1990-1995		165
B. 1995-2000	50[1]	165

[1] Corresponds to upgrading of areas provided with residential fire protection in earlier construction periods.

Number of Connections

The projection of the number of connections is obtained by dividing the population served in the sub-sectors by the average number of consumers per connection. The number of connections projected for each sub-sectors is listed in Appendix Table IX-C-5.

APPENDIX TABLE IX-C-5

PROJECTION OF NUMBER OF CONNECTIONS

Service Area/Sub-Sector	Number of Connections[2] in 1975	1980	1990	2000
A. Present Service Area				
1. Ozamiz City-Poblacion	1,624	1,920	3,022	4,388
B. 1990 Service Area Extension				
1. Ozamiz City-Outside Poblacion		412	1,787	4,300
2. Clarin-Poblacion		340	544	800
3. Clarin-Outside Poblacion		125	600	1,429
C. Year 2000 Service Area Extension				
1. Ozamiz City-Outside Poblacion			400	1,129
2. Clarin-Outside Poblacion				500
Total (Rounded)	1,624	2,800	6,350	12,550

The number of connections are projected to increase from 1,624 in 1975 to 6,350 in 1990 and 12,550 by year 2000.

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[2]Data were obtained by dividing the population served listed in Appendix Table IX-C-1 by the number of consumers per connection from Appendix Table IX-C-2.

It is necessary to project the number of connections between 1975 and 1978, the first year of the proposed First Stage improvements program. It is anticipated that the program to reduce unaccounted-for-water (a part of the Early Action Guidelines) will make water available for consumption by new concessionaires. The total number of connections by 1978 is estimated to be 1,850 corresponding to approximately six connections per month over the next three years. This is within the realm of possibility for water district. Simply eliminating the estimated wastage at flat rate connections (Item 2.c of Table IV-5) will make available 410 cumd, which would be enough water for 380 concessionaires at the present level of consumption and estimated number of people per connection.

From 1978 to 1990 the number of connections is projected to increase at a constant rate during the three construction periods. A constant rate is also projected for the two construction periods from 1990-2000. The projections are listed as follows:

Construction Period	Number of Connections Per Construction Period	Total Number of Connections[3]
I. First Stage		
A. 1978-82	1,500	3,350
B. 1982-86	1,500	4,850
C. 1986-90	1,500	6,350
II. Second Stage		
A. 1990-95	3,100	9,450
B. 1995-2000	3,100	12,550

The cost of connections will be shared between the water district and the concessionaire. The cost of a 12 mm service connection is ₱366 based on 1976 cost estimates. The water district will pay one-third of the service connection cost and the concessionaire will pay two-thirds of the cost. The concessionaire will also pay for the ₱140 water meter (12 mm meter). The service connection costs are itemized below:

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[3]Includes 1,850 connections in 1978.

	Itemized Cost (₱)
A. Service Connection Line	
1. Concessionaire	244
2. Water District	122
B. Water Meter	
1. Concessionaire	140
Total	506

The foreign exchange component of the service connection is assumed to be 80 per cent of the cost of the meter - US$16.

Existing Service Connections

As described in Chapters IV and VI, many existing GS service lines are leaking and the stop-gap method of repair will not alleviate the problem. It is anticipated that the existing service lines will leak even more when the new improvements raise the pressures in the existing service area. Therefore, a schedule for replacement for the existing service lines should be included in the improvement program so that leakage in the existing system can be reduced. During the first two phases of the First Stage improvements the program should be concentrated in those areas where the fewest number of replaced service lines will reduce significantly the leakage. These areas should be identified in the leakage survey recommended in the Early Action Guidelines. The schedule of replacement is projected as follows:

Construction Period	Number of Existing Connections Replaced Per Construction Period	Equivalent Annual Replacement
I. First Stage		
A. 1978-82	200	50
B. 1982-86	300	75
C. 1986-90	400	100
II. Second Stage		
A. 1990-95	724	145

The replacement program will be completed by 1995. The costs of the service line replacement are assumed to be the same as those for new service line installation described in the previous section. The costs of meters is not included since they are included in the meter replacement schedules in the financial feasibility analysis.

Summary

The recommended improvement program for the distribution system has been presented in this section. For each component of the distribution system the recommended schedule of improvements and estimated costs has been itemized. The estimated construction costs for all construction periods are listed in Appendix Table IX-C-6.

APPENDIX TABLE IX-C-6

SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period		Item/Description	Quantity	Unit Cost	Construction Cost (₱)	F.E.C. (US$)
I. First Stage						
A.	1978-82	Internal Network Reinforcement	30 ha	₱16,600/ha	498,000	13,300
		New Service Area	100 ha	₱17,700/ha	1,770,000	52,200
		Fire Hydrants				
		High-Value Area	30 ha	₱ 5,000/ha	150,000	
		Service Connections				
		Replacement	200	₱366	73,200	
		New Connections	1,500	₱506	759,000	24,000
		Sub-total (Rounded)			3,250,000	90,000
B.	1982-86	Internal Network				
		Reinforcement	30 ha	₱16,600/ha	498,000	13,300
		New Service Area	100 ha	₱17,700/ha	1,770,000	52,200
		Fire Hydrants				
		High-Value Area	30 ha	₱ 5,000/ha	150,000	
		Service Connections				
		Replacement	300	₱366	110,000	
		New Connections	1,500	₱506	759,000	24,000
		Sub-total (Rounded)			3,287,000	90,000
C.	1986-90	Internal Network				
		New Service Area	100 ha	₱17,700/ha	1,770,000	52,200
		Fire Hydrants				
		Residential Area	100 ha	₱ 1,600/ha	160,000	
		Service Connections				
		Replacement	400	₱366	146,400	
		New Connections	1,500	₱506	759,000	24,000
		Sub-total (Rounded)			2,835,000	76,000
		Grand Total			9,372,000	256,000

APPENDIX TABLE IX-C-6 (Continued)

SUMMARY OF DISTRIBUTION SYSTEM COSTS

Construction Period	Item/Description	Quantity	Unit Cost	Construction Cost (₱)	F.E.C. (US$)
II. Second Stage					
A. 1990-95	Internal Network				
	New Service Area	95 ha	₱17,700/ha	1,681,500	49,600
	Fire Hydrants				
	Residential Area	165 ha	₱ 1,600/ha	264,000	
	Service Connections				
	Replacement	724	₱366	265,000	
	New Connections	3,100	₱506	1,568,600	49,600
	Sub-total (Rounded)			3,880,000	99,000
B. 1995-2000					
	Internal Network				
	New Service Area	95 ha	₱17,700/ha	1,681,500	49,600
	Fire Hydrants				
	High-Value Area	50 ha	₱ 5,000/ha	250,000	
	Residential Area	165 ha	₱ 1,600/ha	264,000	
	Service Connections				
	New Connections	3,100	₱506	1,568,600	49,600
	Sub-Total (Rounded)			3,764,000	99,000
	Grand Total			7,644,000	198,000

APPENDIX TABLE IX-(Cont)

PEAK HOUR CONDITION

MISAMIS OCCIDENTAL FUTURE TRANSMISSION LINES AND FEEDER MAINS YR 2000

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INPUT AND OUTPUT IN               LPS
NO OF NODES                        37
NO OF PIPES                        57
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           292.95
BANDWIDTH                           7
ITER  1 UNBAL   93.23 LPS
ITER  2 UNBAL   50.50 LPS
ITER  3 UNBAL   23.93 LPS
ITER  4 UNBAL    7.68 LPS
ITER  5 UNBAL    0.94 LPS
ITER  6 UNBAL    0.04 LPS

SOLUTION NO. 1 REACHED IN   6 ITERATIONS
      0.0403     UNBALANCE
```

PIPE NO	NODES FROM	NODES TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS-- MT	MT/1000	CK
1	29	19	200	2650.	110	0.327E-01	16.36	0.52		5.79	2.19	
2	19	18	200	900.	110	0.111E-01	11.38	0.36	LO	1.00	1.12	
3	18	17	200	1650.	110	0.204E-01	39.82	1.27		18.71	11.34	HI
4	17	16	200	650.	110	0.803E-02	41.75	1.33		8.05	12.38	HI
5	16	3	200	750.	110	0.926E-02	38.33	1.22		7.92	10.57	HI
6	3	4	150	590.	100	0.353E-01	14.51	0.82		5.00	8.47	
7	6	4	200	920.	110	0.114E-01	18.26	0.58		2.46	2.68	
8	6	7	150	1100.	100	0.658E-01	7.05	0.40	LO	2.44	2.22	
9	7	8	150	800.	100	0.478E-01	2.52	0.14	LO	0.26	0.33	LO
10	9	8	150	1700.	100	0.102E 00	2.52	0.14	LO	0.56	0.33	LO
11	9	10	150	1200.	100	0.718E-01	2.00	0.11	LO	0.26	0.22	LO
12	11	10	150	1200.	100	0.718E-01	4.83	0.27	LO	1.32	1.10	
13	12	11	195	700.	100	0.117E-01	17.69	0.59		2.38	3.41	
14	14	12	195	500.	100	0.833E-02	41.99	1.41		8.44	16.89	HI
15	29	19	395	2650.	120	0.101E-02	106.92	0.87		5.79	2.19	
16	19	18	395	900.	120	0.344E-03	74.36	0.61		1.00	1.12	
17	18	17	261	1650.	110	0.557E-02	80.19	1.50		18.71	11.34	HI
18	17	5	261	1200.	110	0.405E-02	72.83	1.36		11.39	9.49	
19	5	6	200	650.	110	0.803E-02	39.08	1.24		7.12	10.96	HI
20	5	3	150	750.	100	0.449E-01	12.17	0.69		4.58	6.11	
21	3	2	150	200.	100	0.120E-01	16.70	0.95		2.20	10.99	HI
22	20	2	200	900.	110	0.111E-01	41.56	1.32		11.05	12.28	HI
23	2	1	200	370.	110	0.457E-02	31.29	1.00		2.69	7.26	
24	1	4	150	590.	100	0.353E-01	1.89	0.11	LO	0.12	0.19	LO
25	21	1	150	1300.	100	0.777E-01	1.41	0.08	LO	0.15	0.11	LO
26	20	16	150	800.	100	0.478E-01	4.95	0.28	LO	0.92	1.15	
27	27	23	150	1650.	100	0.987E-01	16.33	0.92		17.39	10.54	HI
28	24	20	150	2370.	110	0.119E 00	12.25	0.69		12.30	5.19	
29	27	24	150	500.	110	0.251E-01	18.40	1.04		5.51	11.02	HI
30	23	20	150	420.	100	0.251E-01	4.60	0.26	LO	0.42	1.01	
31	23	22	150	800.	100	0.478E-01	7.21	0.41		1.86	2.32	
32	25	20	200	1100.	110	0.136E-01	40.42	1.29		12.83	11.66	HI
33	19	28	195	1600.	100	0.267E-01	41.18	1.38		26.06	16.29	HI
34	28	15	195	900.	100	0.150E-01	34.69	1.16		10.56	11.73	HI
35	15	9	195	1150.	100	0.192E-01	17.28	0.58		3.75	3.26	
36	29	19	300	2650.	120	0.387E-02	51.86	0.73		5.79	2.19	
37	19	18	300	900.	120	0.131E-02	36.07	0.51		1.00	1.12	
38	18	37	261	2250.	110	0.760E-02	1.80	0.03	LO	0.02	0.01	LO
39	39	37	296	1850.	110	0.339E-02	50.45	0.73		4.82	2.60	
40	37	25	200	750.	110	0.926E-02	50.04	1.59		12.99	17.31	HI
41	25	38	150	2150.	100	0.129E 00	2.16	0.12	LO	0.54	0.25	LO
42	24	38	150	1000.	100	0.598E-01	0.41	0.02	LO	0.01	0.01	LO
43	22	21	150	900.	100	0.538E-01	18.67	1.06		12.15	13.51	HI
44	5	30	150	1500.	100	0.897E-01	11.53	0.65		8.30	5.54	
45	30	31	150	800.	100	0.478E-01	2.31	0.13	LO	0.22	0.28	LO
46	15	31	150	1200.	100	0.718E-01	7.52	0.43		3.01	2.51	
47	15	32	150	1200.	100	0.718E-01	1.76	0.10	LO	0.21	0.17	LO
48	14	32	150	1250.	100	0.748E-01	12.76	0.72		8.35	6.68	
49	30	7	150	1150.	100	0.688E-01	4.82	0.27	LO	1.26	1.10	
50	31	8	150	1000.	100	0.598E-01	5.28	0.30	LO	1.30	1.30	

PIPE NO	NODES FROM	TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS--- MT	MT/1000	CK
51	32	10	150	800.	100	0.478E-01	10.62	0.60		3.80	4.75	
52	33	14	230	1350.	110	0.844E-02	65.65	1.58		19.57	14.49	HI
53	29	33	271	2150.	110	0.605E-02	67.37	1.17		14.70	6.84	
54	13	34	150	1100.	100	0.658E-01	5.31	0.30	LO	1.45	1.22	
55	34	35	150	700.	100	0.419E-01	2.19	0.12	LO	0.18	0.25	LO
58	12	13	150	1100.	100	0.658E-01	12.25	0.69		6.82	6.20	
59	23	20	100	420.	100	0.181E 00	1.58	0.20	LO	0.42	1.01	

NODE	GROUND ELEV.	FLOW	HGL ELEV	HEAD MTRS	--------PRESSURE-------- ATM	---CK	PCT DROP	---CK
1	1.8	-30.81	23.64U	21.84	2.11		67.98	HI
2	1.5	-26.97	26.32U	24.82	2.40		63.76	HI
3	2.0	-19.27	28.52U	26.52	2.57		61.00	HI
4	1.5	-34.66	23.52U	22.02	2.13		67.85	HI
5	2.0	-10.05	33.11U	31.11	3.01		54.26	HI
6	1.5	-13.78	25.98U	24.48	2.37		64.26	HI
7	1.5	-9.34	23.54U	22.04	2.13		67.83	HI
8	1.5	-10.32	23.28U	21.78	2.11		68.21	HI
9	1.5	-12.76	23.84U	22.34	2.16		67.39	HI
10	1.5	-17.44	23.58U	22.08	2.14		67.77	HI
11	1.5	-12.86	24.90U	23.40	2.27		65.84	HI
12	2.0	-12.05	27.29U	25.29	2.45		62.81	HI
13	1.5	-6.94	20.47U	18.97	1.84		72.30	HI
14	8.0	-10.89	35.73U	27.73	2.68		55.27	HI
15	10.0	-7.93	27.59U	17.59	1.70		70.69	HI
16	3.0	-8.37	36.45U	33.45	3.24		50.08	HI
17	10.0	-5.43	44.49U	34.49	3.34		42.51	
18	20.0	0.0	63.20U	43.20	4.18		13.59	
19	25.0	-12.15	64.21U	39.21	3.80		12.87	
20	3.0	-12.35	37.37U	34.37	3.33		48.70	
21	2.0	-17.26	23.78U	21.78	2.11		67.96	HI
22	3.0	11.46	35.94U	32.94	3.19		50.84	HI
23	8.0	-2.93	37.79U	29.79	2.88		51.94	HI
24	20.0	-5.74	49.67U	29.67	2.87		40.66	
25	18.0	-7.45	50.20U	32.20	3.12		38.08	
27	3.0	34.72	55.18U	52.18	5.05		22.12	
28	15.0	-6.68	38.15U	23.15	2.24		57.91	HI
29	65.0	242.51U	70.00	5.00	0.48	LO	0.00	
30	5.0	-4.41	24.80U	19.80	1.92		69.53	HI
31	5.0	-4.55	24.58U	19.58	1.90		69.88	HI
32	6.0	-3.91	27.38U	21.38	2.07		66.59	HI
33	12.0	-1.72	55.30U	43.30	4.19		25.35	
34	1.5	-3.12	19.02U	17.52	1.70		74.42	HI
35	1.5	-2.19	18.85U	17.35	1.68		74.68	HI
37	30.0	-2.20	63.18U	33.18	3.21		17.04	
38	18.0	-2.57	49.66U	31.66	3.06		39.12	
39	63.0	50.45U	68.00	5.00	0.48	LO	28.57	

APPENDIX TABLE IX-C-8

FILLING CONDITION

MISAMIS OCCIDENTAL FUTURE TRANSMISSION LINES AND FEEDER MAINS YR 2000

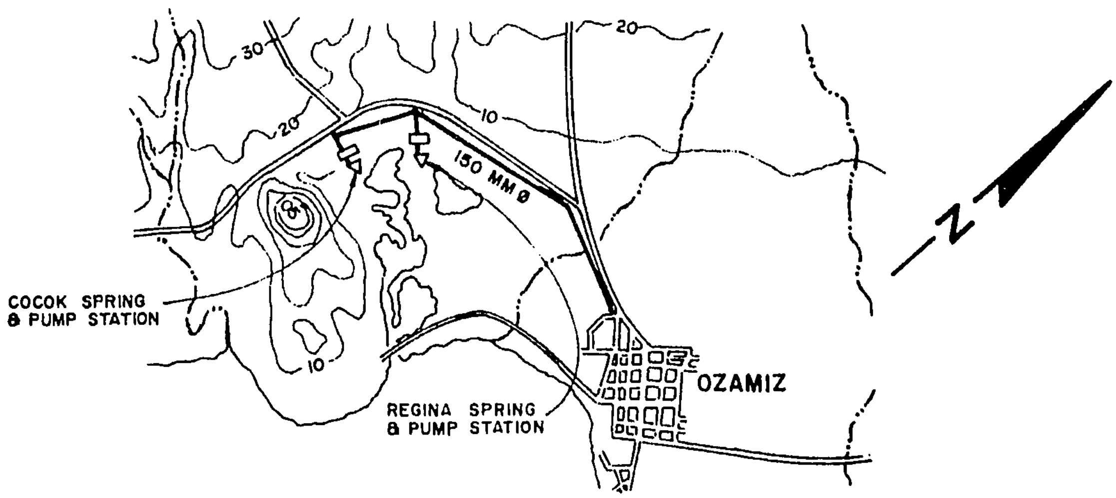
```
INPUT AND OUTPUT IN                  LPS
NO OF NODES                           37
NO OF PIPES                           57
MAX NO OF ITERATIONS                  20
PEAKING FACTOR                   0.30000
ALLOW P-DROP FR/STATIC - PCT 50.0
STATIC HGL FOR P-DROP CALC      70.0
MAX UNBAL - LPS                  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               33.10
BANDWIDTH                              7
ITER  1 UNBAL    38.91 LPS
ITER  2 UNBAL    37.80 LPS
ITER  3 UNBAL     9.53 LPS
ITER  4 UNBAL     1.98 LPS
ITER  5 UNBAL     1.05 LPS
ITER  6 UNBAL     0.15 LPS
ITER  7 UNBAL     0.00 LPS

SOLUTION NO. 1 REACHED IN   7 ITERATIONS
       0.0029     UNBALANCE
```

IPE ID	NODES FROM	TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	CK	--HEADLOSS--- MT	MT/1000	CK
1	29	19	200	2650.	110	0.327E-01	4.35	0.14	LO	0.50	0.19	LO
2	19	18	200	900.	110	0.111E-01	3.41	0.11	LO	0.11	0.12	LO
3	18	17	200	1650.	110	0.204E-01	6.57	0.21	LO	0.66	0.40	LO
4	17	16	200	650.	110	0.803E-02	5.71	0.18	LO	0.20	0.31	LO
5	16	3	200	750.	110	0.926E-02	7.20	0.23	LO	0.36	0.48	LO
6	3	4	150	590.	100	0.353E-01	2.97	0.17	LO	0.26	0.45	LO
7	6	4	200	920.	110	0.114E-01	2.40	0.08	LO	0.06	0.06	LO
8	6	7	150	1100.	100	0.658E-01	1.91	0.11	LO	0.22	0.20	LO
9	7	8	150	800.	100	0.478E-01	0.89	0.05	LO	0.04	0.05	LO
10	9	8	150	1700.	100	0.102E 00	0.14	0.01	LO	0.00	0.00	LO
11	9	10	150	1200.	100	0.718E-01	0.53	0.03	LO	0.02	0.02	LO
12	11	10	150	1200.	100	0.718E-01	0.90	0.05	LO	0.06	0.05	LO
13	12	11	195	700.	100	0.117E-01	3.47	0.12	LO	0.12	0.17	LO
14	14	12	195	500.	100	0.833E-02	8.33	0.28	LO	0.42	0.84	
15	29	19	395	2650.	120	0.101E-02	28.44	0.23	LO	0.50	0.19	LO
16	19	18	395	900.	120	0.344E-03	22.26	0.18	LO	0.11	0.12	LO
17	18	17	261	1650.	110	0.557E-02	13.23	0.25	LO	0.66	0.40	LO
18	17	5	261	1200.	110	0.405E-02	12.99	0.24	LO	0.47	0.39	LO
19	5	6	200	650.	110	0.803E-02	7.07	0.22	LO	0.30	0.46	LO
20	5	3	150	750.	100	0.449E-01	1.48	0.08	LO	0.09	0.12	LO
21	3	2	150	200.	100	0.120E-01	1.85	0.10	LO	0.04	0.19	LO
22	20	2	200	900.	110	0.111E-01	10.05	0.32	LO	0.80	0.89	
23	2	1	200	370.	110	0.457E-02	6.50	0.21	LO	0.15	0.40	LO
24	1	4	150	590.	100	0.353E-01	1.57	0.09	LO	0.08	0.14	LO
25	21	1	150	1300.	100	0.777E-01	1.22	0.07	LO	0.11	0.09	LO
26	20	16	150	800.	100	0.478E-01	3.16	0.18	LO	0.40	0.50	
27	27	23	150	1650.	100	0.987E-01	14.76	0.84		14.43	8.74	
28	24	20	150	2370.	110	0.119E 00	10.32	0.58		8.96	3.78	
29	27	24	150	500.	110	0.251E-01	19.96	1.13		6.41	12.82	HI
30	23	20	150	420.	100	0.251E-01	7.07	0.40	LO	0.94	2.23	
31	23	22	150	800.	100	0.478E-01	4.68	0.26	LO	0.83	1.04	
32	20	25	200	1100.	110	0.136E-01	4.15	0.13	LO	0.19	0.17	LO
33	19	28	195	1600.	100	0.267E-01	7.69	0.26	LO	1.17	0.73	
34	28	15	195	900.	100	0.150E-01	6.36	0.21	LO	0.46	0.51	
35	15	9	195	1150.	100	0.192E-01	3.22	0.11	LO	0.17	0.15	LO
36	29	19	300	2650.	120	0.387E-02	13.80	0.20	LO	0.50	0.19	LO
37	19	18	300	900.	120	0.131E-02	10.80	0.15	LO	0.11	0.12	LO
38	18	37	261	2250.	110	0.760E-02	16.68	0.31	LO	1.39	0.62	
39	37	39	296	1850.	110	0.339E-02	26.87	0.39	LO	1.50	0.81	
40	25	37	200	750.	110	0.926E-02	10.63	0.34	LO	0.74	0.98	
41	38	25	150	2150.	100	0.129E 00	7.97	0.45		6.01	2.80	
42	24	38	150	1000.	100	0.598E-01	8.49	0.48		3.14	3.14	
43	22	21	150	900.	100	0.538E-01	4.68	0.26	LO	0.94	1.04	
44	5	30	150	1500.	100	0.897E-01	2.44	0.14	LO	0.47	0.31	LO
45	30	31	150	800.	100	0.478E-01	0.71	0.04	LO	0.03	0.03	LO
46	15	31	150	1200.	100	0.718E-01	1.24	0.07	LO	0.11	0.09	LO
47	15	32	150	1200.	100	0.718E-01	0.31	0.02	LO	0.01	0.01	LO
48	14	32	150	1250.	100	0.748E-01	2.53	0.14	LO	0.42	0.33	LO
49	30	7	150	1150.	100	0.688E-01	0.85	0.05	LO	0.05	0.04	LO
50	31	8	150	1000.	100	0.598E-01	1.04	0.06	LO	0.06	0.06	LO

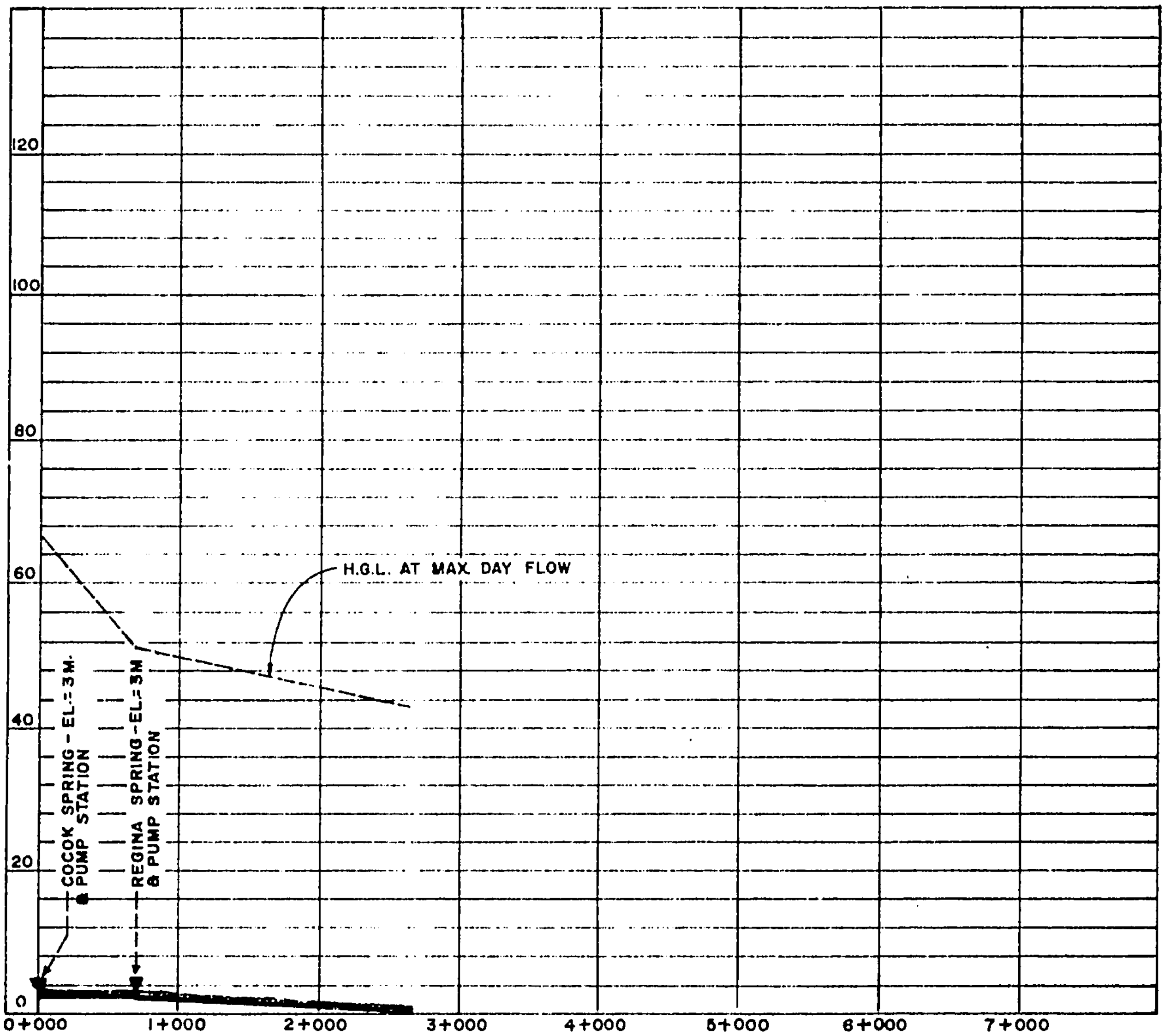
PE O	NODES FROM	TO	DIA MM	L MTRS	H-W C	K-VALUE	FLOW	--VEL-- MPS	--CK	--HEADLOSS--- MT	MT/1000	CK
51	32	10	150	800.	100	0.478E-01	2.06	0.12	LO	0.18	0.23	LO
52	33	14	230	1350.	110	0.844E-02	13.04	0.31	LO	0.98	0.73	
53	29	33	271	2150.	110	0.605E-02	13.38	0.23	LO	0.74	0.34	LO
54	13	34	150	1100.	100	0.658E-01	1.06	0.06	LO	0.07	0.07	LO
55	34	35	150	700.	100	0.419E-01	0.44	0.02	LO	0.01	0.01	LO
58	12	13	150	1100.	100	0.658E-01	2.45	0.14	LO	0.35	0.31	LO
59	23	20	100	420.	100	0.181E 00	2.43	0.31	LO	0.94	2.23	

NODE	GROUND ELEV	FLOW	HGL ELEV	HEAD MTRS	PRESSURE ATM	CK	PRESSURE PCT DROP	CK
1	1.8	-6.16	67.98U	66.18	6.41		2.96	
2	1.5	-5.39	68.13U	66.63	6.45		2.73	
3	[illegible]	-3.83	6[illegible].17U	66.17	6.41		2.69	
4	1.5	-6.93	67.90U	66.40	6.43		3.06	
5	2.0	-2.01	68.26U	66.26	6.41		2.56	
6	1.5	-2.76	67.96U	66.46	6.43		2.98	
7	1.5	-1.87	67.74U	66.24	6.41		3.30	
8	1.5	-2.06	67.70U	66.20	6.41		3.35	
9	1.5	-2.55	67.71U	66.21	6.41		3.35	
10	1.5	-3.49	67.68U	66.18	6.41		3.38	
11	1.5	-2.57	67.74U	66.24	6.41		3.29	
12	2.0	-2.41	67.86U	65.86	6.38		3.15	
13	1.5	-1.39	67.51U	66.01	6.39		3.63	
14	8.0	-2.18	68.28U	60.28	5.84		2.77	
15	10.0	-1.59	67.87U	57.87	5.60		3.54	
16	3.0	-1.67	68.53U	65.53	6.34		2.20	
17	10.0	-1.09	68.73U	58.73	5.69		2.12	
18	20.0	0.0	69.39U	49.39	4.78		1.21	
19	25.0	-2.43	69.50U	44.50	4.31		1.11	
20	3.0	-2.47	68.93U	65.93	6.38		1.60	
21	2.0	-3.45	68.10U	66.10	6.40		2.80	
22	3.0	0.0	69.03U	66.03	6.39		1.44	
23	8.0	-0.59	69.87U	61.87	5.99		0.22	
24	20.0	-1.15	77.89U	57.89	5.60		-15.78	
25	18.0	-1.49	68.74U	50.74	4.91		2.43	
27	3.0	34.72	84.30U	81.30	7.87	HI	-21.34	
28	15.0	-1.34	68.33U	53.33	5.16		3.03	
29	65.0	59.98U	70.00	5.00	0.48	LO	0.00	
30	5.0	-0.88	67.79U	62.79	6.08		3.39	
31	5.0	-0.91	67.77U	62.77	6.08		3.43	
32	6.0	-0.78	67.87U	61.87	5.99		3.33	
33	12.0	-0.34	69.26U	57.26	5.54		1.27	
34	1.5	-0.62	67.44U	65.94	6.38		3.74	
35	1.5	-0.44	67.43U	65.93	6.38		3.75	
37	30.0	-0.44	68.00U	38.00	3.68		5.00	
38	18.0	-0.51	74.75U	56.75	5.49		-9.13	
39	63.0	-26.87U	66.50	3.50	0.34	LO	50.00	HI



PLAN

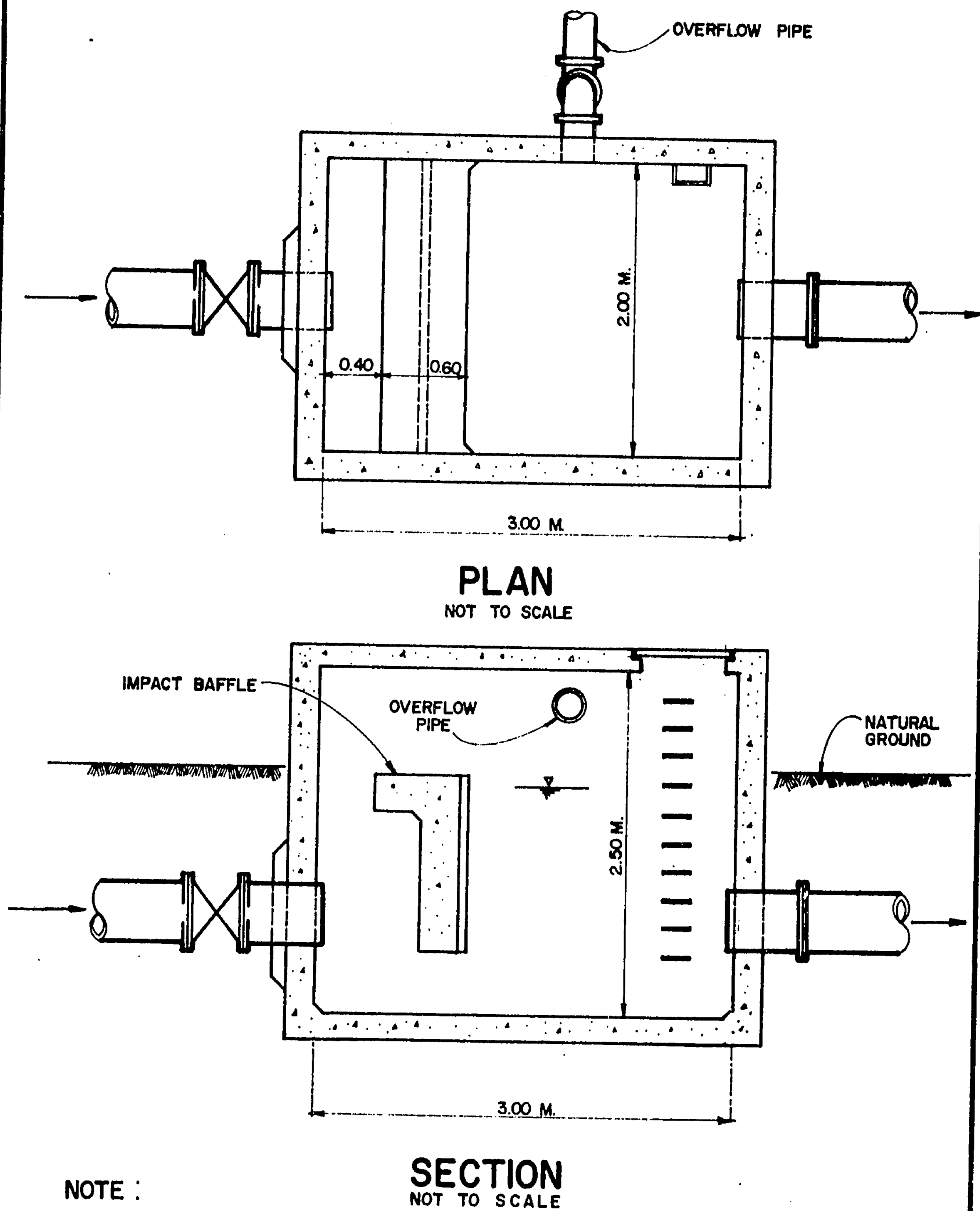
500 0 500 1000
SCALE IN METERS



PROFILE

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

APPENDIX FIGURE IX-C-1
COCOK-REGINA SPRINGS
TRANSMISSION LINE

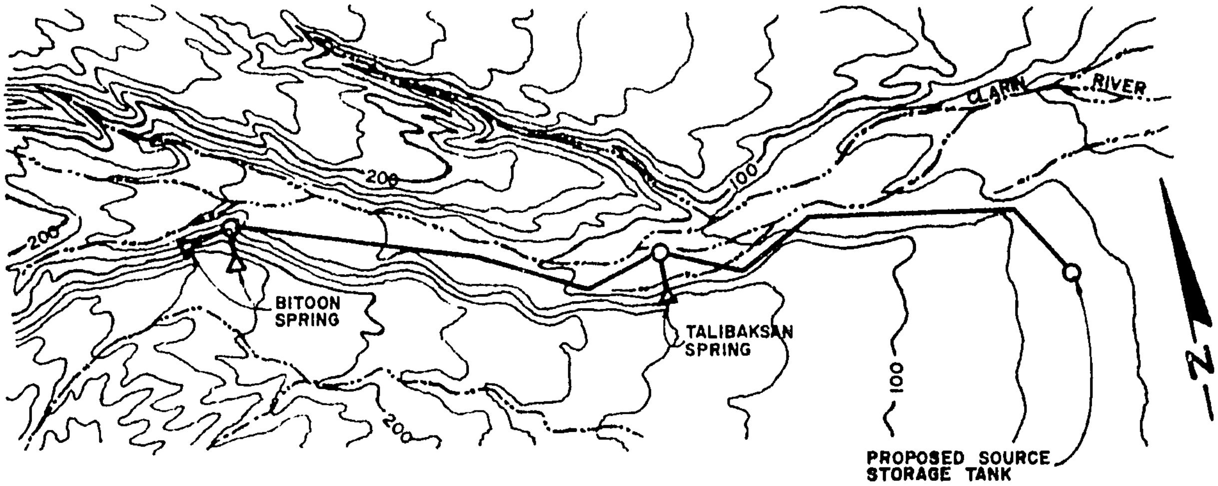


NOTE :

DIMENSIONS SHOWN ARE ONLY TYPICAL ; ACTUAL DIMENSIONS TO VARY WITH DESIGN FLOW.

APPENDIX FIGURE IX - C-2
HYDRAULIC CONTROL CHAMBER
FOR PRESSURE DISSIPATION

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



PLAN

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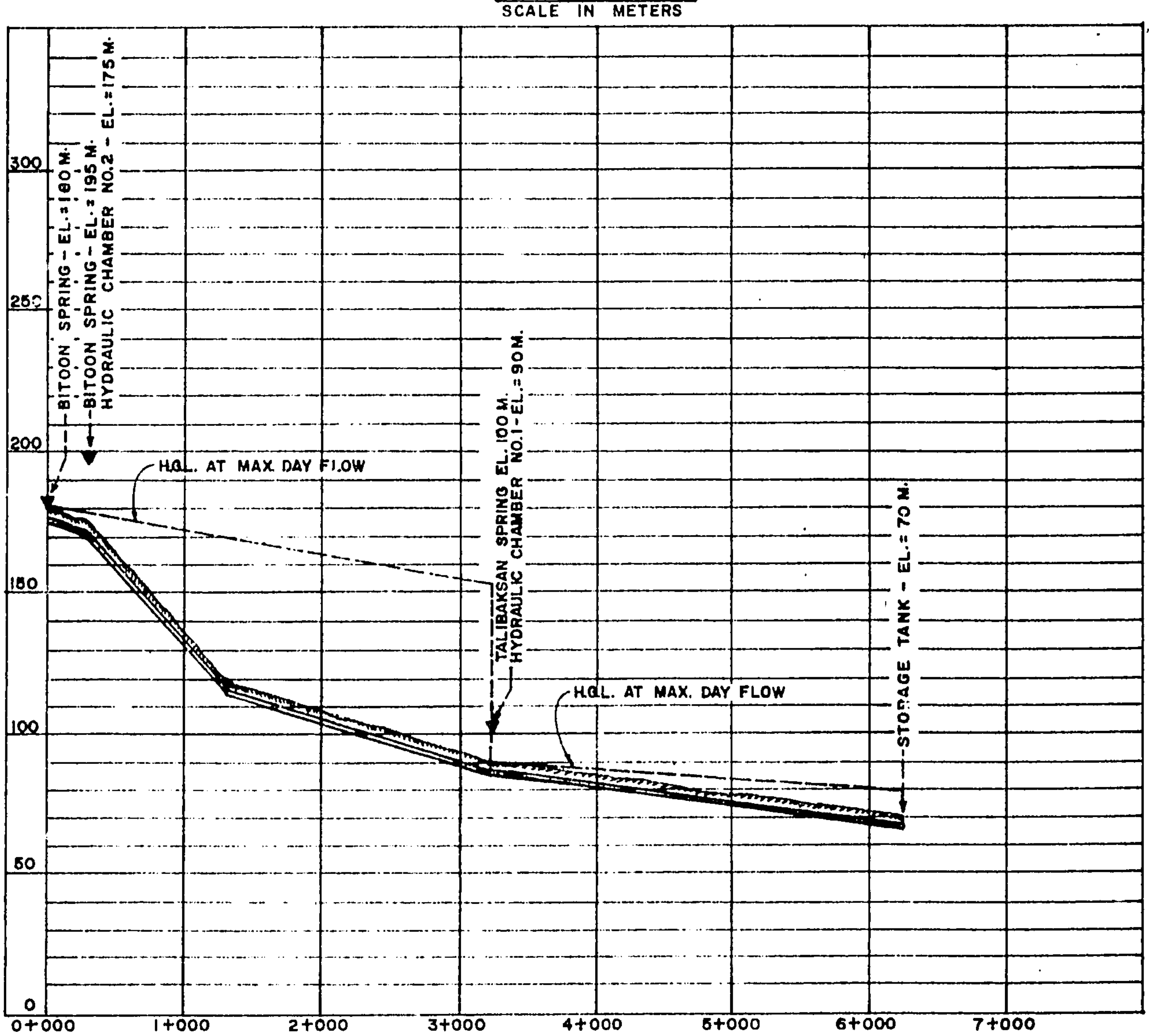
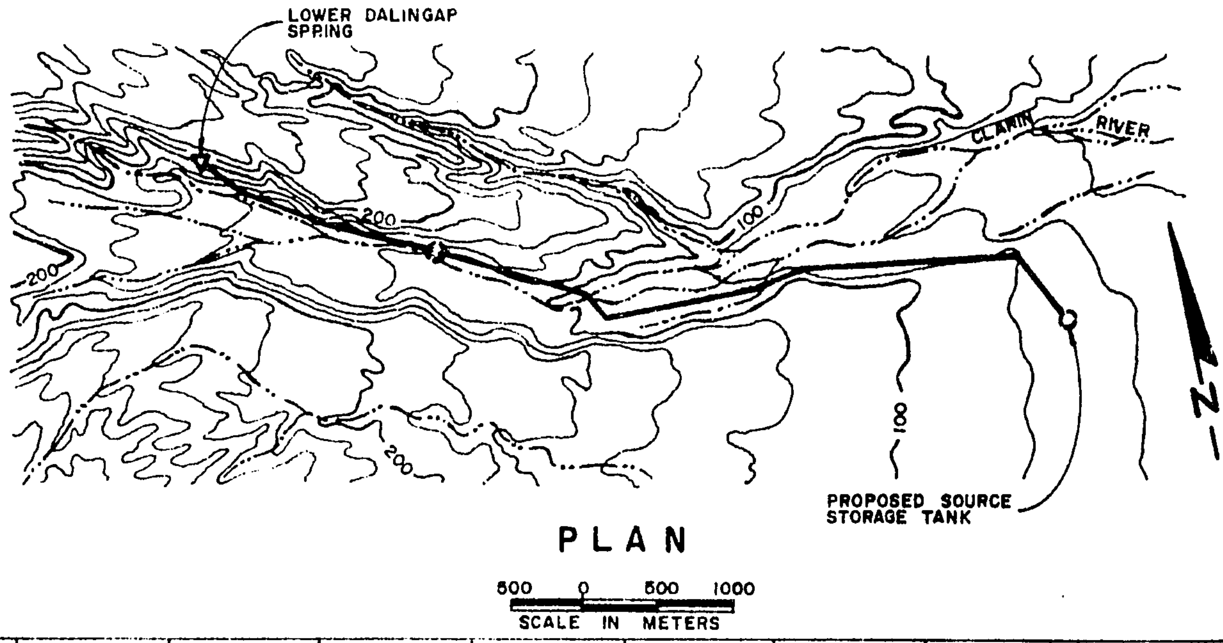
SCALE IN METERS

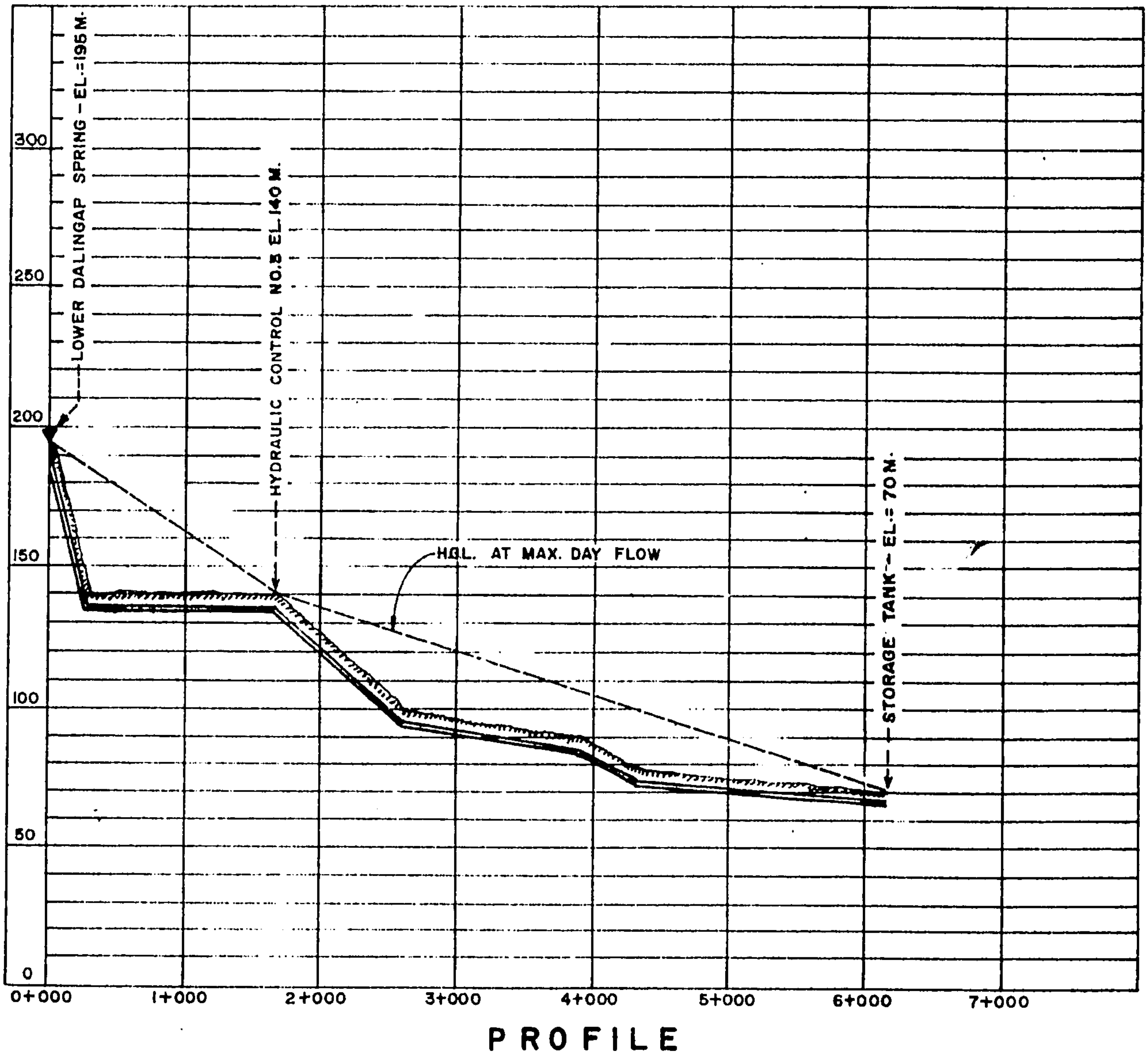


PROFILE

APPENDIX FIGURE IX-C-3
BITOON - TALIBAKSAN SPRINGS
TRANSMISSION LINE

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





APPENDIX FIGURE IX-D-1
LOWER DALINGAP SPRING
TRANSMISSION LINE

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

APPENDIX TO CHAPTER X

APPENDIX X-C
ECONOMIC BENEFITS

Increase in Land Values

Appendix Table X-C-1 shows the present value of benefits associated with "increase in land value", 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 27.5 ha from 1978 to 1980, by 26 ha from 1981 to 1985, and by 30 ha from 1986 to 1990.

2. The 1975 land use distribution of 8 per cent commercial; 4 per cent industrial; and 88 per cent residential, was assumed to remain unchanged during the 13-year projection period.

3. The 1975 costs of land are:

Residential	:	₱40 per sqm
Industrial	:	₱50 " "
Commercial	:	₱85 " "

4. A discount factor of 12 per cent was used to obtain the present day values of the benefits. This is believed to be the opportunity cost of capital and is commonly used for public investment projects like water resource development.

APPENDIX TABLE X-C-1

INCREASE IN LAND VALUES

Year	Land Use (sqm) Commercial	Industrial	Residential	Cost of Land/sqm* Commercial	Industrial	Residential	Cost of Served Land (in M ₱)	Benefit Due To Increase Of Land (in M ₱)	Discount Factor**	Present Value of Benefit (in M ₱)
1975				₱ 85	₱ 50	₱ 40			1.000	
1976				97.75	57.5	46			0.893	
1977				112.41	66.13	52.9			0.797	
1978	22,000	11,000	242,000	129.27	76.04	60.84	₱ 18.404	3.68	0.712	₱ 2.62
1979	22,000	11,000	242,000	148.67	87.45	69.96	21.163	4.23	0.636	2.69
1980	22,000	11,000	242,000	170.97	100.57	80.45	24.337	4.87	0.567	2.76
1981	20,800	10,400	229,000	196.61	115.65	92.52	26.500	5.30	0.507	2.69
1982	20,800	10,400	229,000	226.10	133.00	106.40	30.452	6.09	0.452	2.75
1983	20,800	10,400	229,000	260.02	152.95	122.36	35.020	7.00	0.404	2.83
1984	20,800	10,400	229,000	299.02	175.89	140.72	40.274	8.05	0.361	2.91
1985	20,800	10,400	229,000	343.87	202.28	161.82	46.313	9.24	0.322	2.98
1986	24,000	12,000	264,000	395.45	232.62	186.09	61.410	12.28	0.287	3.52
1987	24,000	12,000	264,000	454.77	267.51	214.01	70.623	14.12	0.251	3.54
1988	24,000	12,000	264,000	522.99	307.64	246.11	81.220	16.24	0.229	3.72
1989	24,000	12,000	264,000	601.43	353.79	283.02	93.400	18.68	0.205	3.83
1990	24,000	12,000	264,000	691.64	406.85	325.48	107.410	21.48	0.183	3.93
								TOTAL		₱40.77

*Escalated by 15 per cent every year.
**Discounted at 12 per cent.

APPENDIX TABLE X-C- 2

HEALTH BENEFITS

Year	Project Area Population	Cost of Time Lost Due to Illness*	Economic Loss Due to Premature Death*	Cost of Medical Expense*	Total	Escalated Total	40% Reduction	Discount Factor**	Present Value
1978	79,943	₱ 27,380.50	₱ 192,303	₱ 95,492	₱315,176	₱ 460,157	₱ 276,094	0.712	₱ 196,579
1979	82,741	28,339	199,033	98,834	326,206	525,192	315,115	0.636	200,413
1980	85,500	29,284	205,670	102,130	337,084	596,639	357,983	0.567	202,976
1981	88,322	30,250	212,459	105,501	348,210	679,010	407,406	0.507	206,555
1982	91,236	31,248	219,468	108,981	359,697	769,752	461,851	0.452	208,757
1983	94,247	32,280	226,711	112,578	371,569	876,903	526,142	0.404	212,561
1984	97,357	33,345	234,192	116,293	383,830	994,120	596,472	0.361	215,326
1985	100,570	34,445	241,921	120,131	396,497	1,130,016	678,010	0.322	218,319
1986	103,889	35,582	249,905	124,095	409,582	1,286,087	771,652	0.287	221,464
1987	107,317	36,756	258,151	128,190	423,097	1,459,685	875,811	0.251	219,829
1988	110,858	37,969	266,669	132,420	437,058	1,660,820	996,492	0.229	228,197
1989	114,517	39,222	275,471	136,791	451,484	1,887,203	1,132,322	0.205	232,126
1990	118,400	40,552	284,811	141,429	466,792	2,147,243	1,288,346	0.183	235,767
								TOTAL	₱2,798,869

*Escalated by 10 per cent every year.
**Discounted at 12 per cent.

APPENDIX TABLE X-C-3

PERSONAL SATISFACTION BENEFITS

Year	Served Pop.	No. of House-holds	Willingness to Pay ₱120.00/Yr*	Willingness to Pay* Escalated	Discount Factor**	Present Value
1978	23,688	3,644	₱437,280	₱ 550,973	0.712	₱ 392,293
1979	25,275	3,888	466,560	625,190	0.636	397,621
1980	26,900	4,138	496,560	705,115	0.567	399,800
1981	28,783	4,428	531,360	797,040	0.507	404,099
1982	30,798	4,738	568,560	904,010	0.452	408,613
1983	32,954	5,070	608,400	1,028,196	0.404	415,391
1984	35,260	5,425	651,000	1,165,290	0.361	420,670
1985	37,729	5,804	696,480	1,323,312	0.322	426,106
1986	40,370	6,211	745,320	1,498,093	0.287	429,953
1987	43,196	6,646	797,520	1,698,718	0.257	436,570
1988	46,219	7,111	853,320	1,928,503	0.229	441,627
1989	49,455	7,608	912,960	2,191,104	0.205	449,176
1990	53,000	8,154	978,480	2,485,339	0.183	454,817
				TOTAL		₱5,476,736

*Escalated by 6 per cent every year.
**Discount at 12 per cent.

APPENDIX TABLE X-C-4a

SHORT-TERM EMPLOYMENT BENEFITS

Year	Cost of Skilled Labor	Cost of Unskilled Labor	Total Labor Cost	Escalated Total Labor Cost*	Discount Factor**	Present Value
1978	₱14,200	₱ 205,500	₱219,700	₱320,762	0.712	₱ 228,383
1979	15,225	304,750	319,975	515,160	0.636	327,642
1980	35,025	478,750	513,775	909,382	0.567	575,620
1981	16,825	336,750	353,575	689,471	0.507	349,562
1982	5,825	116,750	122,575	262,311	0.452	118,565
1983	8,925	179,000	187,925	443,503	0.404	179,175
1984	8,925	179,000	187,925	486,726	0.361	175,708
1985	8,925	179,000	187,925	535,586	0.322	172,459
1986	8,925	179,000	187,925	590,085	0.287	169,354
1987	17,050	339,750	356,800	1,230,960	0.257	316,357
1988	6,350	126,750	133,100	505,780	0.229	115,824
1989	6,350	126,750	133,100	556,358	0.205	114,053
1990	6,350	126,750	133,100	612,260	0.183	112,044
				TOTAL		₱2,894,746

*Escalated by 10 per cent every year.
**Discounted at 12 per cent.

APPENDIX TABLE X-C-4b

LONG-TERM EMPLOYMENT BENEFITS

Year	Annual Salaries Under Present Staffing Arrangement*	Annual Salaries Under Proposed Staffing Arrangement*	Salary Difference	Discount Factor**	Present Value
1978	₱179,735	₱ 335,527	₱155,792	.712	₱ 110,924
1979	197,708	373,509	175,801	.636	111,809
1980	217,479	415,790	198,311	.567	112,442
1981	239,227	462,801	223,574	.507	113,352
1982	263,149	528,704	265,555	.452	120,031
1983	289,464	603,991	314,527	.404	127,069
1984	318,411	690,091	371,680	.361	134,176
1985	350,252	760,480	410,228	.322	132,093
1986	385,277	838,049	452,772	.287	129,946
1987	423,805	919,134	495,329	.257	127,300
1988	466,185	1,012,886	546,701	.229	125,195
1989	512,803	1,116,200	603,397	.205	123,696
1990	564,084	1,224,550	660,466	.183	120,865
				T O T A L	₱1,588,898

*Escalated by 10 per cent every year.
**Discounted at 12 per cent.

Fire Protection Benefits

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.

In 1978-82, 30 high-valued hectares of the present service area will be installed with fire hydrants; in 1983-86, an additional 30 ha will be protected. From 1986 to 1990, 25 low-valued hectares will be covered every year. In the meantime, the service area itself was projected to increase by 27.5 ha every year from 1978 to 1980; by 26 ha every year from 1981 to 1985; and by 30 ha every year from 1986 to 1990.

The average annual loss due to fire in Ozamiz based on records of the fire department for the period 1972 to 1974 was ₱1.243 million. In the absence of similar information on Clarin, it was assumed that fire damages there amount to an annual average of ₱466,000. This was obtained by correlating the ratio of annual fire loss in Ozamiz to the number of dwelling units in the core city (800) and applying the same ratio to Clarin which has about 300 dwelling units. Taken together therefore, estimated annual loss due to fire in both Ozamiz and Clarin amounts to ₱1,709,000.

Since ₱1.709 million represents damage to the entire study area and not the service area, only the corresponding percentages of this amount each year were used, in accordance with the fire hydrant schedule and yearly expansion of service area. The average annual loss due to fire in the portion of the service area with fire hydrants was determined in the following manner:

$$\frac{\text{No. of hectares with installed fire hydrants}}{\text{No. of hectares in service area}} \times ₱1,709,000$$

It was further assumed that the annual loss due to fire would be reduced by half with good, plentiful water supply. They were further discounted at 12 per cent to obtain their present values. The annual loss in the succeeding years up to 1990 was escalated by 10 per cent due to inflation. Appendix Table X-C-5 shows the computations of the fire protection benefits in Ozamiz.

APPENDIX TABLE X-C-5

FIRE PROTECTION BENEFITS

Year	Average Annual Loss Due to Fire	Escalation Due to Inflation*	Reduction Due to Project	Discount Factor**	Fire Protection Benefits
1979	₱ 56,341	₱ 90,709	₱ 45,355	0.636	₱ 28,846
1980	88,397	156,463	78,232	0.567	44,358
1981	123,245	240,328	120,164	0.507	60,923
1982	153,503	328,496	164,248	0.452	74,240
1983	180,021	424,850	212,425	0.404	85,820
1984	203,452	526,941	263,471	0.361	95,113
1985	224,306	639,272	319,636	0.322	102,923
1986	242,986	762,976	381,488	0.287	109,487
1987	327,173	1,128,747	564,374	0.257	145,044
1988	403,412	1,532,966	766,483	0.229	175,525
1989	472,777	1,976,208	988,104	0.205	202,561
1990	536,157	2,466,322	1,233,161	0.182	225,668
				TOTAL	₱1,350,508

*Escalated by 10 per cent every year.
**Discounted at 12 per cent.

Reduction of 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 of fire insurance costs:

1. On the basis of field surveys, the number of dwelling units in the core city was estimated to be 800. Of these, it was assumed that only 9 per cent (72 units) were made of concrete and galvanized iron and are therefore considered insurable. This assumption was based on the 1970 Census on Housing which indicated that 9 per cent of the total number of dwelling units in the entire city of Ozamiz were made of concrete and galvanized iron.

2. It was further assumed that only 50 per cent of the 72 insurable dwelling units are actually insured, equivalent to 36.

3. The dwelling units were classified into 64 per cent residential and 36 per cent commercial. The number of institutional and industrial establishments proved to be insignificant for further computations. This classification was based on the ratio of service connections by consumer category over the total number of connections. (Refer to Chapter IV, Table IV-3.)

4. Based on the projections in Chapter IV, the service connections in Ozamiz for all consumer categories are expected to increase by 3.4 per cent from 1975 to 1980 and 4.64 per cent from 1981 to 1990. It was assumed that the number of insured commercial and domestic units would increase at the same rates.

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 one-third because of an improved and plentiful water supply system and increased fire-fighting capabilities.

8. With the development of the area, specifically its urbanization, additional dwelling units made of stronger materials are expected to be constructed. Accompanying this activity, other fire protection techniques in building construction would be considered. While premium rates in general remain constant over a period of years, the quantification of the reduction of fire insurance costs from 1978 to 1990 is nevertheless presented in Appendix Table X-C-6 to illustrate the impact of an improved water supply system.

APPENDIX TABLE X-C-6

REDUCTION OF FIRE INSURANCE COSTS

Year	Level of Insurance Costs	Escalated Level of Insurance Costs*	Reduction due to Project	Discount Factor**	Present Value
1978	₱28,056	₱ 40,962	₱ 13,517	0.712	₱ 9,624
1979	29,729	47,864	15,795	0.636	10,046
1980	30,151	53,367	17,611	0.567	9,985
1981	31,823	62,055	20,478	0.507	10,382
1982	32,668	69,910	23,070	0.452	10,428
1983	34,340	81,042	26,744	0.404	10,805
1984	36,012	93,271	30,779	0.361	11,111
1985	38,107	108,605	35,840	0.322	11,540
1986	39,778	124,903	41,218	0.287	11,830
1987	41,873	144,462	47,672	0.257	12,252
1988	43,968	167,078	55,136	0.229	12,626
1989	46,062	192,539	63,538	0.205	13,025
1990	47,735	219,581	72,462	0.183	13,261
				TOTAL	₱146,915

*Escalated by 10 per cent every year.
**Discounted at 12 per cent.

APPENDIX TABLE X-C-7

ECONOMIC COSTS
(in million pesos)

Year	Capital Expenditure	Annual Operating	Depreciation	Discount Factor	Capital Expenditure*	Annual Operating*	Depreciation*
1976	₱ 1.222	₱ .315	₱ .026	1.000	₱ 1.222	₱ .315	₱ .026
1977	2.679	.349	.032	0.893	2.392	.312	.029
1978	6.206	.536	.081	0.797	4.946	.427	.065
1979	8.283	.588	.164	0.712	5.897	.419	.117
1980	6.539	1.045	.283	0.636	4.159	.665	.180
1981	4.248	1.148	.544	0.567	2.409	.651	.308
1982	2.623	1.085	.634	0.507	1.330	.550	.321
1983	2.865	1.191	.694	0.452	1.295	.538	.314
1984	3.154	1.511	.758	0.404	1.274	.610	.306
1985	4.652	1.672	.830	0.361	1.679	.604	.300
1986	8.464	2.057	.932	0.322	2.725	.662	.300
1987	9.451	2.366	1.112	0.287	2.712	.679	.319
1988	5.038	2.651	1.312	0.257	1.295	.681	.337
1989	5.448	2.898	1.424	0.229	1.248	.664	.325
1990	3.393	3.353	1.546	0.205	.696	.687	.317
			Sub-Total		₱35.279**	₱8.464	₱3.565
			TOTAL		₱47.308		

*Discounted at 12 per cent.
*Foreign exchange component and unskilled labor cost are still subject to shadow pricing to arrive at final total economic costs.

APPENDIX TO CHAPTER XI

APPENDIX TABLE XI-B-1
PROJECT COST OF RECOMMENDED PROGRAM[1]
MISAMIS OCCIDENTAL WATER DISTRICT
(WITHOUT ESCALATION)

P x 1000

Item	Service Life	FEC (%)	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	Total
Source Development	50	43	153	444	888	888	444	-	-	-	-	-	-	-	-	-	-	2817
Source Transmission Line	50	26	-	171	745	1489	745	-	-	-	-	-	-	-	-	-	-	3150
Transmission Lines	50	25	366	794	1588	1588	1588	794	-	-	-	186	1614	1614	-	-	-	10132
Storage Tank	50	0	-	65	565	565	-	-	-	-	-	-	-	-	-	-	-	1195
Distribution Mains	50	23	-	109	236	471	471	576	464	455	455	601	626	798	798	798	399	7257
Internal Network	50	19	-	169	336	672	672	841	672	672	672	807	605	537	537	537	269	7998
Administration Building[2]	50	43	44	431	481	-	-	-	-	-	-	-	-	-	-	-	-	956
Service Connection:																		
a) Piping	50	0	-	48	48	158	158	158	158	158	158	158	158	158	158	158	158	1992
b) Meter	15	100	-	22	22	71	71	71	71	71	71	71	71	71	71	71	71	[illegible]
Equipment	25	81	388	84	-	-	-	-	-	-	-	-	-	-	-	-	-	472
Total Cost			951	2337	4909	5902	4149	2440	1365	1356	1356	1823	3074	3178	1564	1564	897	36,865
Land			271	-	-	-	-	-	-	-	-	-	-	-	-	-	-	271
Total Project Cost			1222	2337	4909	5902	4149	2440	1365	1356	1356	1823	3074	3178	1564	1564	897	37,136

[1] Includes design (first year of each major segment of development), supervision of construction, contingencies and legal and administrative costs spread uniformly during the period of construction.

[2] Includes meter repair facilities and laboratory equipment.

APPENDIX TABLE XI-B-2

PROJECT COST OF RECOMMENDED PROGRAM
MISAMIS OCCIDENTAL WATER DISTRICT
(WITH ESCALATION)
P x 1000

Item	Service Life	FEC (%)	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	Total
Escalation Factor			1.00	1.12	1.25	1.40	1.57	1.73	1.90	2.09	2.30	2.53	2.74	2.96	3.19	3.45	3.72	
Source Development	50	43	153	497	1110	1243	697	-	-	-	-	-	-	-	-	-	-	3700
Source Transmission Line	50	26	-	191	931	2085	1170	-	-	-	-	-	-	-	-	-	-	4377
Transmission Lines	50	25	366	889	1985	2223	2493	1374	-	-	-	471	4422	4777	-	-	-	19000
Storage Tank	50	0	-	73	706	791	-	-	-	-	-	-	-	-	-	-	-	1570
Distribution Mains	50	23	-	122	295	659	740	997	882	951	1047	1521	1715	2362	2546	2753	1484	18074
Internal Network	50	19	-	189	420	941	1055	1455	1277	1405	1546	2042	1658	1590	1713	1853	1001	18145
Administration Building	50	43	44	483	601	-	-	-	-	-	-	-	-	-	-	-	-	1128
Service Connection:																		
a) Piping	50	0	-	54	60	221	248	273	300	330	363	400	433	468	504	545	588	4787
b) Meter	15	100	-	25	28	99	112	123	135	148	163	180	195	210	227	245	264	2154
Equipment	25	81	388	94	-	-	-	-	-	-	-	-	-	-	-	-	-	482
Total Cost			951	2617	6136	8262	6515	4222	2594	2834	3119	4614	8423	9407	4990	5396	3337	73417
Land			271															
			1222															

APPENDIX TABLE XI-B-3

METER[3] REPLACEMENT SCHEDULE
MISAMIS OCCIDENTAL WATER DISTRICT

Year	New Connections	Converted Flat Rate	Total	Replacement due to Depreciation	Total Conversion and Replacements	1976 Cost Level (₱ x 1000): New Connections	1976 Cost Level (₱ x 1000): Conversion and Replacements	Escalation Factor	Escalated Cost (₱ x 1000): New Connections	Escalated Cost (₱ x 1000): Conversions and Replacements	Total Book Value of Meters (Yr. End and Before Retirement)	Cumulative Book Value	Retirements (Cumulative)	Depreciable Value	Annual Depreciation Expense	Cumulative Depreciation	Assets Added	Net Asset Value Meters
1975			1196					1.0	-	-	227					121	-	106
1976			1196					1.0	5	-	227	227	15	212	14	120	-	92
1977	114	212	1522	80	292	22	55	1.12	25	62	314	227	+15=30	197	13	118	67	166
1978	114	214	1950	80	294	22	56	1.25	28	70	412	314	+15=45	269	18	121	98	246
1979	375	-	2225	80	80	71	15	1.4	99	21	532	412	+15=60	352	23	129	120	343
1980	375	-	2600	80	80	71	15	1.57	112	24	668	532	+15=75	457	30	144	136	449
1981	375	-	2975	80	80	71	15	1.75	123	26	817	668	+15=90	578	39	168	149	559
1982	375	-	3350	80	80	71	15	1.9	135	29	981	817	+15=105	712	47	200	164	676
1983	375	-	3725	80	80	71	15	2.09	148	31	1160	981	+15=120	861	57	242	179	798
1984	375	-	4100	80	80	71	15	2.3	163	35	1358	1160	+15=135	1025	68	295	198	928
1985	375	-	4475	80	80	71	15	2.53	180	38	1576	1358	+15=150	1208	81	361	218	1065
1986	375	-	4850	80	80	71	15	2.74	195	41	1812	1576	+15=165	1411	94	440	236	1207
1987	375	-	5225	80	80	71	15	2.96	210	44	2066	1812	+15=180	1632	109	534	254	1352
1988	375	-	5600	80	80	71	15	3.19	227	48	2341	2066	+15=195	1871	125	644	275	1502
1989	375	-	5975	80	80	71	15	3.45	245	52	2638	2341	+15=210	2131	142	771	297	1657
1990	375	-	6350	80	80	71	15	3.72	264	56	2958	2638	+15=225	2413	161	917	320	1816

[3] Cost of Meter = ₱190

APPENDIX TABLE XI,C-1

PROPOSED STAFFING PLAN
MISAMIS OCCIDENTAL WATER DISTRICT

ORGANIZATIONAL UNIT (Average Salary 1976 Level)	1976-77 1/		1978-79		1981-82		1984-85		1987-88		1990-91	
	Number of Positions	Salaries 2 & Benefits 3	Number of Positions	Salaries & Benefits	Number of Positions	Salaries & Benefits	Number of Positions	Salaries & Benefits	Number of Positions	Salaries & Benefits	Number of Positions	Salaries & Benefits
General Manager's Office	1	23166	2	33977	2	45186	2	60138	2	79845	2	106345
Administrative Div.	9	41128	7	76445	7	101664	8	159339	8	211585	8	281809
Office of the Chief Engineer	2	4325	3	33976	3	45188	3	60137	3	79845	3	106345
Production Division	8	28417	8	67106	8	89244	10	150882	10	191627	10	255230
Construction and Maintenance Division	8	29730	7	47451	8	68907	9	100728	9	133736	9	178125
Commercial Division	15	52200	19	108612	19	144597	23	233027	23	309393	23	412084
Total Positions and Salaries	43	178966	46	367567	47	494786	55	764251	55	1006031	55	1339938

1/ Fiscal year 1976-77 shows projected actual salaries. Reorganization starts in 1978.

2 Salaries shown above indicate increases from current level and implies improved competence/performance and greater responsibilities.

3 All benefits were estimated at 17% of salaries with escalation rate of 10% per annum.

APPENDIX TABLE XI-C-2

SERVICE CONNECTION SCHEDULE
MISAMIS OCCIDENTAL WATER DISTRICT

ITEM	1975	1980	1985	1990
Area Served (ha.)	165	290	400	[illegible]00
Population of Served Area	73,100	85,500	102,000	118,4[illegible]
Population of Service Connections	17,500	26,900	40,000	53,000
NO. OF CONNECTIONS AND ANNUAL BILLED CONSUMPTION (1000 cum)				
Domestic Metered				
3/8" meter	-	-	-	-
1/2" meter	841	1,610	2,771	3,932
3/4" meter	3	6	10	14
1" meter	2	4	7	10
Sub-total	846	1,620	2,788	3,956
Domestic Flat Rate	165	-	-	-
Total Domestic	1,011	1,620	2,788	3,956
Commercial Metered				
3/8" meter	-	-	-	-
1/2" meter	570	915	1,576	2,235
3/4" meter	3	5	8	12
1" meter	2	3	5	7
Sub-total	575	923	1,589	2,254
Commercial Flat Rate	2	-	-	-
Total Commercial	577	923	1,589	2,254
Industrial Metered				
3/8" meter	-	-	-	-
1/2" meter	12	21	35	50
3/4" meter	3	5	8	12
1" meter	2	3	6	8
Sub-total	17	29	49	70
Industrial Flat Rate	1	-	-	-
Total Industrial	18	29	49	70
Governmental/Institutional Metered				
3/8" meter	-	-	-	-
1/2" meter	8	16	28	40
3/4" meter	3	6	10	15
1" meter	2	4	7	10
Bulks	1	2	4	5
Sub-total	14	28	49	70
Govt/Inst't Flat-Rate (hydrants)	30 4/	x	x	x
Total Govt/Inst'l	44	28	49	70
Metered Connections	1,196	1,322	4,475	6,350
Flat Rate/defective meters/ hydrants	454			
Total Service Connections	1,650	2,600	4,475	6,350
Total Billed Consumption per year (1000 cum)	459	1,322	2,103	2,883
Meter Underregistration	25	28	34	40
Unbilled Use and Wastes	286	318	385	454
Leakage	434	482	585	689
Illegal Connections	25	27	34	40
Public Use	12	13	16	18
Total Unbilled Water per year (1000 cum)	782	868(39.6%)	1,054(33.3%)	1,241(30%)
Total Water Production per year (1000 cum)	1,241 5/	2,190	3,157	4,124

4/ hydrants included (26)
5/ historical

APPENDIX TABLE XI.-2-1

ASSET AND DEPRECIATION FORECAST
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1000)

Item	1976	1977	1978	1979	1980	1981	1982	1983
DEPRECIATION VALUES								
50 YEARS SERVICE LIFE								
Existing Facilities	588	588	588	588	588	588	588	588
Source Development	-	-	-	-	-	3700	3700	3700
Source Transmission Lines	-	-	-	-	-	4377	4377	4377
Transmission Lines	-	366	1255	3240	5463	7956	9330	9330
Storage Tank	-	-	-	-	1570	1570	1570	1570
Distribution Mains	-	-	122	417	1076	1816	2813	3695
Internal Network	-	-	189	609	1550	2605	4060	5337
Administration Building	-	-	-	1128	1128	1128	1128	1128
Service Connection (Pipings)	-	-	54	114	335	583	656	1156
Total 50 Yrs. Life	588	954	2208	6096	11710	24323	28422	30881
25 YEARS SERVICE LIFE								
Equipment	-	-	482	482	482	482	482	482
15 YEARS SERVICE LIFE								
Meters	212	197	269	352	457	578	712	861
Total Depreciable Value	800	1151	2959	6930	12649	25383	29616	32224
WORK IN PROCESS								
Source Development	153	650	1760	3003	3700	-	-	-
Source Transmission Line	-	191	1122	3207	4377	-	-	-
Storage Tank	-	73	779	1570	-	-	-	-
Administration Bldg.	44	527	1128	-	-	-	-	-
Equipment	388	482	-	-	-	-	-	-
Total Work In Process	585	1923	4789	7780	8077	-	-	-
ASSETS ADDED BY YEAR END								
Transmission Lines	366	889	1985	2223	2493	1374	-	-
Distribution Mains	-	122	295	659	740	997	882	951
Internal Network	-	189	420	941	1055	1455	1277	1405
Service Connection: a) Piping	-	54	60	221	248	273	300	330
b) Meter	-	87	98	120	136	149	164	179
Total Assets Added	366	1341	2858	4164	4672	4248	2623	2865
ASSETS RETIRED DURING YEAR								
Meters	15	15	15	15	15	15	15	15
BOOK VALUE OF CAPITAL ASSETS BY YEAR END								
Assets Other Than Land	1751	4415	10606	18874	25398	29631	32239	35089
Land	271	271	271	271	271	271	271	271
Total Capital Assets (Book Value)	2022	4686	10877	19145	25669	29902	32510	35360

Item	1984	1985	1986	1987	1988	1989	1990
DEPRECIATION VALUES							
50 YEARS SERVICE LIFE							
Existing Facilities	588	588	588	588	588	588	588
Source Development	3700	3700	3700	3700	3700	3700	3700
Source Transmission Lines	4377	4377	4377	4377	4377	4377	4377
Transmission Lines	9330	9330	9801	14223	19000	19000	19000
Storage Tank	1570	1570	1570	1570	1570	1570	1570
Distribution Mains	4646	5693	7214	8929	11291	13837	16590
Internal Network	6742	8288	10330	11988	13578	15291	17144
Administration Building	1128	1128	1128	1128	1128	1128	1128
Service Connection (Pipings)	1486	1849	2249	2682	3150	3654	4199
Total 50 Yrs. Life	33567	36523	40957	49185	58382	63145	68296
25 YEARS SERVICE LIFE							
Equipment	482	482	482	482	482	482	482
15 YEARS SERVICE LIFE							
Meters	1025	1208	1411	1632	1871	2131	2413
Total Depreciable Value	35074	38213	42850	51299	60735	65758	71191
WORK IN PROCESS							
Source Development	-	-	-	-	-	-	-
Source Transmission Line	-	-	-	-	-	-	-
Storage Tank	-	-	-	-	-	-	-
Administration Bldg.	-	-	-	-	-	-	-
Equipment	-	-	-	-	-	-	-
Total Work In Process	-	-	-	-	-	-	-
ASSETS ADDED BY YEAR END							
Transmission Lines	-	471	4422	4777	-	-	-
Distribution Mains	1047	1521	1715	2362	2546	2753	1484
Internal Network	1546	2042	1658	1590	1713	1853	1001
Service Connection: a) Piping	363	400	433	468	504	545	588
b) Meter	198	218	236	254	275	297	320
Total Assets Added	3154	4652	8464	9451	5038	5448	3393
ASSETS RETIRED DURING YEAR							
Meters	15	15	15	15	15	15	15
BOOK VALUE OF CAPITAL ASSETS BY YEAR END							
Assets Other Than Land	38228	42865	51314	60750	65773	71206	74584
Land	271	271	271	271	271	271	271
Total Capital Assets (Book Value)	38499	43136	51585	61021	66044	71477	74855

APPENDIX TABLE XI-E-2

SCHEDULE OF DEPRECIATION EXPENSES AND ACCUMULATED DEPRECIATION
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1000)

Year	Service Life Category			Total Annual Deprec. Expenses	Accumulated Depreciation Prior Year	Book Value of Assets Retired During the Year				Net Accumulated Depreciation Year End
	50 Yrs.	25 Yrs.	15 Yrs.			50 Yrs.	25 Yrs.	15 Yrs.	Total	
1976	12	-	14	26	300	-	-	15	15	311
1977	19	-	13	32	311	-	-	15	15	328
1978	44	19	18	81	328	-	-	15	15	394
1979	122	19	23	164	394	-	-	15	15	543
1980	234	19	30	283	543	-	-	15	15	811
1981	486	19	39	544	811	-	-	15	15	1340
1982	568	19	47	634	1340	-	-	15	15	1959
1983	618	19	57	694	1959	-	-	15	15	2638
1984	671	19	68	758	2638	-	-	15	15	3381
1985	730	19	81	830	3381	-	-	15	15	4196
1986	819	19	94	932	4196	-	-	15	15	5113
1987	984	19	109	1112	5113	-	-	15	15	6210
1988	1168	19	125	1312	6210	-	-	15	15	7507
1989	1263	19	142	1424	7507	-	-	15	15	8916
1990	1366	19	161	1546	8916	-	-	15	15	10447

APPENDIX TABLE XI.E-3

WORKING CAPITAL REQUIREMENTS FOR REVOLVING FUND FOR NEW CONNECTIONS MISAMIS OCCIDENTAL WATER DISTRICT

Item	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Number of New Connections	114	114	375	375	375	375	375	375	375	375	375	375	375	375
Number Paying Cash	46	46	150	150	150	150	150	150	150	150	150	150	150	150
Number of Installment Plan Added	68	68	225	225	225	225	225	225	225	225	225	225	225	225
No. of Installment Plan Paid	–	–	–	–	–	–	–	–	–	–	34	68	147	225
Total Paying Monthly Installment (Cumulative)	68	136	361	586	811	1036	1261	1486	1711	1936	2127	2284	2362	2362
Monthly Installment Plan (escalated)	₱6.33	7.06	7.91	8.87	9.77	10.74	11.81	13.0	14.29	15.48	16.72	18.02	19.49	21.02
	(₱ x 1000)													
Increment Added[1]	5	6	21	24	26	29	32	35	39	42	45	49	53	57
Increment Deducted[2]	–	–	–	–	–	–	–	–	–	–	3	6	14	23
Cash Receipts:														
Lump Sum Payments (escalated)[3]	22	25	91	102	113	124	136	150	165	178	193	208	225	242
Installment Payments(cumulative)	3	8	22	44	69	97	127	161	198	238	279	320	357	389
Total	25	33	113	146	182	221	263	311	363	416	472	528	582	631
Annual Construction Cost	55	62	228	255	281	309	340	374	412	446	482	519	561	605
Working Capital Required	30	29	115	109	99	88	77	63	49	30	10	(9)	(21)	(26)
Cumulative Capital Requirements	30	59	174	283	382	470	547	610	659	689	699	690	669	643

[1] Accumulated installment payments are calculated on the basis of 100% incremental additions during the previous years and 50% of the last year.

[2] Based on assumption that installment plan will be paid back in 10 years.

[3] Number of connections paying cash x (2/3 366 + 190) x escalation factor.

APPENDIX TABLE XL-E-4

TRIAL FINANCING PLAN AND DEBT SERVICE
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1000)

Fiscal Year	Total Capital Expenditures	Cash Sources: Revolving Fund Revenues	Cash Sources: Amount Disbursed	Loan Disbursements and Debt Service: Outstanding Debt Start of Year	Loan Disbursements and Debt Service: Amortized During Year	Loan Disbursements and Debt Service: Outstanding Debt Year End	Interest	Total Debt Service
1976	1222	-	1222	-	-	1222	-	
1977	2679	25	2654	1222	-	3876	110	110
1978	6206	33	6173	3876	-	10049	349	349
1979	8283	113	8170	10049	-	18219	904	904
1980	6539	146	6393	18219	-	24612	1640	1640
1981	4248	182	4066	24612	185	28493	2215	2400
1982	2623	221	2402	28493	185	30710	2564	2749
1983	2865	263	2602	30710	185	33127	2764	2949
1984	3154	311	2843	33127	381	35589	2981	3362
1985	4652	363	4289	35589	381	39497	3203	3584
1986	8464	416	8048	39497	381	47164	3555	3936
1987	9451	472	8979	47164	939	55204	4245	5184
1988	5038	528	4510	55204	939	58775	4968	5907
1989	5448	582	4866	58775	1704	61937	5290	6994
1990	3393	631	2762	61937	1704	62995	5574	7278

APPENDIX TABLE XI-E-5

FORECAST OF CASH REQUIREMENTS
MISAMIS OCCIDENTAL WATER DISTRICT

	1976	1980	1985	1990
Operating Cost	315	1045	1672	3353
Working Capital	-	109	49	(26)
Debt Service	-	1640	3584	7278
Sub-Total	315	2794	5305	10605
Reserve Funds	9	84	318	1061
Uncollectibles	6	56	106	212
Approximate Revenue Requirements (000)	330	2934	5729	11878
Estimated No. of Connections	1414	2600	4475	6351
Ave. Annual Cost Per Connection	233	1128	1280	1870
Est. Quantity of Billable Water (000)	632	1320	2100	2900
Ave. Cash Requirement per Billable Cum	.52	2.22	2.73	4.10
Escalated Income of ₱440/Mo. Household	440	645	1030	1670
Proportion of Income of Low Income Household Devoted to Water, Assuming 20 cum/mo Consumption (%)	2.36	6.9	5.3	4.9

APPENDIX TABLE XI-E-6
REVENUE UNIT FORECAST
MISAMIS OCCIDENTAL WATER DISTRICT

pe of Connection by meter size	1975 No. of Connections[4]	1975 Prop. Consumption[5]	1975 Estimated Consumption[6]	1975 Use Factor	1975 Total R.U.s.[7]	1980 No. of Connections	1980 Estimated Consumption	1980 Total R.U.s.	1985 No. of Connections	1985 Estimated Consumption	1985 Total R.U.s.	1990 No. of Connections	1990 Estimated Consumption	1990 Total R.U.s.
mestic & Government														
1/2"	849	97	1106	1	1106	1626	3133	3133	2799	4986	4986	3972	6839	683
3/4"	6	1	11	1	11	12	32	32	20	51	51	29	70	7
1"	4	2	23	1	23	8	65	65	14	103	103	20	141	14
SUB-TOTAL	859	100	1140		1140	1646	3230	3230	2833	5140	5140	4021	7050	705
mercial & Industrial														
1/2"	582	81.7	98	2	196	836	319	638	1611	507	1014	2285	695	139
3/4"	6	1.3	2	2	4	10	5	10	16	8	16	24	11	2
1"	4	1.8	2	2	4	6	7	14	11	11	22	15	15	3
Bulks 4"	1	15.2	18	3	54	2	59	177	4	94	282	5	129	38
SUB-TOTAL	593	100.0	120		258	954	390	839	1642	620	1334	2329	850	182
TOTAL	1,19[8]		1260		1398	2600	3620	4069	4475	5760	6474	6350	7900	887

[4] 1975 figures are actual; 1980, 85 and 90 are estimated with the proportion or connections in each size remaining constant.

[5] Proportion of consumption based on Flow Relationship.

[6] Based on Table VI-7.

[7] Includes both "service R.U.s." and "commodity R.U.s."; The effect of minimum monthly charges will be to increase total R.U.s. since there will always be some customer not using the basic quantity of water allowed within the minimum price.

[8] 256 defective meters were excluded from total.

APPENDIX TABLE XI-E-7

REVENUE FORECAST
MISAMIS OCCIDENTAL WATER DISTRICT

Year	Rate/RU ₱	Estimated Number of R.U.s. (Yearly in 000s)	Income From Sales	Other Income[9] (000)	Total Income (000)
1976	1.00	691	691	21	712
1977	1.00	891	891	27	918
1978	1.00	1084	1084	33	1117
1979	1.90	1290	2451	74	2525
1980	1.90	1485	2822	85	2907
1981	1.90	1652	3139	94	3233
1982	2.50	1828	4570	137	4707
1983	2.50	2004	5010	150	5160
1984	2.50	2188	5470	164	5634
1985	3.20	2363	7562	227	7789
1986	3.20	2530	8096	243	8339
1987	3.20	2706	8659	260	8919
1988	4.50	2881	12965	389	13354
1989	4.50	3065	13793	414	14207
1990	4.50	3240	14580	437	15017

[9] Other Income (derived from meter replacement charges, contingency fees of new connections, service fees, etc) estimated just about 3% of sales.

APPENDIX TABLE XI-E-8

FINANCING PLAN AND DEBT SERVICE
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1000)

Fiscal Year	Total Capital Expenditures	Cash Sources: Revolving Fund Revenues	Operating Income	Amount Disbursed	Loan Disbursements and Debt Service: Outstanding Debt Start of Year	Amortized During Year	Outstanding Debt Year End	Interest	Total Debt Servi
1976	1222	-	-	1222	-	-	1222	-	-
1977	2679	25	-	2654	1222	-	3876	110	110
1978	6206	33	-	6173	3876	-	10049	349	349
1979	8283	113	-	8170	10049	-	18219	904	904
1980	6539	146	-	6393	18219	-	24612	1640	1640
1981	4248	182	-	4066	24612	185	28493	2215	2400
1982	2623	221	-	2402	28493	185	30710	2564	2749
1983	2865	263	-	2602	30710	185	33127	2764	2949
1984	3154	311	-	2843	33127	381	35589	2981	3362
1985	4652	363	-	4289	35589	381	39497	3203	3584
1986	8464	416	-	8048	39497	381	47164	3555	3936
1987	9451	472	-	8979	47164	939	55204	4245	5184
1988	5038	528	4510	-	55204	939	54265	4968	5907
1989	5448	582	4866	-	54265	1574	52691	4884	6458
1990	3393	631	2762	-	52691	1574	51117	4742	6316

APPENDIX TABLE XI-E-9

PROJECTIONS OF FINANCIAL STATEMENTS
MISAMIS OCCIDENTAL WATER DISTRICT
(P x 1000)

Item	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Water Sales	691	891	1084	2451	2822	3139	4570	5010	5470	7562	8096	8659	12965	13793	14580
Less: Provision for Bad Debt	(14)	(9)	(11)	(49)	(28)	(31)	(91)	(50)	(55)	(151)	(81)	(87)	(259)	(138)	(146)
Other Income	21	27	33	74	85	94	137	150	164	227	243	260	389	414	437
Total Revenue	698	909	1106	2476	2879	3202	4616	5110	5579	7638	8258	8832	13095	14069	14871
Less: Operating Cost	315	349	536	588	1045	1148	1085	1191	1511	1672	2057	2366	2651	2898	3353
Income Before Depreciation	383	560	570	1888	1834	2054	3531	3919	4068	5966	6201	6466	10444	11171	11518
Depreciation	26	32	81	164	283	544	634	694	758	830	932	1112	1312	1424	1546
Net Operating Income	357	528	489	1724	1551	1510	2897	3225	3310	5136	5269	5354	9132	9747	9972
Plus: Interest on Reserves	1	4	8	15	26	45	77	117	161	216	304	421	572	759	958
Income Before Interest on Long Term Loans	358	532	497	1739	1517	1555	2974	3342	3471	5352	5573	5775	9704	10506	10930
Interest	–	110	349	904	1640	2215	2564	2764	2981	3203	3555	4245	4968	4884	4742
Net Income (Loss)	358	422	148	835	(63)	(660)	410	578	490	2149	2018	1530	4736	5622	6188
Cumulative Net Income (Loss)	358	780	928	1763	1700	1040	1450	2028	2518	4667	6685	8215	12951	18573	24761
Appropriations to Reserves	21	27	33	74	85	188	274	301	328	454	810	866	1297	1379	1458

APPENDIX TABLE XI- B-10

CASH FLOW STATEMENTS
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1000)

Sources of Funds	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Net Income (Before Interest)	358	532	497	1739	1577	1555	2974	3342	3471	5352	5573	5775	9704	10506	10930
Depreciation	26	32	81	164	283	544	634	694	758	830	932	1112	1312	1424	1546
Increase in Current Liab.	5	5	31	9	76	17	(10)	18	53	27	64	51	48	41	76
Total Internal Sources	389	569	609	1912	1936	2116	3598	4054	4282	6209	6569	6938	11064	11971	12552
Long Term Debt	1222	2654	6173	8170	6393	4066	2402	2602	2843	4289	8048	8979	-	-	-
Capital Contributions	-	25	33	113	146	182	221	263	311	363	416	472	528	582	631
Total External Sources	1222	2679	6206	8283	6539	4248	2623	2865	3154	4652	8464	9451	528	582	631
Total Sources	1611	3248	6815	10195	8475	6364	6221	6919	7436	10861	15033	16389	11592	12553	13183
Application of Funds															
Investment in Utility Plant	1222	2679	6206	8283	6539	4248	2623	2865	3154	4652	8464	9451	5038	5448	3393
Interest on Debt	-	110	349	904	1640	2215	2564	2764	2981	3203	3555	4245	4986	4884	4742
Principal Repayment	-	-	-	-	-	185	185	185	381	381	381	939	939	1574	1574
Total Capital Charges	1222	2789	6555	9187	8179	6648	5372	5814	6516	8236	12400	14635	10945	11906	9709
Increase in Current Assets	109	77	50	411	119	91	359	135	131	523	171	158	1053	260	226
Total Applications	1331	2866	6605	9598	8298	6739	5731	5949	6647	8759	12571	14793	11998	12166	9935
Net Increase (Decrease)	280	382	210	597	177	(375)	490	970	789	2102	2462	1596	(406)	387	3248
Cash at Beginning of Period	20	300	682	892	1489	1666	1291	1781	2751	3540	5642	8104	9700	9294	9681
Cash At End of Period	300	682	892	1489	1666	1291	1781	2751	3540	5642	8104	9700	9294	9681	12929

APPENDIX TABLE II-E-11

PROJECTED BALANCE SHEET
MISAMIS OCCIDENTAL WATER DISTRICT
(P x 1000)

Assets	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
FIXED ASSETS															
Gross Value of Fixed Assets	1437	2763	6088	11365	17592	29902	32510	35360	38499	43136	51585	61021	66044	71477	74855
Less: Accumulated Depreciation	311	328	394	543	811	1340	1959	2638	3381	4196	5113	6210	7507	8916	10447
Net Value of Fixed Assets	1126	2435	5694	10822	16781	28562	30551	32722	35118	38940	46472	54811	58537	62561	64408
Work in Process	585	1923	4789	7780	8077	-	-	-	-	-	-	-	-	-	-
Total Fixed Assets	1711	4358	10483	18602	24858	28562	30551	32722	35118	38940	46472	54811	58537	62561	64403
CURRENT ASSETS															
Cash	300	682	892	1489	1666	1291	1781	2751	3540	5642	8104	9700	9294	9681	12929
Accounts Receivables	173	223	271	615	706	785	1143	1253	1368	1891	2024	2165	3241	3448	3645
Less: Provision for Uncollectibles	(3)	(2)	(3)	(12)	(7)	(8)	(23)	(13)	(14)	(38)	(20)	(22)	(65)	(34)	(36)
Inventories	1	27	30	108	129	142	158	173	190	214	234	253	273	295	326
Total Current Assets	471	930	1190	2198	2494	2210	3059	4164	5084	7709	10342	12096	12743	13390	16864
Total Assets	2182	5288	11673	20800	27352	30772	33610	36886	40202	46649	56814	66907	71280	75951	81272
EQUITY & LIABILITIES															
EQUITY															
Government Contributions	549	549	549	549	549	549	549	549	549	549	549	549	549	549	549
Capital Contributions	-	25	58	171	317	499	720	983	1294	1657	2073	2545	3073	3655	4286
Reserves	21	48	81	155	240	428	702	1003	1331	1785	2595	3461	4758	6137	7595
Unappropriated Retained Earnings	337	732	847	1608	1460	612	748	1025	1187	2882	4090	4754	8193	12436	17166
Total Equity	907	1354	1535	2483	2566	2088	2719	3560	4361	6873	9307	11309	16573	22777	29596
LONG TERM DEBT															
Long Term Loans (Less Current Maturities)	1222	3876	10049	18219	24427	28308	30525	32746	35208	39116	46225	54265	52691	51117	49543
CURRENT LIABILITIES															
Accounts Payable	53	58	89	98	174	191	181	199	252	279	343	394	442	483	559
Current Maturities of Long Term Debt	-	-	-	-	185	185	185	381	381	381	939	939	1574	1574	1574
Total Current Liabilities	53	58	89	98	359	376	366	580	633	660	1282	1333	2016	2057	2133
Total Equity & Liabilities	2182	5288	11673	20800	27352	30772	33610	36886	40202	46649	56814	66907	71280	75951	81272

APPENDIX TABLE XI-E-12

FORECASTED RATE OF RETURN ON NET FIXED ASSETS IN SERVICE
MISAMIS OCCIDENTAL WATER DISTRICT
(₱ x 1,000,000)

Item	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Average Net Fixed Assets in Service	.82	1.78	4.06	8.26	13.80	22.67	29.56	31.64	33.92	37.03	42.71	50.64	56.67	60.55	63.48
Net Operating Income	.36	.53	.49	1.72	1.55	1.51	2.90	3.23	3.31	5.14	5.27	5.35	9.13	9.75	9.97
Rate of Return (%)	43.9	29.7	12.1	20.8	11.2	6.7	9.8	10.2	9.8	13.9	12.3	10.5	16.1	16.1	15.7

MEMORANDUM OF UNDERSTANDING

1. The Asian Development Bank (ADB) Appraisal Mission for the proposed Provincial Cities Water Supply Project (the Project) visited the Misamis Occidental Water District (the Water District) on 24 October 1975. Subject to the approval of the Government of the Philippines, the Local Water Utilities Administration (LWUA) and the management of ADB, the following matters concerning the Project as related to the Water District are agreed upon:

Scope of the Project

2. The part of the Project involving the Water District (the Sub-Project) consists of:

(a) Development of the Cocok, Regina and Bitoon Springs together with improvements to the Talibaksan Spring to provide a reliable yield of 10,400 CMD sufficient for the projected demand up to 1986. Treatment by controlled chlorination with all equipment provided with standby units.

(b) A 1,900 m^3 capacity storage tank.

(c) Transmission mains:- 250,200 and 150 mm dia. (Total length 15.2 km)

(d) Distribution mains:- 200 and 150 mm dia. (Total length 5.2 km).

(e) Internal network mains:-

(i) reinforcement:- 150 and 100 mm dia. (Total Length 2.1 km)

(ii) extension:- 150 and 100 mm dia. (Total length 11.5 km)

(f) 17 fire hydrants.

(g) 1,500 new service connections and replacement of 200 existing connections.

(h) Administration building, meter repair, and laboratory facilities.

(i) Recruitment of consultant services as specified in paragraph 6(a) through (c) below.

3. The Sub-Project is at present estimated to cost ₱31,000 million.

Relending Arrangement

4. The relevant part of the proposed ADB loan will be relent to the Water District through the Government and LWUA. The Water District should arrange with the Government and/or LWUA for the balance of funds required for the implementation of the Sub-Project. The ADB loan to the Government will be from the ordinary capital resources of the Bank and extended at the prevailing rate of interest at the time of loan approval by ADB Board. The terms and conditions of relending to the Water District would be the standard LWUA lending terms which are generally satisfactory to the Bank. The relending agreement shall be approved by the Bank.

Execution of the Project.

5. LWUA shall be responsible for the overall implementation of the Project including procurement on behalf of the Water District. However, the Water District shall retain ultimate responsibility in the carrying out of the Sub-Project and will work closely with LWUA and the consultants. When appropriate, certain aspects of the Sub-Project may be implemented directly by the Water District.

Consultants:

6. The consultants to be engaged by LWUA on behalf of the Water District shall look into, among other things, the following aspects of the Project.

(a) Preparation of detailed designs, tender documents, assistance in the prequalification of tenderers and analysis of bids and recommendations for the award of contracts for the first construction phase works as detailed by the consultants, Messrs. Camp Dresser & McKee International Inc.

(b) Supervision of construction, installation, and commissioning of the supply source works, transmission lines and storage tanks and the training of the staff of LWUA and the Water District to carry out the functions for the extensions to the distribution and internal network mains after 1978, and the transmission lines after 1900.

(c) Development of appropriate leakage control, metering and data collection program including hydrological studies of existing and potential supply sources to establish their reliable yield and the training of the Water District staff in these respects.

Financial Measures

7. The Water District undertakes to:

(a) Appropriately adjust water rates with a view to cover all operating and maintenance expenses, debt amortization and a reasonable proportion of its capital expenditures program. The water rate is expected to be increased to ₱1/cum (weighted average) per cubic meter by 31 December 1976.

(b) Obtain concurrence of LWUA and ADB prior to undertaking any major investment program outside the Project;

(c) Have its accounts audited annually by an independent auditor acceptable to ADB and submit its financial statements to the Bank for review.

Operations

8. The Water District will:

(a) Establish leakage control and metering programs, and collection of other necessary data required for the operation and future development of the water system;

(b) Improve billing and collection procedures;

(c) Institute laboratory facilities for continuous water analysis;

(d) Maintain its water supply facilities in accordance with sound public utility practices;

(e) Not dispose of its assets required for its efficient operation and implementation of the Project without prior approval of LWUA and the Bank.

Land and Water Rights

9. The Water District shall exert its best efforts to ensure that the necessary land, water rights, or other rights are obtained for the implementation of the Project and shall include in its quarterly reports to LWUA and the Bank a statement of the progress

made on the acquisition of land and water rights.

October 24, 1975

MISAMIS OCCIDENTAL WATER DISTRICT

By:

(SGD)
DR. SOLOMON J. GUIRNELA, M.D.
(Chairman of the Board of Directors)

ASIAN DEVELOPMENT BANK
APPRAISAL MISSION

By:

(SGD)
GERHARD H. KAHL
(Mission Head)

REPUBLIC OF THE PHILIPPINES

MISAMIS OCCIDENTAL WATER DISTRICT
Office of the Board of Directors
CITY OF OZAMIZ

EXCERPTS FROM THE MINUTES OF THE SPECIAL MEETING OF THE BOARD OF DIRECTORS, MISAMIS OCCIDENTAL WATER DISTRICT, HELD AT OZAMIZ CITY, ON OCTOBER 20, 1975.

PRESENT:	Dr. Solomon J. Guirnela	Chairman
	Dr. Jose P. Meñez	Vice-Chairman
	Mr. J. Antonio Lim	Member
	Mrs. Eufrocina Y. Tan	Member
	Engr. Violeta C. Calicinao	General Manager
	Atty. Yolando Villarus	Legal Counsel
ABSENT:	Dr. Oscar Remulla	Member

RESOLUTION NO. 100

WHEREAS, an ADB appraisal team is coming to Ozamiz City to confer with the General Manager and the Board of Directors of the Misamis Occidental Water District regarding the comprehensive development of Ozamiz-Clarin Waterworks System, per Memorandum from the General Manager of LWUA;

WHEREAS, the purpose of the team is to finalize agreements which will be covered in the Memorandum of Understanding between ADB and MOWD;

WHEREAS, the Board in meeting assembled, unanimously

RESOLVED, as it does hereby resolve to authorize the Chairman, Board of Directors, this District, to sign the Memorandum of Understanding.

RESOLVED further to furnish copies of this resolution to the ADB Team, the General Manager, this District, and files.

I hereby certify to the correctness of the above-quoted resolution.

(SGD) LIGAYA T. DONGGON
Secretary

APPROVED:

(SGD)
SOLOMON J. GUIRNELA
Chairman

APPENDIX XI-F-2

LOCAL WATER UTILITIES ADMINISTRATION

Asian Development Bank
P. O. Box 789
Manila, Philippines

Dear Sirs:

Re: Loan No. PHI: Provincial Cities Water Supply Project - Allocations of Proceeds of the Loan; Withdrawals

1. We refer to Sections 3.02 and 8.01 of the Loan Agreement between the Republic of the Philippines (the Borrower) and the Bank of even date herewith and attach hereto a table showing an allocation of the proceeds of the Loan to which we request your agreement.

2. We confirm that the amount allocated to Category III includes the amount of US$3,600,000 for financing local currency expenditures. We understand that the Bank will disburse in foreign currency the equivalent of 38% of each local currency payment made by us under the contracts for the category, subject to the ceiling of US$3,600,000.

3. Some of the items included in Categories I to V may be imported goods purchased from local suppliers or may be goods fabricated by local manufacturers from imported components and raw materials. With respect to contracts involving any such local procurement, we confirm that where evidence of the actual foreign exchange cost is not available, 45% of the contract price (or such other percentage as shall be agreed between the Bank and us from time to time) will apply for the purpose of withdrawals from the loan account under Section 3.03(a)(ii) of the Loan Agreement in respect of such contracts. In support of applications for withdrawals in respect of the foreign exchange cost of such contracts, we shall supply a copy of the local supplier's invoice indicating, where possible, the origin of the imported items or goods, and evidence that payment has been made to the local supplier.

4. If the foreign exchange cost of the items in any of the Categories I to VII should increase, an amount equal to such increase will be reallocated by the Bank, at our request, to such Category from Category IX, subject, however, to the requirements for contingencies, as determined by the Bank, in respect of the foreign exchange cost of the items in the other Categories. If the foreign exchange cost of the items in any of the Categories I to VIII should decrease, the amount of the Loan then allocated to and

no longer required for, such Category will be reallocated by the Bank to Category IX.

5. On the basis of the attached Table, the amounts to be allocated to the various Water Districts for the Project would be as follows:

(a) Zamboanga City Water District: $4,036,000 together with that portion of training program attributable to it.

(b) Misamis Occidental Water District: $1,749,000 together with that portion of training program attributable to it.

(c) Butuan City Water District: $2,953,000 together with that portion of training program attributable to it.

(d) Camarines Norte Water District: $4,171,000 together with that portion of training program attributable to it.

Accordingly, and subject to possible reallocation pursuant to paragraph 4 above, the amount specified above for the respective Water Districts will be made available to the Water Districts under the respective Relending Agreements referred to in Section 3.01(a) of the Loan Agreement.

6. Please indicate your agreement with the foregoing by signing the confirmation form on the enclosed copy of this letter and returning it to us.

Yours faithfully,

LOCAL WATER UTILITIES
ADMINISTRATION

By ______________________
Authorized Representative

CONFIRMED:

ASIAN DEVELOPMENT BANK

By ______________________

ALLOCATION OF PROCEEDS OF LOAN

(Provincial Cities Water Supply Project)

	Category	Amount (U.S. Dollar Equivalent)
I.	Construction Materials for Supply Source Works and Storage Tanks.	$ 122,000
II.	Pumps and Treatment Works.	1,551,000
III.	Pipelines including Fire Hydrants.	7,829,000
IV.	Service Connections.	263,000
V.	Administration Buildings, Office Equipment, Laboratory, Meters, Meter Repair Facilities, Leakage Detection Equipment and Vehicles.	507,000
VI.	Consulting Services	2,578,000
VII.	Training Program	200,000
VIII.	Interest and Other Charges during Construction	2,700,000
IX.	Unallocated	1,050,000
	Total	$16,800,000

APPENDIX XI-F-3

SCHEDULE I

Description of the Project

The Project consists of the expansion and improvement of the water supply facilities of the four Water Districts to meet the requirements up to 1985, and the interim improvement and expansion of the existing water supply facilities and the detailed engineering design of a major water supply source for another Water District, all as described below:

1. **Misamis Occidental Water District**

Development of existing and new supply sources to provide a total reliable yield of about 10,400 CDM; installation of new pumps; provision of chlorinators with standby units; construction of a 1,900 m^3 capacity storage tank; construction of transmission, distribution and internal network mains, fire hydrants and service connections; provision of administration building, office equipment, laboratory, meters, meter repair facilities, leakage detection equipment and vehicles.

2. **Butuan City Water District**

Development of wells to provide reliable yield of about 11,400 CMD; provision of chlorinators with standby units; construction of transmission, distribution and internal network mains, fire hydrants, and service connections; provision of administration building, office equipment, laboratory, meters, meter repair facilities, leakage detection equipment and vehicles.

3. **Zamboanga City Water District**

Construction of a new diversion dam, intake structure, fine screen/grit chamber, raw water transmission main; rejuvenation and extension of the existing treatment works up to a capacity of about 32,700 CMD including converting the existing sedimentation tanks into storage tanks; construction of transmission, distribution, and internal network mains, fire hydrants; provision of service connections, office equipment, meters, meter repair facilities, leakage detection equipment and vehicles.

4. Camarines Norte Water District

Development of existing and new supply sources to provide a total reliable yield of about 21,000 CMD; provision of chlorinators with standby units; construction of 1,000 m^3 and 400 m^3 capacity storage tanks; construction of transmission and internal network mains, fire hydrants and service connections; provision of office equipment, laboratory facilities, meters, meter repair facilities, leakage detection equipment and vehicles.

5. Metropolitan Cebu Water District

Procurement and installation of chlorinators for existing supply sources; development of new wells and existing spring supply sources to provide a total reliable yield of about 35,000 CMD; construction of a new transmission main and improvement of existing transmission mains; provision of fire hydrants and meter repair and laboratory facilities; conduct of a leakage survey; and for the development of a major dam supply source, acquisition of the necessary land, construction of a road to the proposed dam site, and provision of hydrological and meteorological study equipment and facilities.

6. Services of Consultants relating to (1) to (5) above.

7. Overseas Training Program for LWUA and Water District Staff.

The Project is expected to be completed by 30 June 1981.

SCHEDULE 2

Amortization Schedule

Date Payment Due	Payment of Principal (expressed in dollars)*
1 March 1982	$161,700
1 September 1982	168,800
1 March 1983	176,200
1 September 1983	183,900
1 March 1984	192,000
1 September 1984	200,400
1 March 1985	209,100
1 September 1985	218,300
1 March 1986	227,800
1 September 1986	237,800
1 March 1987	248,200
1 September 1987	259,000
1 March 1988	270,400
1 September 1988	282,200
1 March 1989	294,500
1 September 1989	307,400
1 March 1990	320,900
1 September 1990	334,900
1 March 1991	349,600
1 September 1991	364,900
1 March 1992	380,800
1 September 1992	397,500
1 March 1993	414,900
1 September 1993	433,000

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*To the extent that any part of the Loan is repayable in a currency other than dollars (see Loan Regulations Section 3.03), the figures in this column represent dollar equivalents determined as for purposes of withdrawal.

Date Payment Due	Payment of Principal (expressed in dollars)*
1 March 1994	$452,000
1 September 1994	471,800
1 March 1995	492,400
1 September 1995	513,900
1 March 1996	536,400
1 September 1996	559,900
1 March 1997	584,400
1 September 1997	610,000
1 March 1998	636,600
1 September 1998	664,500
1 March 1999	693,600
1 September 1999	723,900
1 March 2000	755,600
1 September 2000	788,600
1 March 2001	823,100
1 September 2001	859,100
Total	$16,800,000

*To the extent that any part of the Loan is repayable in a currency other than dollars (see Loan Regulations Section 3.03), the figures in this column represent dollar equivalents determined as for purposes of withdrawal.

PREMIUMS ON PREPAYMENT AND REDEMPTION

The following percentages are specified as the premiums payable on prepayment in advance of maturity of any part of the principal amount of the Loan pursuant to Section 2.05(b) of the Loan Regulations or on the redemption of any Bond prior to its maturity pursuant to Section 6.16 of the Loan Regulations.

Time of Prepayment or Redemption	Premium
Not more than 3 years before maturity	1.50%
More than 3 years but not more than 6 years before maturity	3%
More than 6 years but not more than 11 years before maturity	4.5%
More than 11 years but not more than 17 years before maturity	6%
More than 17 years but not more than 21 years before maturity	7%
More than 21 years but not more than 24 years before maturity	8%
More than 24 years before maturity	8.75%