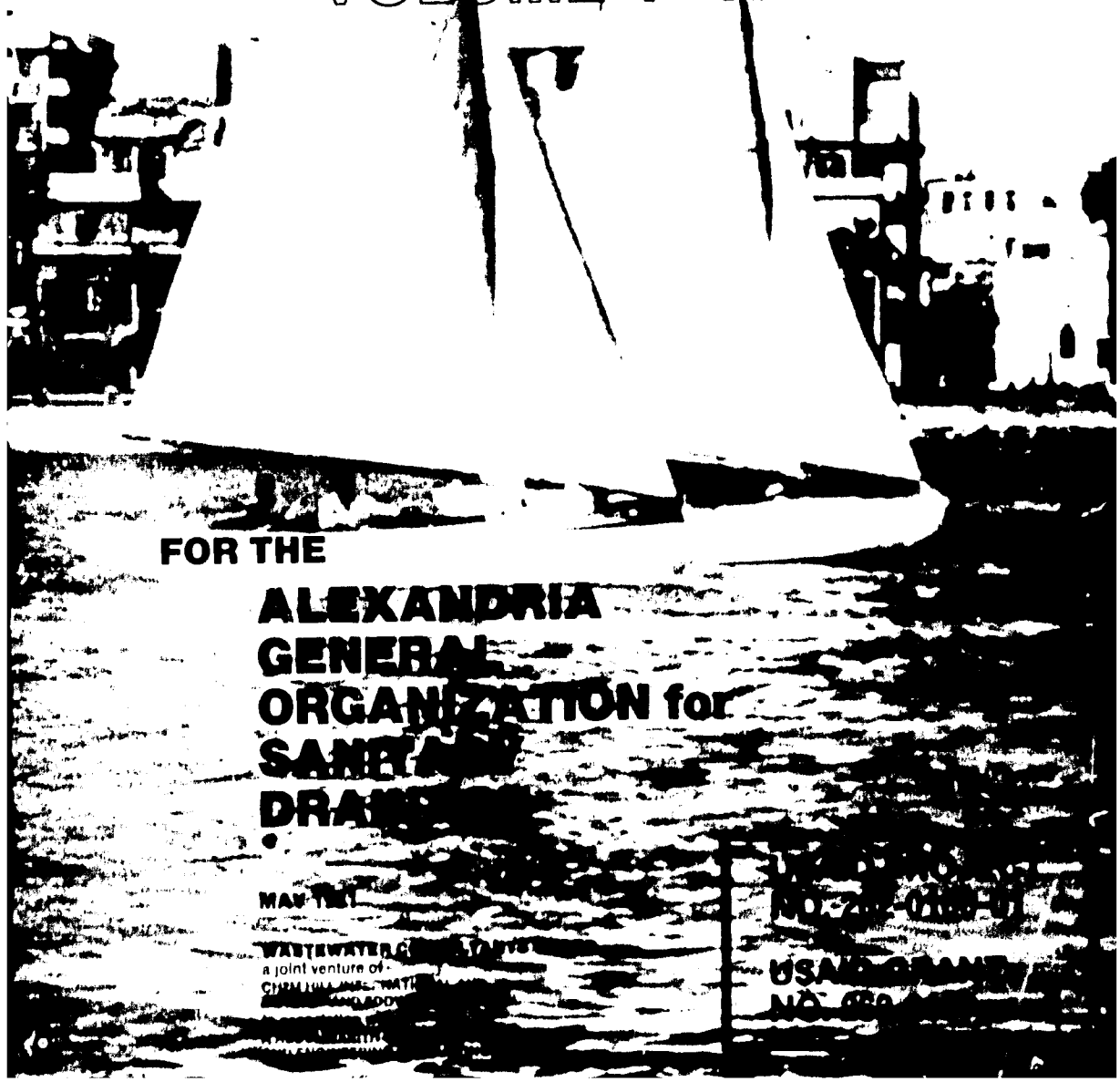
REVIEW AND UPDATE

OF THE

1978 ALEXANDRIA WASTEWATER MASTER PLAN

VOLUME 1 REPORT



FOR THE

ALEXANDRIA GENERAL ORGANIZATION for SANITARY DRAINAGE

MAY 1981

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CHAPTER 1

INTRODUCTION

PURPOSE AND SCOPE OF STUDY

This report summarizes the results of studies undertaken by the Wastewater Consultants Group (WWCG), in association with Dr. A. Abdel Warith Consulting Engineers and Engineering Consultants Group, to review and update the "Alexandria Wastewater Master Plan Study" completed in 1978.

This study has several purposes:

- Reevaluate sewage treatment alternatives.
- Reevaluate the advisability of sea disposal preceded by pri-treatment in comparison with other possible alternatives.
- Investigate the handling and disposal of sludge generated by treatment facilities.
- Reevaluate the siting and number of treatment plants needed for the wastewater service areas of Alexandria.

The scope of work to accomplish the objectives stated above includes the following tasks as related to a study area composed of the eastern, western, and central wastewater service areas of Alexandria:

- Review existing data and documents.
- Evaluate the support data that formed the basis for the 1978 Master Plan.
- Participate in technical meetings with interested individuals and organizations.
- Determine the impact of revising the treatment and disposal alternatives in the top priority and Phase I projects.
- Review certain high-priority projects currently scheduled.
- Investigate improvements planned for the existing East and West Treatment Plants.
- Provide a "Master Plan Review" report containing the evaluation, findings, and recommendations of WWCG.

In addition to the scope of work necessary to review and uodate the 1978 Master Plan, WWCG's scope of work includes certain special studies concerning the future extension of sewer services to outside areas west of the Noubaria Canal and to Mamoura and Abu Qir east of the study area. The results of these special studies are contained in a separate report entitled "Special Report on Sewer Services for Outside Areas."

This report is intended to be used in conjunction with the existing Alexandria Master Plan documents. It is not a completely new master plan and is not so intended. It is primarily intended to reevaluate the 1978 recommended plan in the light of changed conditions (especially the need to add primary treatment) and to reinvestigate the possibility of land application of wastewater. Finally, this report is intended to provide recommendations to the Alexandria General Organization for Sanitary Drainage (A/GOSD) to assist in the selection of appropriate methods to provide sewerage services to Alexandria.

REPORT ORGANIZATION

This report is organized into seven chapters and thirteen technical appendices. Figures and tables are numbered consecutively within chapters (appendices). Each is designated by an Arabic numeral (English letter) followed by an Arabic numeral. The Arabic numeral (English letter) designates the chapter (appendix) in which the figure/table is presented. The Arabic numeral designates the order of presentation within the chapter (appendix). Similarly, pages are consecutively numbered within a chapter (appendix); e.g., Table J-4 is the fourth table presented in Appendix J, and page 2-4 is the fourth page of Chapter 2.

Appendices

The 13 technical appendices contain detailed information concerning particular subjects addressed in the report. The seven chapters are more general in nature and are used to synthesize and evaluate the minutiae of data presented in the appendices.

Appendix A, "Facilities Design Criteria Update," discusses the design life of facilities, wastewater flow characteristics, collection system design, wastewater system design, storm sewer system design and sludge management.

Appendix B, "Cost Data Base Update," presents the updated cost data base used in this report, including cost analysis techniques, basic cost assumptions, and the conceptual methodology used to produce cost information.

Appendix C, "Planning Considerations Update," presents the results of the 1980 CAPMAS census, provides updated future population estimates by census tract, identifies present and possible future land uses, and contains projected sewage flows and characteristics.

Appendix D, "Industrial Update," discusses industrial data and waste flow projections. This appendix also addresses the impact of industrial

wastes on treatment and disposal of wastewater, as well as sludge management. It provides information on limits of potentially toxic industrial waste constituents and contains recommendations concerning pretreatment requirements and sampling programs.

Appendix E, "East Treatment Plant Evaluation," presents WWCG's evaluation of the existing East Treatment Plant. This appendix evaluates the plant in light of the ongoing rehabilitation program, recommends certain additional changes, and estimates capital and operation/ maintenance costs for the recommended program.

Appendix F, "West Treatment Plant Evaluation," presents WWCG's evaluation of the Alexandria West Treatment Plant, which is currently under construction. The appendix's contents are similar to those described for Appendix E.

Appendix G, "Pump Station Review," contains an evaluation of the existing major pump stations and force mains in the Alexandria sewerage system. The evaluation addresses mechanical systems, instrumentation and control, electrical systems, operation, maintenance, and safety. Recommended improvements are noted, along with capital cost estimates for such improvements.

Appendix H, "Collection System Review," describes the existing system and its condition, reviews collection system improvements recommended by the 1978 Master Plan, evaluates top priority projects, and presents recommended short-term improvements and capital cost estimates for improvements.

Appendix I, "Environmental Data Review," updates and discusses environmental impacts and potential corrective measures associated with the sewage disposal alternatives discussed in Appendices J and K.

Appendix J, "Treatment Alternatives," presents the rationale used to evaluate and select both liquid treatment processes and sludge management processes. Potential treatment/disposal alternatives are formulated and discussed. Those alternatives judged most appropriate are discussed in detail sufficient to compare the alternatives in Chapter 6.

Appendix K, "Land Application of Wastewater," discusses various means of land application of wastewater. It begins with a detailed review of the 1978 Master Plan land application alternatives. This is followed by detailed sections including basic concepts of land application, investigation of potential sites, development of site-specific land application systems, and evaluation of those systems. Finally, the most appropriate land application systems are selected with sufficient detail to permit comparison with other alternatives in Chapter 6.

Appendix L, "Sea Disposal and Outfall Review," presents an updated review of discharging effluent into the Mediterranean Sea via long outfalls after primary treatment. Updated outfall construction costs are presented, along with projections of the impact of this disposal system on the environment.

Appendix M, "Conveyance System Alternatives," presents a detailed discussion of the overall sewage conveyance systems that would be associated with the most appropriate treatment and disposal alternatives developed and discussed in Appendices J and K. Necessary systems and pump stations are shown along with capital, operation, and maintenance costs.

Appendix N, "Economic Analysis," presents an economic analysis of the three wastewater treatment and disposal alternatives that appear to be the most attractive on the basis of the cost-effectiveness analysis. The incremental benefits and costs for each land application alternative are evaluated to provide Egyptian and USAID officials with a basis for deciding which alternative would maximize benefits to Egyptian society as a whole.

Report Chapters

The seven report chapters are:

1. Introduction
2. Planning Considerations
3. Existing Situation
4. Sea Disposal Alternative IB
5. Land Application Alternatives
6. Evaluation of Alternatives
7. Conclusions and Recommendations

Chapters 2 through 5 each summarize and present pertinent data and conclusions drawn from the appropriate appendices. Based on the planning projections and the existing situation detailed in Appendices A through I, and on the alternatives formulated and analyzed in Appendices J through M, the most attractive overall alternative solutions are presented and analyzed in Chapters 4 and 5. Each viable overall alternative is described and analyzed on the following basis:

- Feasibility
- Environmental and social concerns
- Reliability
- Flexibility
- Constructibility
- Operability

- Maintenance needs
- Future expansion
- Future upgrading
- Energy consumption
- Capital cost
- Operation and maintenance cost
- Present worth

Chapter 6 compares the viable overall alternatives, both quantitatively and qualitatively. Quantitative comparisons include:

- Capital cost
- Operation and maintenance cost
- Present worth
- Economic analysis
- Sensitivity analysis

Qualitative comparisons include:

- Environmental concerns
- Socio-political concerns
- Technical concerns
- Energy consumption

The alternatives are analyzed and compared as to their sensitivity to changed costs and the effect of changed costs on both present worth and operation costs. Then, the alternatives are compared and ranked by their acceptability using a weighted system that favors either qualitative or quantitative factors. Finally, the three most promising alternatives were analyzed using economic benefits and costs to determine the apparent best project from the viewpoint of society as a whole.

Chapter 7 presents WWCG's recommended changes to the Master Plan.

STUDY AREA

Figure 2-1 is a map showing the area covered by the studies undertaken in the preparation of this report. This area is discussed in detail in Chapter 2. Portions of the Alexandria Governorate not included within the study area are discussed in a companion report entitled "Special Report on Sewer Services for Outside Areas."

REVIEW PROCEDURES

Review of the Master Plan was generally performed at a reconnaissance level and entailed a mainly theoretical office-based review analysis of

existing data, supplemented by field investigations of existing sewers, pump stations, treatment plants, and potential sites for alternative locations for treatment/disposal. Flow measurements and laboratory analyses of wastewaters were not included in the scope of this work; hence, the sewage characterization that was used was based largely on the studies reported in the 1978 Master Plan. As indicated in Appendices A, C, and D, WWCG believes the wastewater flow data presently available understate domestic wastewater flows. WWCG further questions the validity of wastewater characterization data presently available. Therefore, the data are used only for comparison purposes and not for design.

1980 CAPMAS census records were used to formulate new population and population distribution projections. These new population projections have been disaggregated in a manner consistent with that used in the 1978 Master Plan.

The basis for mapping and topographic information for the urbanized Alexandria area was aerial mapping done by Consortium SFS/IGN (France) dated April 1977. These maps, done at a 1:5000 scale and 1-m contour interval, provided the basis for ground elevations within the urban area.

A variety of other mapping was also used, including 1:100,000 scale, 1- to 5-m contour intervals; topographic maps published by the Survey of Egypt, first edition, 1940; and Reconnaissance Soil Maps, scale 1:200,000 and 1:100,000, from the UAR High Dam Soil Science Project, Executive Agency of FAO, 1963. Mapping for land use projections was based on information provided by the Alexandria Planning Department, and satellite imagery of the Alexandria-Nile Delta-Mediterranean Sea was obtained from the U.S. National Aeronautics and Space Administration.

Existing soils information was supplemented by approximately 70 new soils borings taken in the western desert during the search for appropriate land application sites.

A new review of the 1978 Master Plan recommendation for twin 10-km sea outfalls was undertaken. This review included an analysis of the effects of adding primary treatment prior to disposal, but it assumed the same alignment as proposed in the EIS. No new oceanographic data were taken; rather, the review was based on existing information.

Cost estimates used in this report were based on Egyptian construction practices as exemplified by the larger Egyptian contractors. The estimating techniques used assumed the use of local products where possible. Equipment and products not available locally were assumed to be imported from the United States in accordance with USAID grant policies and pro-

cedures. Finally, the cost estimates were based on end-of-1980 prices and were intended to be reconnaissance-level estimates.

PRINCIPAL DOCUMENTS REVIEWED

Numerous documents were used as data sources in the course of the work. Unless otherwise noted, these documents were produced by the engineering firm of Camp Dresser and McKee International, Inc.

- Alexandria Wastewater Master Plan Study
  - Vol. I--Summary Report
  - Vol. II--Technical Report
  - Vol. III--Appendices
  - Vol. IV--Marine Studies
- Special Report No. 1--Oceanographic Studies
- Special Report No. 1--Program for Oceanographic Studies
- Special Report No. 2
- Special Report No. 3
- Special Report No. 4--Top Priority Projects
- Excluded Waste Management Studies
- Supplemental Report--Toxic Industrial Wastes Study
- Environmental Impact Statement
  - Vol. I--Final Statement
  - Vol. II--Final Technical Report
  - Vol. III--Final EIS Comments and Responses
- Summary Report of Evaluation of Alternative Designs for the Ras El-Soda Pump Staion
- Preliminary Design Report--Ras El-Soda Influent Pump Station and Preliminary Treatment Plant
- Preliminary Design Report--Smouha Drainage Improvements
- Design Report--Ras El-Soda Wastewater System
- Preliminary Feasibility Report--Smouha Drainage Improvements
- Preliminary Design Report--Smouha Wastewater Facilities Improvements
- Preliminary Design Report for Mohamed Ali Square Drainage
- Final Design Report--Tannery Wastewater Pretreatment Facilities
- Eastern Harbor Collectors--Design Report
- Collection-Haul of Excluded Wastes
- Recommended Sewer Cleaning Program for 1980
- Summary of Mazout Pilot Study
- Summary of El-Mex Abbatoir

- Final Design Report--Dairy Manure Storage Facilities
- Design Report for (the) Western Harbor Pump Station
- Sewer Use Control Program
- Preliminary Design Report--Alternatives for Management of Mazout Residue
- Preliminary Design Report--Western Harbor Sewerage Improvements
- Design Report--Reconstruction of Madras Collector
- Preliminary Design Report--Septage Disposal Facilities
- Design Report--Cattle Slaughterhouse Waste Disposal Facilities
- Design Reports--East Zone Pump Stations No. 1 through No. 10, No. 12, No. 13
- Sewer Maintenance Operations
- Master Plan for Water Resources Development and Use, Economic Evaluation of Land Reclamation
- Assessment of the Agricultural Inputs Supply System and Policies in Egypt
- U.S. Agricultural Attache' Reports
- USAID Handbook
- Eastern Harbour Bottom Sampling Study--Final Report
- Feasibility Study of West (Nubarya) Extension Reclamation and Settlement Project--Final Report, Vol. I, Annexes I and II (ULG Consultants, Ltd.)
- Classification of Water Bodies, Sewage Treatment Requirement and Sewage Collection Requirements (MISR Laws)
- Memorandum for the Record--Technical Review of the AID Alexandria, Egypt Wastewater Treatment Facilities EIS Documents (USEPA)

Additional data were obtained from numerous field visits, individuals, organizations, selected engineering textbooks, and general sanitary engineering literature.

Complete copies of some of the above documents were unavailable. In some instances, data (particularly tables and figures) were missing.

FORMULATION AND DESCRIPTION OF ALTERNATIVES

A large number of solutions are possible for resolving the overall problem of wastewater collection, treatment, and disposal in Alexandria. The master planning process therefore must include methodology to suc-

cessively screen potential solutions by rejecting the least appropriate until only those alternatives best suited to the situation remain. The remaining alternatives can then be carefully analyzed and compared in order to select the best solution.

For the purpose of this master plan review, the progressive screening process is an integral part of several of the separately bound appendices. Appendix J examines various conventional liquid and sludge treatment processes, selects the most viable and formulates these into treatment and disposal alternatives in which effluent is directly or indirectly discharged into the sea. Appendix K similarly formulates alternatives in which effluent is applied to the land's surface.

After the initial screening process, five viable overall alternatives remained and were analyzed in detail. These were:

- Alternative IA--Sea Disposal with Three Primary Plants
- Alternative IB--Sea Disposal with Two Primary Plants
- Alternative II--Secondary Treatment with Effluent Discharged to Drains
- Alternatives IIIA and IIIB--Land Application

These alternatives are described in more detail in the following sections.

Alternative IA

This alternative is the sea disposal alternative recommended by the 1978 Master Plan with the addition of primary treatment and sludge treatment/disposal. New primary treatment plants would be located at Ras El-Soda (585 Ml/d) and Kait Bey (195 Ml/d). The capacity of the West Treatment Plant, currently under construction, would be expanded to 200 Ml/d. Chlorinated effluent from the West and Kait Bey plants would be discharged into the Mediterranean Sea through an 8-km outfall extending seaward from Kait Bey. A 10-km outfall at Sidi Bishr would similarly serve the Ras El-Soda plant. Sludge would be dewatered and composted for distribution to local farmers. The existing East Treatment Plant would be rehabilitated in this alternative (as is the case with all alternatives) and would provide 45 Ml/d capacity of secondary (activated sludge) treatment. This work would be completed prior to other construction, which would reduce the frequency of raw sewage discharges. As discussed in Appendix J, Alternative IA was found to be a less favorable alternative than Alternative IB and was not evaluated further.

Alternative IB

This alternative is similar to Alternative IA except that only two primary treatment plants would be required. The largest, at Ras El-Soda, would be the same as for Alternative IA. The second, a larger (395 Ml/d) expansion of the West plant, would serve the balance of the sewered area of Alexandria. Each plant would have its own outfall similar to Alternative IA. Sludge treatment and disposal, as well as the rehabilitation of the East plant, would be as previously described. As discussed in Appendix J, this alternative was found superior to Alternative IA; therefore it was fully evaluated in Chapters 4 and 6 of this report.

Alternative II

Alternative II differs from Alternative IB in that secondary treatment by biological towers would be provided in addition to primary treatment. Effluent would be discharged to the Qala and Weqf Drains from two plants sized and located as in Alternative IB. Long sea outfalls would therefore not be required. In other respects Alternative II is similar to Alternative IB. This alternative, as discussed in Appendix J, was found to be reasonably economically attractive. For environmental reasons, questionable reliability, and because the effluent discharge would violate Egyptian law, this alternative was not included in further evaluation.

Alternatives IIIA and IIIB

These alternatives would reuse the treated Alexandria effluent to irrigate up to 76,000 feddans of new farmland reclaimed from the western desert. Raw sewage would be pumped via a long, multiple-barreled force main system to a site 60 km southeast of central Alexandria. The East Treatment Plant would be rehabilitated as with previous alternatives to provide 45 Ml/d of capacity. All other flows (980 Ml/d) would be used for irrigation. The West Treatment Plant (now under construction) would be constructed to its design 85 Ml/d capacity prior to completion of the irrigation system. Together with the East plant, it would prevent some of the raw sewage discharges now occurring. Upon completion of the irrigation system, the West plant would be converted to provide industrial pretreatment.

Two differing possibilities for the treatment and irrigation portions of Alternative III were considered. Alternative IIIA would provide wastewater treatment by anaerobic lagoons followed by rapid infiltration into a ground water storage system. The highly treated water would be

withdrawn by a series of wells as needed for irrigation by center pivot mechanisms. Crops grown would be unrestricted by water quality parameters other than dissolved solids. Alternative IIIB would provide wastewater treatment by anaerobic lagoons only. Lagoon effluent after screening would be used for irrigation by center pivot mechanisms. Crops grown would be generally restricted to feed and forage.

As addressed in Appendix K, Alternatives IIIA and IIIB were found attractive and were therefore fully evaluated in Chapters 4 and 6 of this report.

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- Alexandria University, Faculty of Engineering
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- Alexandria University, High Institute of Public Health
- Alexandria Planning Department
- Alexandria Water General Authority
- Ministry of Irrigation
- Ministry of Agriculture
- Ministry of Housing and Reconstruction
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- Camp Dresser and McKee International, Inc.
- Boyle Engineering Corporation
- Arthur Young and Company, Inc.
- Mediterranean Pollution Control Conference

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This study has been accomplished by a joint venture consisting of CH2M HILL International and Metcalf & Eddy International in association with the Egyptian engineering firms of Dr. A. Abdel Warith Consulting

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- Dr. Ibrahim Elsammak, Planning
- Mr. Jerry Knapp, Economic Analysis

Special consultants for the project included:

- Dr. Helmy M. A. Bakr, Water Resources
- Dr. Abdel-Hakeem M. Gomaa, Soil Sciences
- Dr. M. A. Khalil, Agricultural Economics
- Dr. A. Sabry Abdel-Ghaffar, Soil Microbiology
- Engr. Mostafa Elkadi, Irrigation Engineering
- Dr. R. M. Rizk, Sanitary Engineering
- Dr. H. M. Mitwally, Sanitary Engineering
- Dr. S. D. Wahby, Marine Biology
- Dr. Nabila Fawzy, Marine Biology
- Dr. Amin Samaan, Marine Biology
- Mr. H. V. Anderson, Outfall Construction

The technical review team for the project included:

- Mr. John Chalas
- Mr. C. Pound
- Mr. S. Lasswell
- Mr. G. Suhr
- Mr. J. Filbert
- Mr. M. S. Abdelkawi
- Dr. A. Abdel-Warith
- Mr. M. E. Nauman

TERMS USED IN THIS REPORT

The international metric system of weights and measures is used throughout this report. One Egyptian area measurement, the feddan, which is equal to 0.42 hectare, is also used.

Elevations used are based on mean sea level at Alexandria.

The unit of monetary exchange used is the Egyptian pound (LE). Where necessary, Egyptian pounds are converted to US dollars ($) and vice versa at the rate of exchange of LE 0.70 = $1.00. The exchange rate changed to LE 0.82 = $1.00 after preparation of the initial draft report and is likely to change again before construction of the project is completed. Therefore, the influence of changes in the exchange rate was tested in a sensitivity analysis to determine the potential impact on the study conclusions (see Chapter 6).

Unfortunately, there is no standardized authoritative source for the English spelling of many Arabic proper nouns and place names. Transliteration of Arabic sounds into English varies among different sources. For the purpose of this report, WWCG has adopted a standard spelling that approximates the Arabic sounds in English. We are indebted to Dr. Hassan Mitwally of the University of Alexandria for his assistance in developing the following Arabic-English glossary of proper nouns.

Arabic-English Glossary of Place Names

Abu El-Matimir
Abu Hummus
Abu Qir
Abu Soliman
Alam El-Marqab
Agami
Amria
Anfoshi
Mahmoudiya Canal
Mamoura
Mandara
Manshiya
Marsa Matruh R. R.
Mex
Miami
Minyet El-Basal

Asafra
Bacos
Bahari
Bahig
Bir Masoud
Bisiry
Bulkley
Burg El-Arab
Dekheila
Dir
Fleming
Furn El-Gueraya
Gabbary
Gheit El-Enab
Glym (Glymenopoulo)
Gomrok
Hadra
Hagar El-Nawatiya
Ibrahimia
Kafr El-Dawar
Kait Bey
Karmouz
Khurshid
Kom El-Dikka
Lake Idku
Lake Maryout
Moharrem
Mohsein Pasha
Montazah
Muhit Drain
Mustafa Pasha
Noubaria Canal
Nouzha
Port Est
Qala Drain
Ramleh
Ras El-Soda
Ras El-Tin
Roushdy
San Stefano
Sidi Bishr
Sidi Gaber
Silsila
Siouf Keblia
Smouha
Sporting
Tabia
Umum Drain
Wadi El-Natroun
Wadi El-Qamar
Wardyan
Weqf Drain
Zahriya

Acronyms

A/GOSD	Alexandria General Organization for Sanitary Drainage
APD	Alexandria Planning Department
AWGA	Alexandria Water General Authority
CDM	Camp Dresser & McKee, Inc.
CAPMAS	Control Agency for Public Mobilization and Statistics
CH2M HILL	CH2M HILL International, Inc.
ECG	Engineering Consultants Group

M&E	Metcalf & Eddy International, Inc.
MFDT	Ministry of Finance, Department of Taxation
UNDP	United Nations Development Program
UNESCO	United Nations Educational, Scientific, and Cultural Organization
USAID	United States Agency for International Development
USEPA	United States Environmental Protection Agency
WWCG	Wastewater Consulting Group

Metric Units

cm	centimetre
cm^2	square centimetre
gm	gram
ha	hectare = 10,000 m^2
hr	hour
hp	horsepower
kg	kilogram
km	kilometre = 1000 m
km^2	square kilometre = 100 ha
kt	knot
kW	kilowatt
kWh	kilowatt-hour (power)
mg	milligrams
mm	millimetre
m	metre = 1000 mm
m^2	square metre
m^3	cubic metre = 1000 litres
m^3/sec	cubic metre per second
mg/l	milligram per litre
ml	millilitre
Ml	megalitre = 10^6 litres = 1000 m^3
mmho/cm	micromho per centimetre
ppm	parts per million
tonne	metric ton = 1000 kg
sec	second
ug-at	microgram-atom

Abbreviations

ADWF	average dry weather flow
BOD	biochemical oxygen demand (5-day)
BODR	basis of design report
°C	degrees Celsius
cal	calorie
CIF	import price (dock delivery)
COD	chemical oxygen demand
CU	consumptive use
DO	dissolved oxygen
dw	dry weight
DWF	dry weather flow
EC	electrical conductivity
ep	effective precipitation
FC	fecal coliforms
fd	feddan (.42 hectare)
FOB	export price
gcd	grams per capita per day
HWL	high water level
IDOD	immediate dissolved oxygen demand
lcd	litres per capita per day
LE	Egyptian pound
LR	leaching requirement
Mils	Egyptian Millem = LE 0.001
MPN	most probable number (coliforms)
MSL	mean sea level
No.	number
O&M	operation and maintenance
pH	logarithm of reciprocal of hydrogen ion concentration
p/ha	people per hectare
PDWF	peak dry weather flow
pt	Egyptian piaster = LE 0.01
SS	suspended solids
TDS	total dissolved solids
TPP	top priority project
WTPD	wet tons per day
ZID	zone of initial dilution
%	percent
$	United States dollar = LE 0.70 (at parallel or "tourist" rate)

CHAPTER 2

PLANNING CONSIDERATIONS

INTRODUCTION

To prepare a cost-effective and technically sound wastewater master plan, the existing and projected growth patterns for the project area must be studied and incorporated into the planning process. This chapter first describes the project area and then discusses the population projections and land use considerations used to review the 1978 Master Plan. Based on the existing and projected populations, wastewater flows were estimated for Alexandria through the year 2000. Appendix C presents the detailed development of all the planning projections.

Project Area

The sewerage system study area is shown in Figure 2-1. It includes the heavily urbanized areas of Alexandria, and Mex Dekheila, Amria, and Abu Qir. For study purposes, the project area was subdivided into three heavily urbanized inner areas (the western, central, and eastern zones) and three outer areas (Mex Dekheila, Amria, and Abu Qir). A larger study area was used for the land application investigation.

The urbanized study area for this Master Plan Update is essentially the same as that considered in the 1978 Master Plan. Detailed sewerage system master planning for the outer areas of Mex Dekheila, Amria, and Abu Qir was not included in the scope of work for this review and update. Studies of these outside areas are scheduled in the near future and will be discussed in a separate report.

Climate

Alexandria lies in an arid Mediterranean zone. There are two seasons in Alexandria: summer, which lasts from May through October, and winter from November through April. The summer is characterized by hot dry days and cooler nights. Winter is cooler, with the majority of rainfall occurring during these months. Storms have a duration of 3 to 5 days, contain variable quantities of rainfall, and occur more or less at regular intervals.

The summer temperature in the study area averages 27 degrees C during the day and 20 degrees C at night. In winter, the averages drop to

19 degrees C for daytime and 9 degrees C at night. Frost and freezing conditions seldom occur.

Total annual rainfall averages 192 mm. Annual evaporation is high and exceeds total rainfall by an order of magnitude. Winds are predominantly from the northwest. The mean wind speed is somewhat higher during the winter, but rarely do wind speeds exceed gale force (34 knots).

Public Health

Although improvements have been made in recent years, the public health of the Egyptian people continues to be generally poor. Statistics for the country as a whole have been presented in detail by others, notably the World Health Organization, and will only be discussed briefly here. A more detailed presentation on public health conditions in Alexandria appears in Appendix I, Environmental Data Base Review.

The average life expectancy in Egypt has risen from 42 years in 1947 to 53 years in the early 1970's; mortality rates have also fallen. Infant mortality rates, which are a good indication of the public health of a country, have decreased but are still high. Between 1962 and 1973, approximately one out of every ten infants died.

The waterborne diseases of typhoid, paratyphoid, bacillary dysentery, amoebic dysentery, and infectious hepatitis are all endemic to Egypt. Available statistics indicate the incidence of waterborne diseases in Alexandria. Since the relationship between the incidence of enteric diseases and availability of potable water supplies and wastewater disposal facilities has been well demonstrated, the presence of these diseases may be taken as an indication of inadequate facilities. Poor sanitation or nonhygenic habitats may also be a primary source of disease transmission.

POPULATION

The population of Egypt was approximately 42 million in 1980 and is projected to increase to approximately 66 million by 2000. During the period from 1976 to 1980 the nationwide annual population growth was 2.4 percent. In response to this potential increase in population, the Egyptian government has planned for the development of satellite cities outside existing urbanized areas. Even with such development and an intensive industrial planning effort, urbanized Alexandria will continue to expand over the next 20 years.

Existing Population

The Central Agency for Public Mobilization and Statistics (CAPMAS) conducted the official 1976 census and estimated the 1980 population residing in Alexandria. These statistics have been used in this study.

The Alexandria Governorate is subdivided into 14 census tracts as indicated in Figure 2-2. Table 2-1 presents the 1976 and 1980 population by census tract. This table indicates that the population of Alexandria was 2,319,000 and 2,537,000 for the years 1976 and 1980, respectively, which represents an annual increase of 2.27 percent.

Alexandria is a major sea resort. Therefore, the impact of its seasonal population must be considered when planning wastewater facilities. For this purpose, the 1980 peak seasonal population has been estimated at approximately 400,000, which concurs with the 1978 Master Plan. The seasonal population is mainly located in the Montazah area. The estimated 1980 peak monthly seasonal population and its distribution are presented in Table 2-2.

Projected Population

Population projections for the years 1990 and 2000 by census tract are presented in Table 2-3. The projections indicate that, by the year 2000, Alexandria will have a permanent population of 4,660,000. This projection indicates that the population will increase annually by 3.08 percent between 1980 and 2000, which is somewhat greater than the 2.27 percent annual increase experienced between 1976 and 1980. We believe that a projection of 4,660,000 is moderately conservative. For wastewater planning purposes, conservative (high) population projections should be used to ensure that planned facilities are adequate to meet the future needs of a community. In the event that population growth is deferred, then components of the system can either be phased or delayed to meet actual growth experience.

The population projections presented in Table 2-3 are in substantial agreement with those developed in the 1978 Master Plan. However, the distribution among census tracts has been modified somewhat. Planning officials of the Governorate of Alexandria, as well as district chiefs, indicate that the growth in older districts of the city should be tailed to alleviate already overcrowded conditions. Accordingly, census tracts of Gomrok, Minyet El-Basal, Labban, and Karmouz (Figure 2-3) are shown to have a stabilized population between the years 1990 and 2000. To compensate for this adjustment, a substantial growth is indicated in Montazah. This plan of development is consistent with past growth patterns and local planning concepts.

Projected seasonal populations are also shown on Table 2-3. Montazah, Bab Sharky, Ramleh, and Agami (Dekheila tract) will continue to grow as recreational centers, with the major growth occurring in the Agami area. Seasonal population growth is expected in the Amria area. Present seasonal population estimates indicate that by the year 2000, the Agami-Amria areas will accommodate a monthly peak seasonal population of 230,000. This implies a maximum seasonal population for the Alexandria area of 670,000.

LAND USE

The land use map developed for this investigation is shown on Figure 2-3. The map indicates existing and future residental and industrial developments, and the location of existing and future facilities expected to have a major impact on the development of Alexandria. The map was developed from data secured from the Planning Department of the Governorate, other appropriate agencies, and field investigations.

Urbanization in Alexandria has occurred linearly along the coast, extending east of the older parts of the city (Gomrok). Development has not extended west because of the influence of the port activities in Minyet El-Basal and the lack of infrastructure in the western areas. Development to the south has been delayed by agricultural activities in the area, the Mahmoudiya and Montazah Canals, and Lake Maryout which are natural barriers to development of the southern area. Present planning concepts propose the development of Amria City south of Alexandria along Desert Road. The planned development envisions an urbanized area large enough to house 500,000 persons and a major industrial free zone. This planned development with proper access to port facilities may well accelerate the development of the portion of the southern area that is south of Lake Maryout, part of which is outside the Governorate.

Population densities range from a high of 879 people per hectare (p/ha) in a portion of the central zone, to 487 and 31 p/ha in the Ramleh and Montazah census tracts, respectively (see Appendix C). Typically, population densities in the southern and western areas are substantially lower, ranging from 1 to 17, respectively.

Residential

Major residential developments are planned in Amria and Montazah (Sadat City), which are located on the southern and eastern areas of the Governorate. These areas presently have limited water and wastewater service. If their development potential is to be realized, then adequate water and wastewater facilities must be provided in the very near future.

Areas that are proposed for immediate development or are now under development are located in Ras El-Soda, Sidi Gaber, and Ramleh.

The land use map shows more than sufficient land within the Governorate to accommodate a population of 4,660,000, the projected population for the year 2000. However, to house this population and to meet the goals for accommodating seasonal population, growth will require the active development of western and southern areas.

To accommodate a permanent population of 4,660,000, and to maintain reasonable levels of density in developed areas, additional residential areas will be required. To provide this need, we anticipate that the areas of Siouf Manshiya, El Bahameyeh, and Mahmoudiya Keblia will be developed to house a population of 162,000 by the year 2000. These residential areas will accommodate, along with Amria and Sadat City, more than 50 percent of the population growth expected between 1980 and 2000.

As indicated in Figure 2-3, the residential areas that are to be developed beyond the year 2000 lie east of the planned City of Amria and west of Dekheila, south of Matrouh Road. This pattern of development is consistent with present planning concepts and past growth trends, and recognizes the speed with which water and wastewater facilitites may be developed or extended to these areas.

Seasonal

As indicated in Figure 2-3, the growth in seasonal population will impact the Agami and Amria areas. Agami has, over the years, experienced unplanned resort development. Our investigations indicate that average population densities are approaching 250 to 300 people per hectare, with appreciably higher densities along the shoreline. These densities preclude the use of septic tank systems for wastewater disposal (the present practice), and accordingly indicated the need for wastewater collection and treatment facilities. Due to irregular housing and street patterns, providing wastewater facilities in this area will be difficult and expensive.

So that similar problems do not develop for the proposed Amria resort area, we recommend that planning goals and programs be developed quickly and implemented for this area. This will permit providing infrastructure facilities at a more reasonable cost.

Industrial

The city of Alexandria is one of the major industrial centers of Egypt, employing approximately 14 percent of the country's manufacturing labor force. Existing industrial areas are located in Abu Qir, Siouf Keblia, Smouha, Nouzha, and Dekheila.

As indicated on the land use map, areas that are expected to undergo substantial industrial development are located in Amria, east of Amria, along the northern edge of Lake Maryout, in Dekheila and Karmouz, and in Abu Qir along Abu Qir Bay. Other significant industrial developments consist of a major petroleum transfer station located in the western part of the Governorate and a proposed nuclear energy project located west of the petroleum transfer station.

Warehouses supporting the activities in the western port are located in Minyet El-Basal and Dekheila. Grain storage facilities are being expanded in the Dekheila area. With the development of the proposed Dekheila port facilities, warehousing activities should increase in these areas limiting their potential for residential development.

The forecasted rate of industrial development presented in the 1978 Master Plan has not been realized. One of the contributing factors may have been the lack of good wastewater service. Accordingly, in planning for wastewater service in industrial areas, we have assumed that the estimated growth rate between 1980 and 1990 will be accelerated so that 1990 planning goals will be achieved.

WATER USE

In the 1978 Master Plan, water use by category (e.g., domestic-governmental use, etc.,) was established and used to estimate existing and projected wastewater flows. In the absence of wastewater flow records, this is an acceptable procedure. We have reviewed the water use projections presented in the 1978 Master Plan and find them acceptable for the purpose intended (i.e., projecting wastewater flows). The following paragraphs briefly discuss our analysis, which is presented in more detail in Appendix C.

Existing Water Use

Estimated 1976 water consumption by category is presented in Table 2-4. Studies (Alexandria Waterworks Master Plan) of water production records, water sales, and an intensive analysis of eight selected areas were used

to determine unit domestic and governmental consumption rates, industrial consumption, other uses, and the quantity of water lost through leakage in the system. These studies indicated that, within the study area, the unit domestic consumption, and unit governmental consumption was 116.5 and 30.0 litres per capita per day (lcd), respectively. The water consumption data presented in Table 2-4 was developed based on this analysis. Losses were taken as 15 percent of the total estimated water consumption in the study area.

In the absence of metered data, the methodology used in developing existing water uses, which included some actual measurement of water production, is the only reasonable recourse available to evaluate particular water uses. However, due to the absence of specific water sales data in certain categories and the practice of providing free water to certain categories of users, the analysis must be somewhat subjective in nature. The Alexandria Water General Authority is undertaking a program to improve its metering programs. A/GOSD should follow the results of this program closely so that, in the absence of recorded wastewater flows, the data can be used to more accurately predict future flows.

Projected Water Use

Table 2-5 indicates unit consumption projections used in the 1978 Master Plan to develop water consumption projections from 1976 to the year 2000. Unit domestic water consumption rates are expected to increase from 116.5 to 159.4 lcd over the planning period, a 1.32-percent increase. This rate of increase is appreciably higher than that now being experienced in the United States and appropriately reflects a society experiencing rapid development.

Industrial per capita consumption ranges from 44.3 to 70.2 lcd, which represents an annual increase of 1.9 percent over the study period. Other categories of unit water consumption are estimated to remain relatively constant with system losses ranging from 15 to 12 percent of the total demand.

Since the unit water consumption projections appear conservative and are in reasonable agreement with studies the consultant has undertaken elsewhere and in Egypt, the unit water consumption data as set forth in the 1978 Master Plan was used in this investigation.

WASTEWATER FLOWS AND CHARACTERISTICS

In developing wastewater master plans, the identification of existing wastewater flows and characteristics is an important element in the pro-

cess. This is because future wastewater flows and characteristics are more accurately projected when existing wastewater flows and characteristics have been properly identified. Our findings indicate a need for a wastewater flow, sampling, and analysis program to better define existing wastewater flows and characteristics. The need for such programs is fully discussed in Appendix C.

Our findings also indicate that existing and future wastewater flows as developed in the 1978 Master Plan may be too low. However, for comparison of disposal alternatives they were used with some minor adjustments in this investigation. When the basis of design reports are prepared, we recommend that flow design parameters be adjusted to ensure that the proposed facilities will meet the future needs of their service areas.

Existing Wastewater Flows

In the 1978 Master Plan, analysis determined the percentage of water consumption that could be expected to be returned to the wastewater system. This analysis apparently indicated that a wastewater return ratio of 83 percent of the domestic and commercial water consumption would adequately account for all wastewater flows derived from all community activites, excluding industrial flows. Using an 83-percent return ratio, a wastewater flow of 100 lcd was deemed appropriate for determining the 1977 wastewater flow generated through domestic, commercial, and governmental activities in Alexandria. Based on the 1976 unit water consumptions as set forth in Table 2-6, this wastewater flow represents only a 68-percent return when domestic, commercial, and governmental water demands are considered. We believe this return rate to be low in the light of conditions in Alexandria and our experience elsewhere in Egypt.

Table 2-7 presents the estimated 1977 wastewater flows. Domestic wastewater flows include wastes generated from all community activities, other than industrial, and were determined by multiplying the population served by a unit wastewater flow of 100 lcd.

Industrial discharge estimates were based on a 1978 Master Plan survey that covered 145 industries and detailed data secured from 129 of these industries. This indicates that 129 industries account for 90 percent of the industrial waste discharge. The information on the industrial wastewater flows were reviewed as part of this Master Plan (Appendix D). This review found that conditions have not changed significantly since the earlier study, particularly in the inner areas.

Projected Wastewater Flows

Based on analysis of population statistics, water consumption, and wastewater flow data, the following assumptions were used in estimating future wastewater flows:

- Total permanent and peak monthly seasonal population projections were used in projecting future wastewater flows. The estimated flows accordingly represent peak monthly wastewater flows for the year under consideration.
- The population distribution developed for this review was used in determining wastewater flows from specific areas.
- The area within each district was determined by actual measurement using recent aerial mapping.
- Unit domestic wastewater flows were assumed to include domestic, commercial, and governmental activities. Unit domestic wastewater flows as presented in the 1978 Master Plan and subsequent reports were used throughout.
- In accordance with the industrial update as presented in Appendix D, present and future industrial wastewater flows as set forth in the 1978 Master Plan were used.
- Where applicable, a special allowance to include military installations, hospitals, and recreational areas was used in developing future wastewater flows.
- Infiltration rates were determined using the infiltration curves presented in Appendix A of this report.

Wastewater flows for the years 1990 and 2000 were developed and are shown in Tables 2-8 and 2-9. Tables 2-8 and 2-9 indicate that the total average daily flow during the maximum month can be expected to reach 1115 and 1476 Ml/day by the years 1990 and 2000, respectively. Industry is expected to contribute more than 50 percent of the total flow in the year 2000. This is in accordance with local planning concepts to maintain Alexandria's role as an industrial center of Egypt.

As part of this investigation, future wastewater flows indicating average annual, peak monthly, maximum day, and maximum hourly rates of flows were developed for the central and western zones, Nouzha, and the area that is tributary to the proposed Ras El-Soda Treatment Plant. These projected flows are shown in Appendix J.

Existing Wastewater Characteristics

Wastewater characteristics determine the type and degree of treatment required and the impact the wastes will have on collection systems and the ecology of the disposal environment. Wastewater characteristics are normally defined by noting the oxygen demand (BOD and COD), solids and nitrogen concentrations, and various constituents (mainly heavy metals) that may interfere with the treatment process. The characteristics of wastewater also determine the impact sludge will have on the environment if applied to land.

The 1978 Master Plan estimates that the domestic BOD contribution in 1977 was approximately 50 grams per capita per day. Since the 1977 population was estimated at 2.8 million, this implies a domestic BOD loading of 140,000 kilograms per day (kg/day).

Industrial wastewater loads were evaluated through the industrial survey undertaken in the 1978 Master Plan. Estimated industrial BOD and COD loadings were 145,000 and 280,000 kg/day, respectively. Miscellaneous constituents that might be harmful or have an adverse impact on existing and proposed wastewater treatment processes were also identified. The findings of this survey are presented in Table 2-10.

Under present planning concepts, all industrial wastewater, except cooling waters, will be discharged to the wastewater system. The loadings indicated in Table 2-10 are of sufficient magnitude to warrant investigating their impact on proposed treatment processes. Preliminary investigations indicate that several constituents may be excessive, and industrial pretreatment will be required to reduce their concentrations before discharging them to the wastewater system.

Projected Wastewater Characteristics

Domestic BOD contributions were estimated in the 1978 Master Plan to increase from 50 to 74.6 grams per capita. Industrial BOD concentrations are estimated to increase from 640 milligrams per litre (mg/l) in 1977 to 700 mg/l by 2000, which is a nominal increase. Using these criteria, BOD loadings for the years 1990 and 2000 derived from domestic and industrial sources have been estimated. These estimates are presented in Tables 2-11 and 2-12, respectively.

Table 2-12 indicates the total BOD loading on the wastewater system will be over 900,000 kg/day, of which 56 percent will be contributed by industries. Domestic BOD, based on a unit domestic contribution of 74.6

grams per capita will account for the remaining 44 percent of the total loading. A unit domestic contribution of 74.6 grams per capita per day implies an appreciable increase in the standard of living in Alexandria. However, it does not allow for a widespread use of electrical dishwashers, clothes washing machines, or garbage grinders. In projecting future industrial BOD loadings, the present mix and diversity of industry has been assumed to hold throughout the planning period. This implies that future industries that may contribute larger BOD loadings should be considered on a case-by-case basis.

Table 2-1
1976 AND 1980 POPULATIONS BY CENSUS TRACT

Census Tract	1976	1980
Gomrok	142,800	156,464
Labban	78,496	86,003
Karmouz	213,737	234,179
Mina of Alex.	501	547
Ramleh	446,155	488,828
Montazah	310,054	336,706
Bab Sharky	215,639	236,265
Attarine	75,062	82,238
Manshiya	44,485	49,129
Moharem Bey	336,228	368,386
Sidi Gaber	134,516	147,382
Dekhella (includes Agami)	45,868	50,255
Amria	47,025	51,522
Minyet El-Basal	227,725	249,505
Alex. Total	2,318,291	2,537,409

Source: Central Agency for Public Mobilization and Statistics.

Table 2-2
ESTIMATED 1980 PEAK MONTHLY SEASONAL POPULATION

Census Tract	Population
Montazah	200,000
Bab Sharkey	85,000
Ramleh	40,000
Sidi Gaber	50,000
Dekhella (includes Agami)	25,000
Total	400,000

Source: 1978 Master Plan

Table 2-3
POPULATION PROJECTIONS FOR YEARS
1990 AND 2000

Census Tract	Permanent Population 1990	Permanent Population 2000	Seasonal Population 1990	Seasonal Population 2000	Total Population 1990	Total Population 2000
Gomrok	160,000	160,000	-	-	160,000	160,000
Labban	90,000	90,000	-	-	90,000	90,000
Karmouz	250,000	250,000	-	-	250,000	250,000
Mina of Alex.	1,000	1,000	-	-	1,000	1,000
Ramleh	600,000	700,000	50,000	54,000	650,000	754,000
Montazah	818,800	1,400,000	205,000	214,000	1,023,800	1,614,000
Sidi Gaber	304,000	360,000	65,000	66,000	369,000	426,000
Attarine	85,000	85,000	-	-	85,000	85,000
Manshiya	52,000	52,000	-	-	52,000	52,000
Moharem Bey	406,900	449,300	-	-	406,900	449,300
Bab Sharky	261,000	288,100	100,000	106,000	361,000	394,100
Dekhella (Includes Agami)	58,300	64,600	80,000	160,000	138,300	224,600
Amria	193,000	500,000	40,000	70,000	233,000	570,000
Minyet El-Basal	260,000	260,000	-	-	260,000	260,000
Alex. Total	3,540,000	4,660,000	540,000	670,000	4,080,000	5,330,000

Table 2-4
1976 ESTIMATED WATER CONSUMPTION BY CATEGORY

Category	Average Water Consumption[a] (1000 m^3/day)	Percent
Domestic	281.3	50
Industrial	106.9	19
Government	72.3	13
Other	19.1	3
Losses	81.4	15
	561.0	100

Source: Alexandria Waterworks Master Plan

[a]Does not include the 39,000 m^3/day that is consumed in the Marsa Matrouh and western areas, the western desert, and southern Beheiria.

Table 2-5
UNIT WATER CONSUMPTION PROJECTIONS
TO 2000 BY CATEGORY
(lcd)

Category	1976	1982	1990	2000
Domestic	116.5	129.2	148.3	159.4
Industrial	44.3	64.1	70.2	70.2
Government	30.0	31.0	32.0	32.0
Other	7.9	8.0	9.0	9.1
Losses	33.7	40.1	39.1	37.3
Total	232.4	272.4	298.6	308.0

Source: 1978 Master Plan

Table 2-6
1976 WATER CONSUMPTION BY CATEGORY

Population Served (1000's)	Category	Daily Consumption (1000 m^3/day)	Unit Consumption (lcd)
2414	Domestic-commercial	281	116
	Governmental	72	30
	Subtotal	353	146
	Other uses[a]	19	8
	Total	372	154

Source: Alexandria Waterworks Master Plan

[a]Includes nongovernment institutions, water sales stations, fire-fighting systems, irrigation, and small special rate users.

Table 2-7
ESTIMATED 1977 WASTEWATER FLOWS

Population	
Total Alexandria (permanent plus seasonal peak)	2,800,000[a]
Within presently sewered areas (percent sewered = 85%)	2,380,000
Domestic sewage (lcd)	100
Wastewater flows (Ml/day)	
Domestic:	
Sewered areas	238
Unsewered areas	42
Total	280
Contaminated industrial:	
To sewers	89
To drains, canal, and the sea	137
Total	226
Infiltration (4300 ha x 0.15 l/sec/ha)	56
Average dry weather flow:	
In sewered areas	383
In unsewered areas	179
Total	562

Source: 1978 Master Plan

[a]1976 permanent population (2.382 million) times annual growth rate (2.7%) plus a a seasonal population peak of 358,000

Table 2-8
ESTIMATED 1990 WASTEWATER FLOWS

District	Area (ha)	Population[a] (1000's)	Unit Domestic Rate (lcd)	Domestic Average Flows[b] (Ml/day)	Industrial Average Flow (Ml/day)	Special Allowance[c] (Ml/day)	Infiltration (Ml/day)	Total Average Flow (Ml/day)
Inner Areas								
Western zone	1,068	576	105	60	92	-	13	165
Central zone	1,322	933	125	117	28	4	21	170
Eastern zone	4,306	2,061	118	243	130	4	33	410
Nouzha[d]	266	25	83	2	25	-	2	29
Outer Areas								
Abu Qir	1,100	75	112	8	132	2	18	160
Amria	1,535	146	132	19	43	-	10	72
Mex-Dekhella	1,027	104	87	9	102	-	8	119
Unsewered		160	--	--	--	--	--	--
Totals	10,624	4,080		458	552	10	105	1,125

[a]Includes seasonal population.

[b]Includes domestic, governmental, and commercial flows.

[c]Includes allowances for military establishments, hospitals, and recreational centers.

[d]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.

Table 2-9

ESTIMATED 2000 WASTEWATER FLOWS

District	Area (ha)	Population[a] (1000's)	Unit Domestic Rate (lcd)	Domestic Average Flows[b] (Ml/day)	Industrial Average Flow (Ml/day)	Special Allowance[c] (Ml/day)	Infiltration (Ml/day)	Total Average Flow (Ml/day)
Inner Areas								
Western zone	1,265	602	125	75	110	-	15	200
Central zone	1,322	1,016	136	137	34	4	20	195
Eastern zone	4,660	2,741	122	334	189	4	35	562
Nouzha[d]	521	30	100	3	36	-	3	42
Outer Areas								
Abu Qir	1,480	106	108	12	150	2	10	174
Amria	2,900	430	136	58	62	-	16	136
Mex-Dekhella	1,350	135	103	14	144	-	9	167
Unsewered		270	-	-	-	-	-	-
Totals	13,498	5,330		633	725	10	108	1,476

[a]Includes seasonal population.

[b]Includes domestic, governmental and commercial flows.

[c]Includes allowances for military establishments, hospitals, and recreational centers.

[d]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.

Table 2-10
MISCELLANEOUS CONSTITUENT LOADINGS

Constituent	Loadings (kg/day)
Iron	289
Chromium	85
Copper	15
Manganese	26
Zinc	45
Fluoride	0.3
Sulfide	4
Lead	9
Nickel	14
Cobalt	0.8

Source: 1978 Master Plan

Table 2-11
ESTIMATED 1990 BIOCHEMICAL OXYGEN DEMAND LOADS
(kg/day)

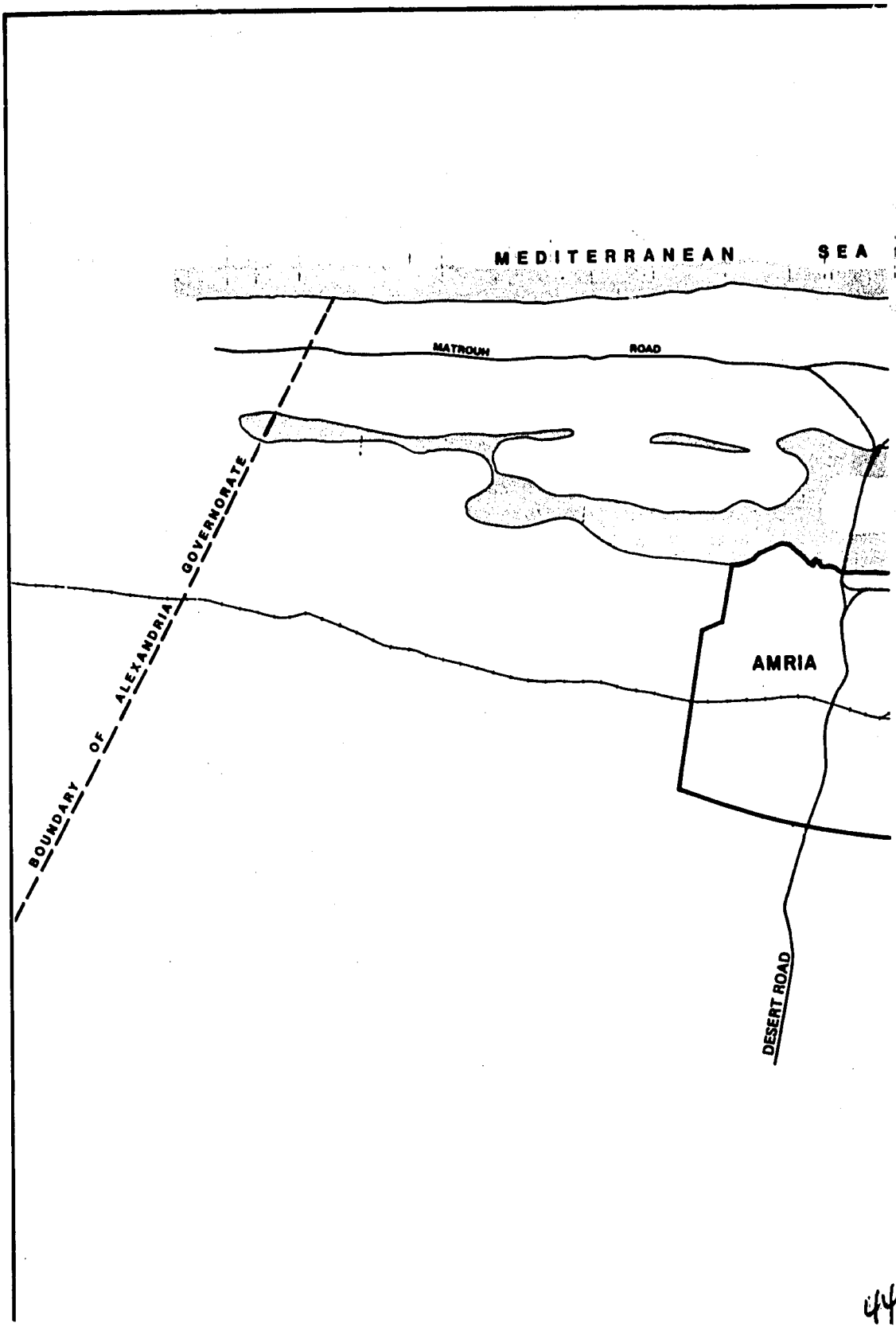
District	Domestic	Industrial	Total
Inner Areas			
Western zone	42,000	62,000	104,000
Central zone	70,000	19,000	89,000
Eastern zone	152,000	88,000	240,000
Nouzha[a]	2,000	17,000	19,000
Subtotal	266,000	186,000	452,000
Outer Areas			
Abu Qir	8,000	89,000	97,000
Amria	11,000	29,000	40,000
Mex-Dekhella	8,000	69,000	77,000
Subtotal	27,000	187,000	214,000
Unsewered	6,000	---0	6,000
Total	299,000	373,000	672,000

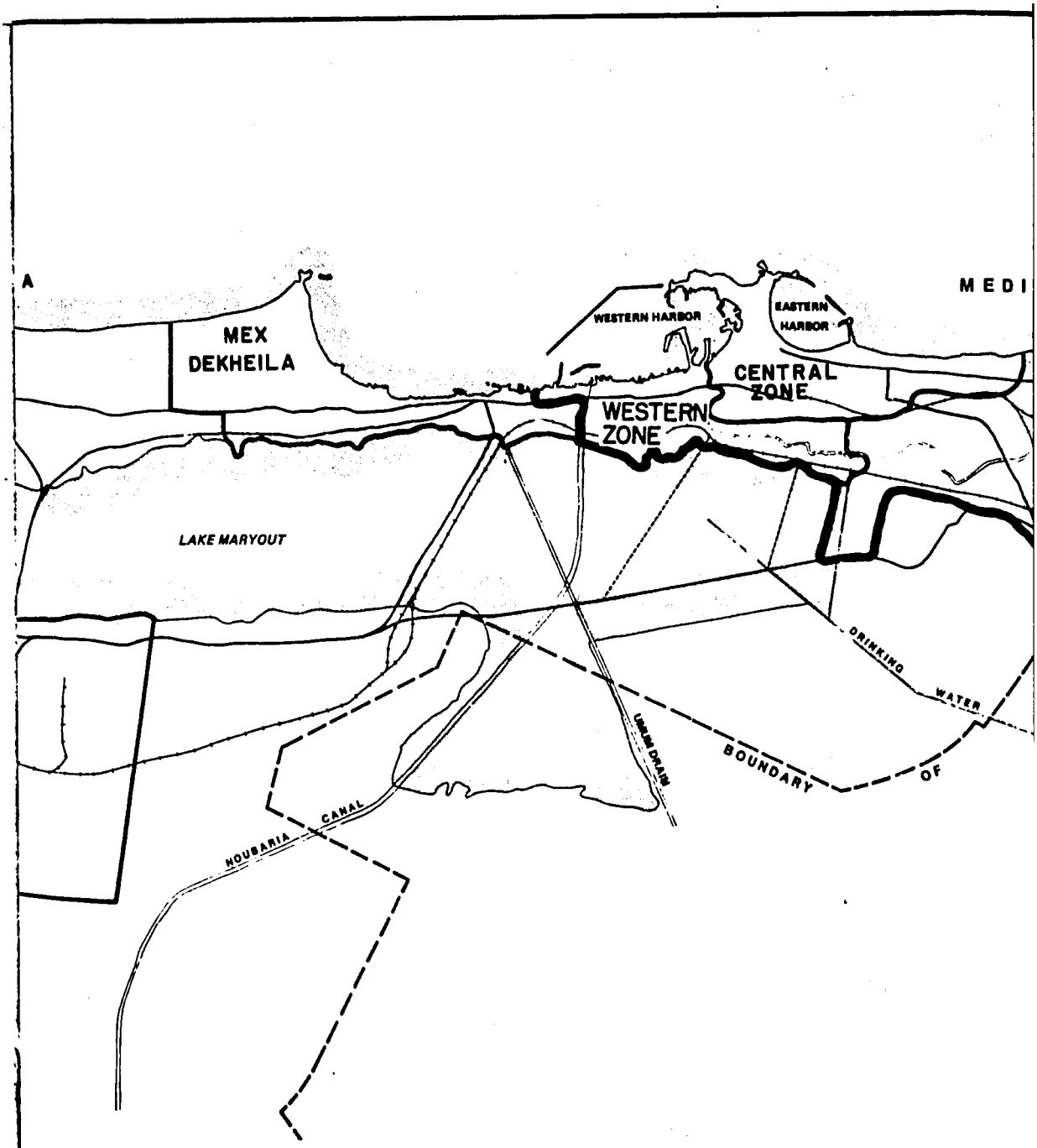
[a]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.

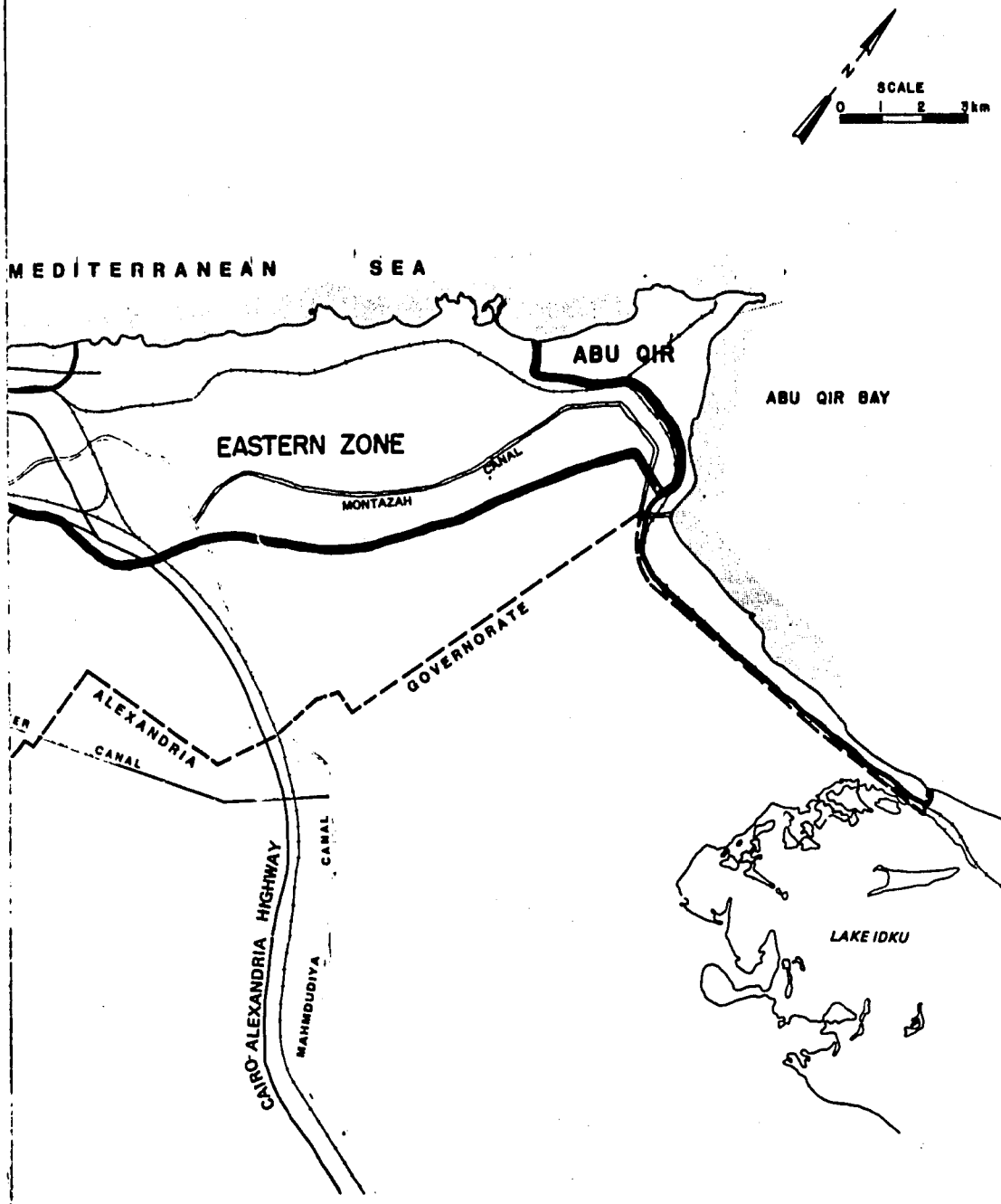
Table 2-12
ESTIMATED 2000 BIOCHEMICAL OXYGEN DEMAND LOADS
(kg/day)

District	Domestic	Industrial	Total
Inner Areas			
Western zone	45,000	77,000	122,000
Central zone	78,000	24,000	102,000
Eastern zone	207,000	132,000	339,000
Nouzha[a]	2,000	25,000	27,000
Subtotal	332,000	258,000	590,000
Outer Areas			
Abu Qir	11,000	105,000	116,000
Amria	32,000	43,000	75,000
Mex-Dekhella	10,000	101,000	111,000
Subtotal	[illegible],000	249,000	302,000
Unsewered	10,000	-----	10,000
Total	395,000	507,000	902,000

[a]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.


MEDITERRANEAN SEA
MATROUH ROAD
BOUNDARY OF ALEXANDRIA GOVERNORATE
AMRIA
DESERT ROAD


A
MEDI
MEX
DEKHEILA
WESTERN HARBOR
EASTERN
HARBOR
CENTRAL
ZONE
WESTERN
ZONE
LAKE MARYOUT
DRINKING
WATER
BOUNDARY
OF
UMUM DRAIN
NOUBARIA
CANAL



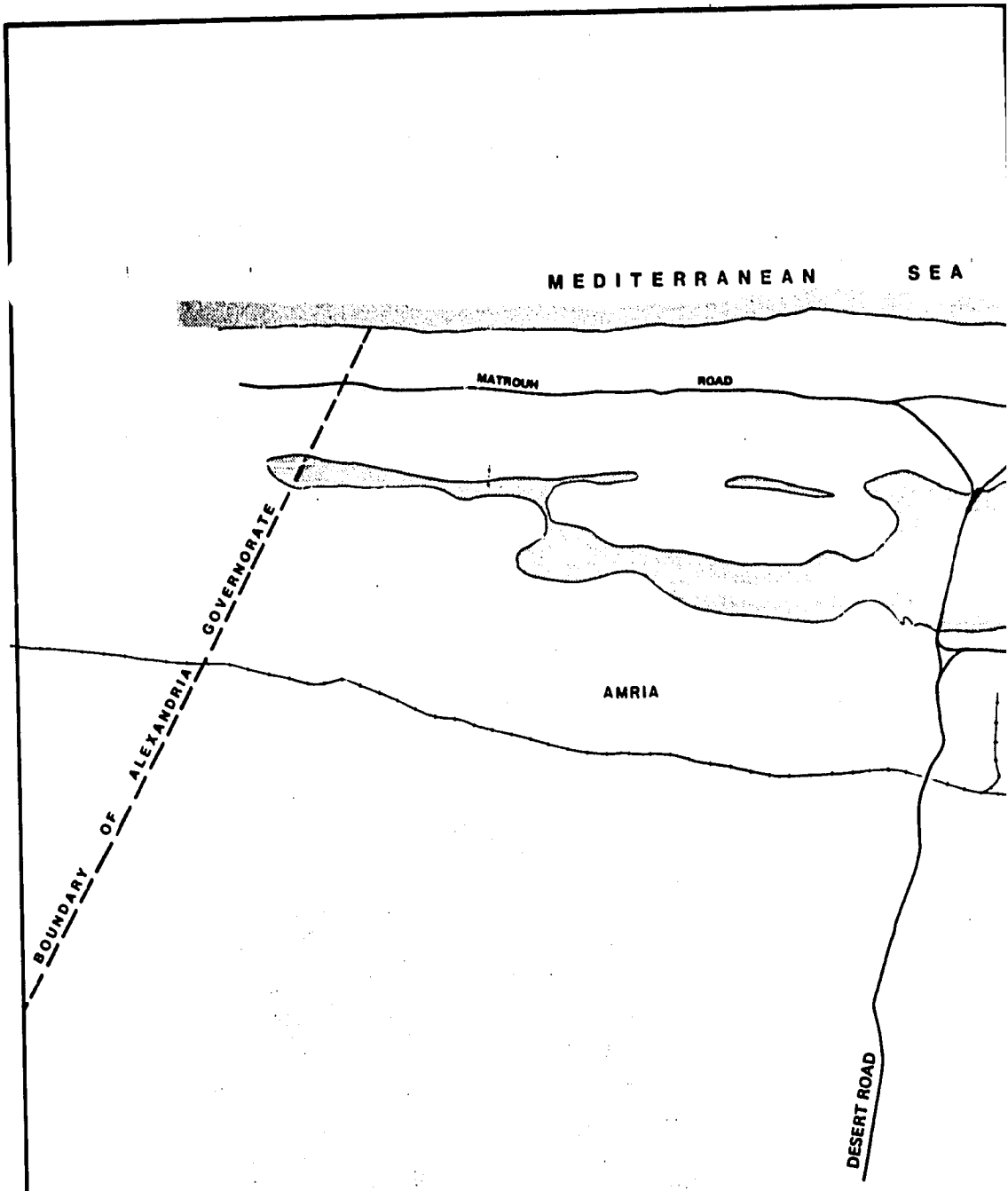
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

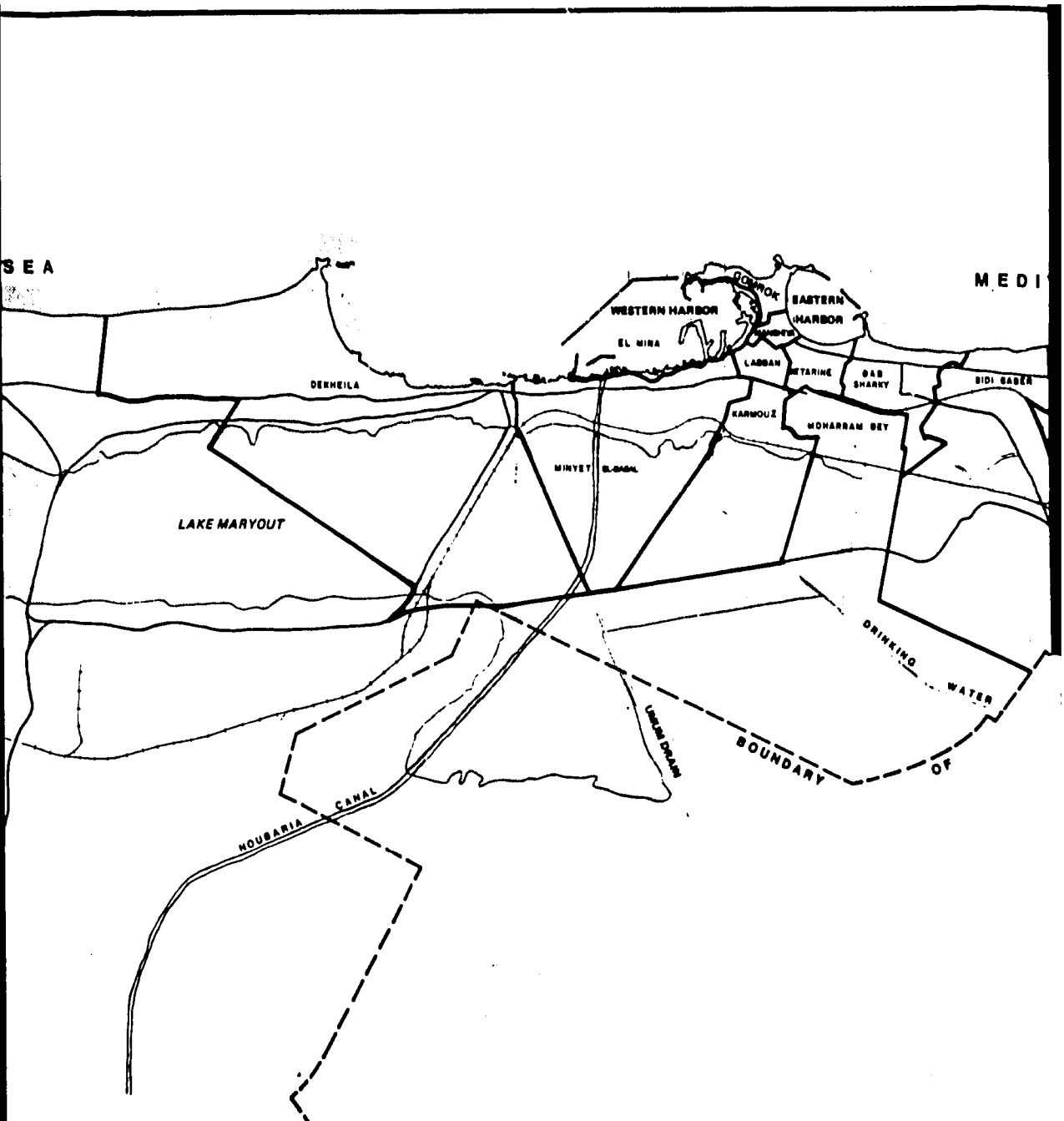
FIGURE 2-1

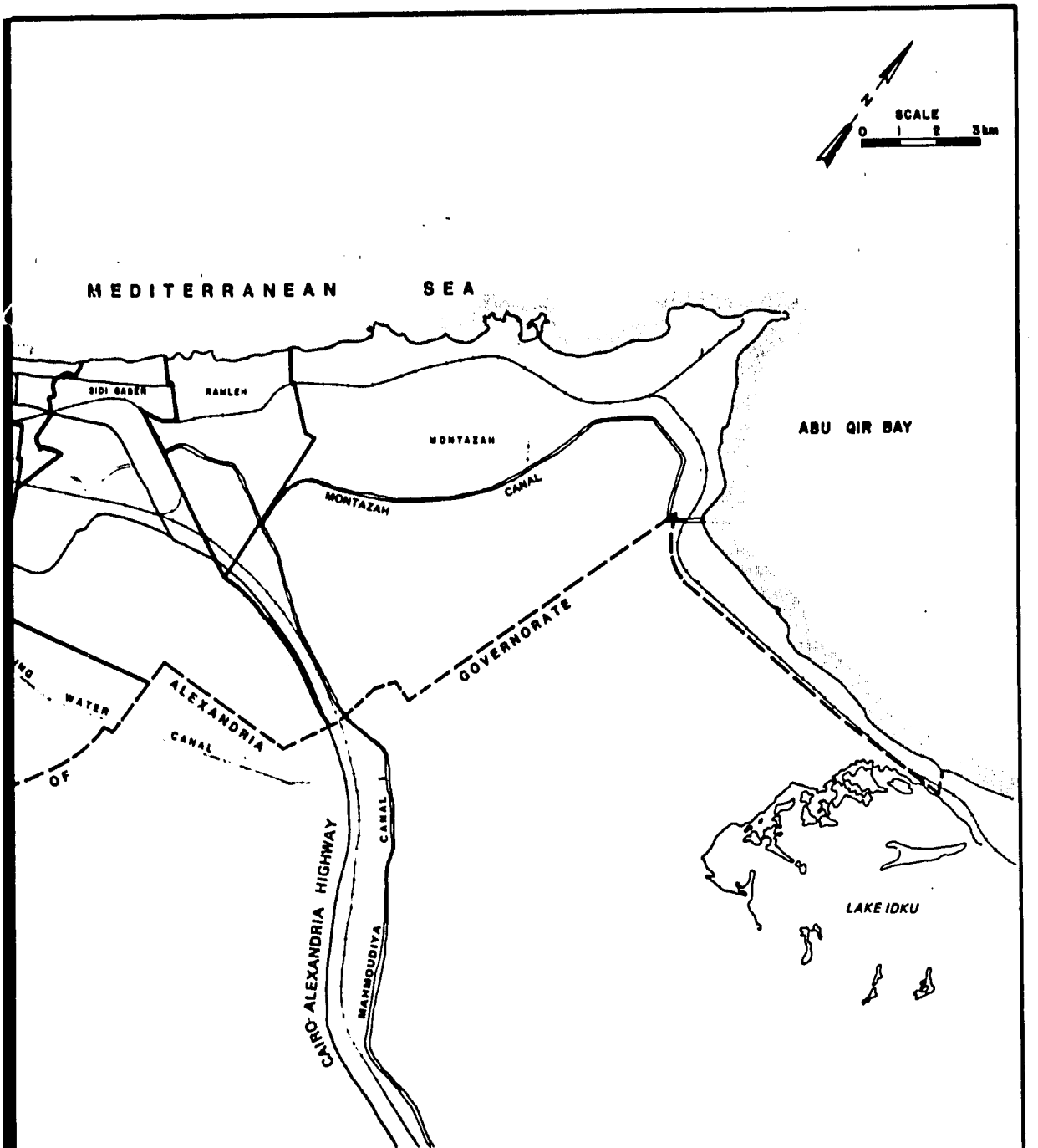
SEWERAGE SYSTEM STUDY AREA

A/GOSD WWCG W/ECG

4b


MEDITERRANEAN SEA
MATROUH ROAD
BOUNDARY OF ALEXANDRIA GOVERNORATE
AMRIA
DESERT ROAD

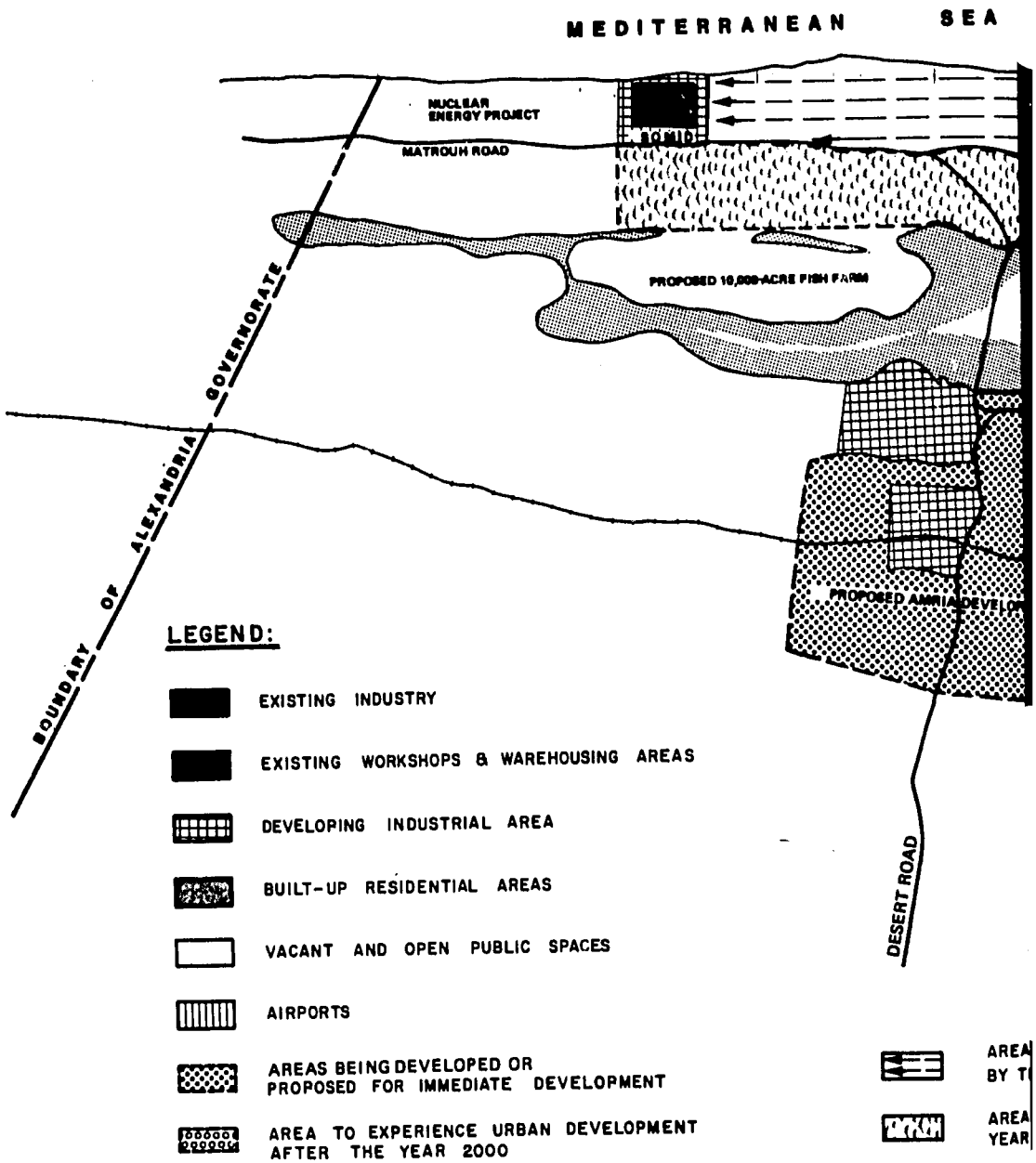

SEA
MEDI
WESTERN HARBOR
EASTERN HARBOR
EL MINA
DEKHEILA
LABBAN
ETARINE
BAB SHARKY
BIDI BADER
KARMOUZ
MOHARRAM BEY
MINYET EL BASAL
LAKE MARYOUT
DRINKING WATER
BOUNDARY OF
NOUBARIA CANAL

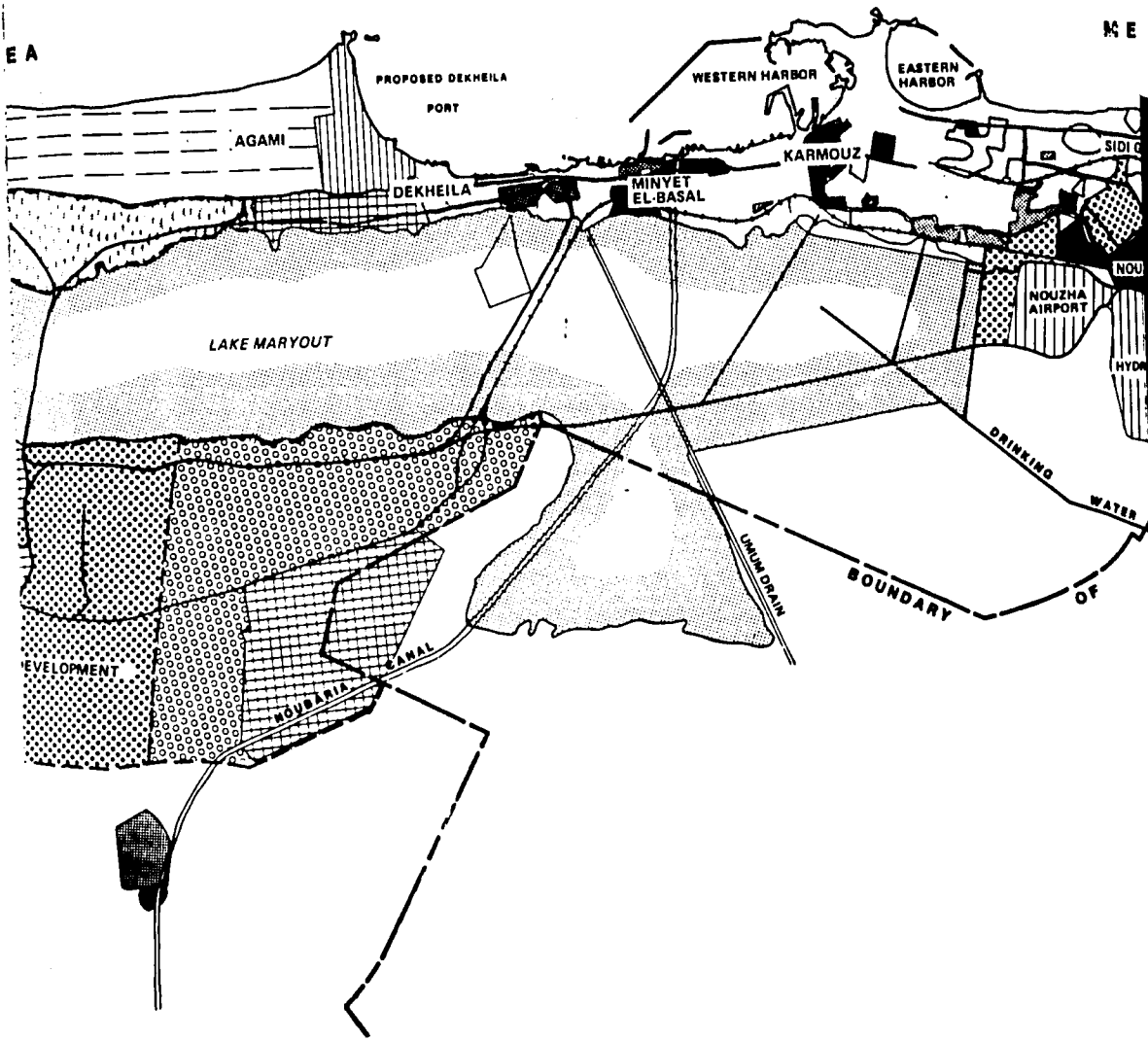


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE 2-2

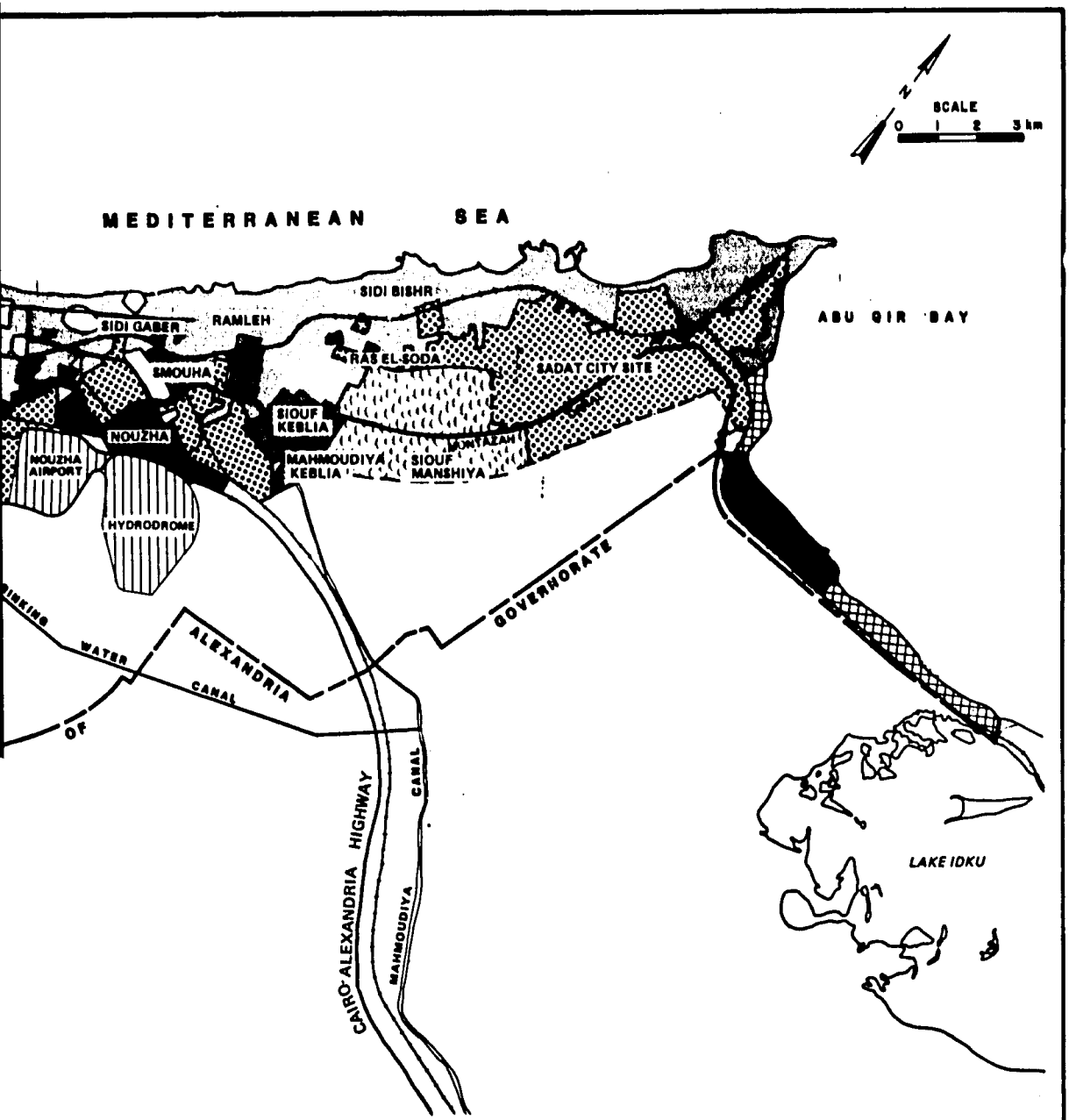
CENSUS TRACTS


MEDITERRANEAN SEA
NUCLEAR ENERGY PROJECT
MATROUH ROAD
PROPOSED 10,000-ACRE FISH FARM
BOUNDARY OF ALEXANDRIA GOVERNORATE
DESERT ROAD
LEGEND:
EXISTING INDUSTRY
EXISTING WORKSHOPS & WAREHOUSING AREAS
DEVELOPING INDUSTRIAL AREA
BUILT-UP RESIDENTIAL AREAS
VACANT AND OPEN PUBLIC SPACES
AIRPORTS
AREAS BEING DEVELOPED OR PROPOSED FOR IMMEDIATE DEVELOPMENT
AREA TO EXPERIENCE URBAN DEVELOPMENT AFTER THE YEAR 2000



AREA TO EXPERIENCE SEASONAL URBAN DEVELOPMENT BY THE YEAR 2000

AREA TO EXPERIENCE URBAN DEVELOPMENT BY THE YEAR 2000



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE 2-3

LAND USE

A/GOSD WWCG W/ECG

CHAPTER 3

EXISTING SITUATION

The existing sewerage system in Alexandria is comprised of gravity collectors, pump stations, force mains, and treatment plants, all of various ages and in differing operating conditions. These system components were reviewed as part of the 1978 Master Plan and various improvements, both immediate Top Priority Projects (TPP's) and long-term projects, were recommended. As part of this Master Plan Review these components were re-evaluated as were the recommendations of the 1978 Master Plan. Another objective of the review was to develop and recommend short-term improvements to the system to be constructed prior to implementation of a long-term wastewater management plan. Because of the nature of many of the improvements, they will actually be long-term in nature. An example is the project to improve the East Treatment Plant where modifications are recommended to enhance the plant operability regardless of which wastewater management plan is selected.

This chapter describes the existing system components, their condition, and the recommended short-term improvements. Each of the system components is described and analyzed in detail in the appendices to this report. Therefore, the reader should refer to the following appendices for a detailed discussion of each system component:

Appendix E--East Treatment Plant Evaluation
Appendix F--West Treatment Plant Evaluation
Appendix G--Pump Station Review
Appendix H--Collection System Review.

SYSTEM DESCRIPTION

The existing sewerage system is divided into 3 zones--central, western, and eastern--and includes approximately 150 km of main interceptors, 1500 km of secondary collectors and street sewers, 30 km of force mains, 32 pump stations, and two treatment plants. Figure 3-1 illustrates the main features of the existing system including major conveyance pipelines, pump stations, treatment plants, and wastewater discharge and overflow points.

For the most part, wastewater is now discharged untreated into area waterbodies including the Mediterranean Sea and Lake Maryout. The exception has been brief periods when the East Treatment Plant has operated. The major discharge points are the Kait Bey outfall on the Mediterranean Sea, the Tabia pump station discharge to Abu Qir Bay, and

local drains into the Western Harbor. Additionally, numerous local overflows are located along the Mediterranean shore. Finally, wastewater discharged into Lake Maryout is eventually released into the Western Harbor by the Mex pump station.

Central Zone

Wastewater from the western part of the central zone is collected by two separate systems and discharged to the Mediterranean Sea through the Kait Bey outfall or directly into the Western Harbor. In the eastern portion, three separate systems convey wastewater to the Kait Bey outfall.

Western Zone

In the western zone, wastewater from the northern tributary areas discharged to the Western Harbor, and that from the southern areas discharged to Lake Maryout. When the construction of the West Treatment Plant is completed, present plans call for all the western zone wastewater to be treated and discharged to Lake Maryout.

Eastern Zone

Wastewater from the western portion of the eastern zone goes to the Smouha Drain and eventually discharges to Lake Maryout. In the eastern portion of this zone, wastewater is collected by numerous individual systems that collect both stormwater and wastewater in parallel conduits. At the individual pump stations, stormwater is pumped to the sea while wastewater is eventually pumped to the East Treatment Plant. When the system is surcharged, as is normally the present situation, combined flows are discharged to both the sea and the East Treatment Plant. The East plant is presently not operating, so untreated wastewater is discharged, eventually to Lake Maryout, from this source.

SEWERAGE SYSTEM

The following discussion of the existing sewerage system is divided into reviews of the gravity, pump station/force main, and treatment systems. In many cases, discussing one part of the sewerage system separately from the remainder is difficult. Therefore, as appropriate for clarity, interface of the system components is described. The reader is referred to Appendix H of this report for an evaluation of the existing system as well as Special Report No. 2 of the Alexandria Wastewater

Facilities Development Program, April 1977, for a complete system inventory.

Gravity Systems

Central Zone. The gravity collection system in this zone is the oldest one in Alexandria. In its western portion, two systems collect the wastewater and convey it to different points. The Port Est system has many smaller collectors tributary to the Port Est collector, which follows the Eastern Harbor shoreline to the Kait Bey pump station. From there, sewage is pumped to the sea through the existing Kait Bey outfall. The remaining portion is served by local sewers, which discharge directly into the Western Harbor. A new interceptor and pump station now under construction will convey the wastewater now emptying into the Western Harbor to the Port Est system, where it will also be temporarily discharged to the sea through the existing Kait Bey outfall.

In the eastern portion of the central zone, three systems convey the wastewater to eventual disposal through the Kait Bey outfall. Two of the systems are served by the Sporting and Smouha pump stations, which pump the flows into the third system. The combined wastewater is then conveyed by gravity in collectors running parallel to the Eastern Harbor shoreline to the Kait Bey pump station. Stormwater that exceeds the capacity of the stormwater conduits overflows at points along the harbor. The excess flow at the Sporting pump station flows directly to the Mediterranean Sea at the station, while that from Smouha pump station flows by gravity to the Smouha Drain, a large canal located south of the Mahmoudiya Canal.

Western Zone. The western zone collection system is presently in an interim state of operation until the West Treatment Plant is completed. Flows from the northern three tributary areas flow directly to the Western Harbor while those from the southern four areas go directly to Lake Maryout. When the West plant is finished, all flows will be pumped there for treatment and disposal to Lake Maryout in a single outfall. This plan may be altered depending on the wastewater management plan selected.

Eastern Zone. The eastern zone collection system conveys all wastewater from this zone to two points to the south. The western portion of the zone flows to the Smouha Drain by gravity through two large collectors which discharge under the Mahmoudiya Canal through a siphon. The Smouha Drain eventually discharges to Lake Maryout. Two collectors, the Hagar El-Nawatiya and the Mohamed Ali, serve large areas adjacent to the Smouha Club. The Sidi Bishr, Sarwat, and Glym pump stations serve the areas north along the Mediterranean Sea and pump in series to the Mohamed

Ali system. The Sidi Gaber station is currently not operating, but is planned to pump stormwater to the sea.

The eastern portion of the zone is a system that has a series of 10 local subsystems, each with its own pump station and force main discharging to the next pump station westward. Eventually all flow goes to the No. 11 East pump station either directly or through the Abu Soliman interceptor. From the No. 11 East pump station, the wastewater is pumped to the East plant, when it is operating, or to the Smouha Drain when the plant is out of service. Currently all of the system is operating in a surcharged condition, so that mixed storm and sanitary sewage is discharged to the sea and the East Treatment Plant or Smouha Drain.

Existing System Condition and Deficiencies. Some of the sewers serving Alexandria are in good condition and have been designed and constructed according to modern standards. These can be incorporated into an expanded system and should provide many more years of useful service. However, many older interceptors were constructed at very flat slopes. This causes solids to settle and drastically increases maintenance requirements. In addition, the former practice of matching invert elevations at pipe size changes causes flooding in upper areas when the downstream system is full.

Several sections of collapsed sewers urgently need repair. Some sewers have been or are currently being replaced under the Top Priority Project (TPP) program, but more sections need to be repaired or replaced. A/GOSD staff are currently preparing a list of additional collapsed sewers. A concentrated program should be initiated to start immediate repairs on these sewers.

In addition to older portions of the system that are surcharged, the newer eastern zone system (consisting of dual collectors for sanitary and storm sewage) also operates in a surcharged condition. This surcharge is caused by the improper operation of the pump stations and not by interceptor deficiency. If the pump stations were operated correctly, the collection system would be adequate to handle flows at the present time.

Table H-2 shown in Appendix H summarizes the deficiencies in the present system. It was compiled after analysis of the capacities and velocities of the existing sewers and is based on information obtained through discussion with A/GOSD staff.

Major Pump Stations and Force Main Systems

Appendix G of this report evaluates the existing pump station and force main systems. The reader should refer to that appendix for a detailed discussion of those systems as well as Special Report No. 2 of the Alexandria Wastewater Facilities Development Program, April 1977, for a complete system inventory.

Central Zone. Four pump stations are located in the central zone: Sporting, Bisiry, Sinin Pasha, and Kait Bey.

The Sporting station discharges either stormwater or combined stormwater and wastewater to an overflow to the sea, or wastewater or combined flow to a force main to the gravity interceptor system paralleling the Eastern Harbor shoreline. The station has five pumps: two at 720 m^3/hr and 35 m of head, two at 900 m^3/hr and 25 m of head, and one at 600 m^3/hr and 20 m of head. The force main is 410 mm in diameter and 760 m long.

The basic purpose of the Bisiry pump station is to prevent local flooding from the surcharged Ras El-Tin collector. It pumps to the collector general interceptor through a force main 210 m long and 510 mm in diameter. The station's rated capacity is 600 m^3/hr at 15 m total head.

The Sinin Pasha pump station also relieves local flooding in the Mohamed Ali Square area. The excess flow from local collectors is pumped to the collector general interceptor through a force main 510 mm in diameter and 70 m long. The rated capacity of the station is 600 m^3/hr at a total head of 15 m.

The Kait Bey pump station is a major facility that pumps into the existing outfall. It has a rated capacity of 3000 m^3/hr at a total head of 5 m. There is no force main; rather, the sewage is lifted to a receiving chamber to provide sufficient head to discharge by gravity through the outfall.

Western Zone. Western zone pump stations include:

- No. 1 West
- No. 2 West
- No. 3 West
- New Furn El-Gueraya
- Old Furn El-Gueraya

- Industries
- Gheit El-Enab Main
- Gheit El-Enab Auxiliary
- Mohsein Pasha

The No. 1 West station is not presently operating because the West Treatment Plant is not yet completed. When it becomes operational, the No. 1 West pump station will lift sewage from Gabbary collector to the plant headworks. The station has a rated capacity of 1440 m^3/hr at a total head of 13 m. It discharges through a force main 1300 m long and 610 mm in diameter.

At present, the No. 2 West pump station pumps directly to the Western Harbor. When the West Treatment Plant is completed, however, it will pump directly to the No. 3 West pump station, which in turn, will pump to the plant. The rated capacity of the station is 576 m^3/hr or 436 m^3/hr at 15 m or 8.7 m of total head, respectively, depending on high-speed or low-speed, single-pump operation. The proposed force main to the No. 3 West pump station is 1150 m long with a 356-mm diameter.

When the West Treatment Plant is completed, the No. 3 West pump station will discharge to its headworks. At that time this station will also receive wastewater from the No. 2 West pump station. Until completion of the West Treatment Plant, the No. 3 West pump station will continue to pump to a nearby agricultural drain that flows to Lake Maryout. The station's rated capacity is 1440 m^3/hr at a total head of 14 m. The force main is 780 m long and 559 mm in diameter.

The new Furn El-Gueraya pump station is intended to replace the old Furn El-Gueraya station. It will pump both wastewater and combined wastewater and stormwater. The wet well at the station is compartmentalized, one for dry weather flow and a second for wet weather flow. The dry weather flow will be pumped to the West plant when it is completed, while wet weather flow will be discharged to the Gabbary collector for final discharge to Lake Maryout. The rated capacity of this station is 720 m^3/hr at a total head of 10 m. Special Report No. 2 for the Alexandria Wastewater Facilities Development Program indicates that the new Furn El-Gueraya station will not be able to pump wastewater to the new waste treatment plant when it is complete because of improper pump selection. According to this report, the estimated head required to pump to the West plant at rated discharge is about three times the rated pump head and exceeds the pump shutoff head. At the present time, flow from the Gheit El-Enab collector, which is tributary to the new pump station, is diverted to the old Furn El-Gueraya station because the West plant is not completed.

The old Furn El-Gueraya pump station receives wastewater from a collector serving the Furn El-Gueraya tributary area and a portion of Gheit El-Enab tributary area. All sewage thus collected is pumped northward underneath the Mahmoudiya Canal to the Mahmoudiya east collector where it flows by gravity toward the Mohsein Pasha pump station. However, most of the sewage returns to the south side of the canal through the Mohsein Pasha bypass siphon located just south of the Mohsein Pasha pump station. The sewage then flows by gravity past Gheit El-Enab main pump station into an open drain that discharges into Lake Maryout. The station's rated capacity is unknown, but it contains two pumps with capacities of 300 m^3/hr and 600 m^3/hr, respectively. Its force main is 204 mm in diameter and about 150 m long. This pump station will be abandoned when the new Furn El-Gueraya station is placed into operation.

The Industries pump station pumps mostly untreated industrial wastewater to Lake Maryout at present through a temporary force main that is 700 m long and 610 mm in diameter. It will pump wastewater to the West Treatment Plant when it is completed through a force main 3500 m long; approximately 1600 m of the force main is 610 mm in diameter, and 1900 m of it is 914 mm in diameter. The pump station's rated capacity is 2160 m^3/hr and 1440 m^3/hr at high and low speeds, respectively, and the heads are, respectively, 23 m and 10 m.

The Gheit El-Enab main pump station receives sewage from the eastern part of Gheit El-Enab and lifts it to an open pit adjacent to the station structure. The sewage then flows by gravity in an open drain leading to Lake Maryout. When the lake is high, it overflows into a sewer near the Industries pump station, which returns it to the station's wet well. Two horizontal pumps then pump this reverse flow into the open drain adjacent to the station. The pump station's rated capacity is unknown, but the six pumps vary in capacity from 300 m^3/hr at 4 m of total head to 900 m^3/hr at 15 m of head.

The Gheit El-Enab auxiliary pump station has two pumps, each with a rated capacity of 600 m^3/hr at about 10 m of total head and one pump (900 m^3/hr) at about 25 m of total head. This station serves the south-central portion of Gheit El-Enab and is intended to pump wastewater through 1200 m of force main 254 mm in diameter to the Gheit El-Enab main pump station. Due to problems with the force main, this station has apparently never worked as designed. Instead, sewage is pumped to the Furn El-Gueraya collector, which currently conveys it to the old Furn El-Gueraya pump station.

The Mohsein Pasha pump station receives wastewater from a sewer in Mohsein Pasha Street serving the Mahmoudiya east and west collectors and local lateral sewers. Sewage is then pumped through either of two force mains, each 610 mm in diameter, for about 370 m to the Salah El-Din

collector, from which it is eventually conveyed by gravity to the Kait Bey pump station for disposal in the sea. Flows in excess of the station's capacity flow south through a siphon beneath the Mahmoudiya Canal to an open channel that discharges into Lake Maryout. The pump station has four pumps: two at 1200 m^3/hr at 15 m of head and two at 480 m^3/hr at 11 m of head.

Eastern Zone. The eastern zone has 19 pump stations: Numbers 1 East through 11 East, Montazah, Mamoura, Hospitals, Sidi Bishr, Sarwat, Glym, Sidi Gaber, and Smouha.

The No. 1 East pump station receives sewage from its own tributary area and the Montazah, Mamoura, and Hospitals pump stations. Sewage is pumped directly to the No. 5 East pump station through about 2000 m of force main 406 mm in diameter before entering a force main 559 mm in diameter, which also conveys sewage from the No. 3 East pump station. In addition, No. 1 East pumps stormwater through about 70 m of force main 356 mm in diameter to a force main 508 mm in diameter that discharges to the Mediterranean Sea at Mandara Beach. The latter also conveys stormwater from the No. 2 East pump station. Sewage from Mamoura can also be diverted directly to the No. 2 East pump station in order to reduce the peak demand on No. 1 East during the summer. The station's rated capacity is 648 m^3/hr or 428 m^3/hr at 34 and 15 m of total head, respectively, with high-or low-speed, single-pump operation. The stormwater pumps have rated capacities of 516 or 436 m^3/hr at 15 and 8.7 m of head, respectively, at high or low speeds, respectively.

The No. 2 East pump station receives sewage from its own tributary area and pumps it directly to the No. 6 East pump station. Sewage is pumped through about 1250 m of force main 356 mm in diameter before entering a force main 508 mm in diameter, which can also convey sewage from the No. 4 East pump station. In addition, No. 2 East pumps stormwater from its own tributary area and through about 1250 m of force main 406 mm in diameter before entering a force main 508 mm in diameter, which also conveys stormwater from the No. 1 East pump station. Stormwater from both stations is discharged directly to the Mediterranean Sea at Mandara Beach. The station's rated capacity is similar to No. 1 East.

The No. 3 East pump station receives sewage from its tributary area and pumps it directly to the No. 5 East pump station. Sewage is pumped through about 100 m of force main 356 mm in diameter before entering a force main 559 mm in diameter, which also conveys sewage from the No. 1 East pump station. The No. 3 East pump station also pumps stormwater from its own tributary area and the Nos. 5 and 4 East pump stations. This stormwater is pumped through a force main 559 mm in diameter directly to the Mediterranean Sea at Bir Masoud. The rated capacity of the No. 3 East pump station is 540 m^3/hr at 17 m of head for the sewage pumps and 1440 m^3/hr at 10 m of head for the stormwater pumps.

The No. 4 East pump station pumps sewage from its own tributary area and directly to the No. 6 East pump station. Sewage is pumped through about 50 m of force main 356 mm in diameter before entering a force main 508 mm in diameter, which also conveys sewage from the No. 2 East pump station. The No. 4 East pump station also pumps stormwater from its own tributary area directly to No. 3 East pump station. Stormwater is pumped through about 600 m of force main 356 mm in diameter before entering a force main 406 mm in diameter, which also conveys stormwater from No. 5 East. The pump station's rated capacity is 540 or 410 m^3/hr at a total head of 17 and 9.8 m and high- or low-speed, single-pump operation, respectively.

The No. 5 East pump station receives sewage from its own tributary area and from the Nos. 1 and 3 East pump stations. Sewage is pumped through a force main 559 mm in diameter and 2950 m long to the Abu Soliman siphon, which passes under the Montazah Canal and discharges into the Abu Soliman collector. The sewage then flows by gravity to the No. 11 East pump station. The No. 5 East pump station also pumps stormwater from its own tributary area through a force main 305 and 406 mm in diameter, and 970 m to the stormwater wet well at the No. 3 East pump station. The station's capacity for sewage is 1440 or 1102 m^3/hr at 25 or 15.5 m of head, with high- or low-speed, single-pump operation. For pumping stormwater, the capacity is 360 or 248 m^3/hr at heads of 20 and 9 m, respectively, at high- or low-speed, single-pump operation.

The No. 6 East pump station receives sewage from the Nos. 2 and 4 East pump stations and its own tributary area. Sewage is pumped through a force main 509 mm in diameter through the Abu Soliman siphon to the Abu Soliman collector. The sewage then flows by gravity to the No. 11 East pump station. The No. 6 East pump station also receives stormwater from its tributary area and pumps it through a force main 405 mm in diameter directly to the Mediterranean Sea at Sidi Bishr. The pump station's capacity for pumping sewage is the same as for the No. 5 East pump station. For pumping stormwater, the rated capacity is 540 or 410 m^3/hr at 20 or 9 m of head and at high- or low-speed, single-pump operation, respectively.

The No. 7 East pump station receives sewage from its own tributary area and pumps it directly to the Abu Soliman collector. Sewage is pumped through about 300 m of force main 356 mm in diameter before entering one 457 mm in diameter, which also conveys sewage from the No. 8 East pump station. Once in the Abu Soliman collector, sewage flows by gravity to the No. 11 East pump station. The No. 7 East pump station does not have separate stormwater pumping capability. Its rated capacity is the same as for the No. 4 East pump station.

The No. 8 East pump station receives sewage from its own tributary area and pumps it directly to the Abu Soliman collector. Sewage is pumped

through about 530 m of force main 406 mm in diameter before entering a force main 451 mm in diameter, which also conveys sewage from the No. 7 East pump station. Once in the Abu Soliman collector, sewage flows by gravity to the No. 11 East pump station. The No. 8 East pump station does not have separate stormwater pumping capability. Its rated capacity is the same as for No. 4 East.

The No. 9 East pump station is intended to pump sewage from its own tributary area to the No. 10 East pump station, but it has never been placed into operation. Flow tributary to this station is diverted through the Hagar El-Nawatiya collector to the Smouha Drain. According to Special Report No. 2 for the Alexandria Wastewater Facilities Development Program, the difference in elevation between the influent sewer invert at the No. 9 East pump station and the influent sewer invert at the No. 10 East pump station would allow construction of a gravity sewer with capacity equal to the rated pump discharge. The need for the No. 9 East pump station is therefore not apparent. Its rated capacity is 396 and 263 m^3/hr at 53 and 10 m of head, at high- or low-speed, single-pump operation, respectively.

The No. 10 East pump station receives sewage from its own tributary area and pumps it through a force main 508 mm in diameter directly to the wet well of the No. 11 East pump station. It is also intended to receive sewage from the No. 9 East pump station, which has not been placed into operation. The No. 10 East pump station was also intended to pump directly to the Hagar El-Nawatiya collector near the No. 9 East pump station; however, the stormwater force main has not been completed and the stormwater pump is not used. Therefore, any stormwater received at this pump station is eventually discharged to the No. 11 East pump station. The rated capacity of the station for pumping sewage is 1296 and 871 m^3/hr at 10 and 4.5 m of head at high- or low-speed, single-pump operation, respectively. For stormwater pumping, the rated capacity is the same as for the No. 6 East pump station.

The No. 11 East pump station receives sewage directly from the No. 10 East pump station and the Abu Soliman collector. The Abu Soliman collector receives sewage from its own tributary area and the remainder of the eastern zone through the Nos. 5, 6, 7 and 8 East pump stations. Discharge from the Montazah, Mamoura, and Hospitals pump stations also eventually reaches the No. 11 East pump station. Sewage thus collected is pumped through about 4000 m of force main 1200 mm in diameter directly to the headworks of the East Treatment Plant (when operating) or to the Smouha Drain when the plant is bypassed. The rated capacity at high-speed, single-pump operation is 4320 m^3/hr at 28 m of head, and at low-speed, single-pump operation the capacity is 2898 m^3/hr at a head of 11.8 m.

The Montazah pump station receives sewage from its own tributary area and pumps it through about 450 m of force main 294 mm in diameter to a sewer located in Horreya Avenue. Sewage then flows by gravity to the No. 1 East pump station. The Montazah pump station also pumps stormwater from its own tributary area through about 550 m of force main 294 mm in diameter directly to the Mediterranean Sea near the western wall surrounding the Montazah Palace grounds. For pumping sewage or stormwater, the rated station capacity at high-speed, single-pump operation is 288 m^3/hr at 25 m of head. At low-speed, single-pump operation, the capacity is 191 m^3/hr at 11 m of head.

The Mamoura pump station pumps sewage from its own tributary area through about 3500 m of force main 356 mm in diameter directly to the wet well of the No. 1 East pump station. Originally, output from this pump station was not included in the design criteria for the No. 1 East pump station and, because of the influx of summer residents, its discharge contributes to that station's capacity deficiency. A second force main leading to the No. 2 East pump station provides the flexibility to pump to either station. Rated capacity is 432 m^3/hr at a total head of 35 m.

The Hospitals pump station receives sewage from several hospitals and conveys it through a force main 152 mm in diameter to the No. 1 East pump station. Its rated capacity is 288 m^3/hr at a total head of 15 m.

The Sidi Bishr pump station serves a small tributary area along the Mediterranean Sea and pumps sewage through a force main 254 mm in diameter to a gravity collector about 350 m west along the Corniche. This station can also pump directly to the sea by using a second force main 254 mm in diameter, which discharges about 220 m west of the station location. The rated capacity is 300 m^3/hr at about 8 m of head.

The Sarwat pump station receives sewage from its tributary area, which includes the Sidi Bishr pump station and lifts it to an adjacent manhole located in the Corniche. The sewage then flows by gravity to the Glym pump station. The station's rated capacity is similar to that of the Sidi Bishr pump station.

The Glym pump station receives sewage from its own tributary area and the Sarwat pump station (including Sidi Bishr), and pumps it through either of two parallel force mains 457 mm in diameter to a gravity collector about 170 m west along the Corniche. The sewage then flows by gravity to Horreya Street past the old Sidi Gaber pump station and eventually into the Smouha Drain. Ultimately, this sewage is pumped into Lake Maryout. The station has four pumps: one rated at 480 m^3/hr at 15 m of head, two rated at 480 m^3/hr at 10 m of head, and one rated at 540 m^3/hr at 25 m of head.

The new Sidi Gaber pump station will have two pumps that will be able to pump up to 121,000 m^3/day at 18 m of total head. This pump station will be able to pump flows through either of two force mains 610 m in diameter directly to the Mediterranean Sea. However, according to Special Report No. 2 (April 1977), this pump station is unnecessary. The flows that this station would be pumping could be accommodated by the Cromer collector, which has a capacity of 260,000 m^3/day. This collector could accept the full capacity of its two principal contributors (the Abu Qir and Sidi Gaber collectors) and the flow designated for the Sidi Gaber pump station without becoming surcharged.

The Smouha pump station receives sewage and industrial waste from its tributary area and pumps it through about 1050 m of force mains 660 mm in diameter to the Horreya-New collector. Sewage then flows by gravity to the existing Kait Bey pump station for disposal in the sea. Flow in excess of station capacity continues by gravity to the Mohamed Ali collector, which conveys it to the Smouha Drain. This sewage is ultimately discharged into Lake Maryout. The pump station has two pumps at 900 m^3/hr and two at 1800 m^3/hr rated at a total head of 26 m.

Existing System Condition and Deficiencies. Many deficiencies are common to most of the pump stations in the system. These include:

- Inadequate wet well volume
- Poor access to wet and dry wells
- Poor wet and dry well ventilation and lighting
- Insufficient bottom slope in wet wells which allows solids to collect, decompose, and produce explosive or toxic gases
- Improper pump suction design
- Excess suction and discharge velocity
- Lack of standby power
- Lack of flowmeters
- Potential presence of explosive gases in the pump rooms of stations that pump both stormwater and sewage
- Inefficient pump operation due to improper pump selection

- Lack of screening equipment in disrepair
- Presence of corrosive and toxic industrial wastes that have caused or will cause severe deterioration of equipment--particularly true at the Industries pump station in the western zone
- Inoperability of the direct current power supply for circuit breaker control

For a detailed discussion of specific pump station and force main deficiencies, the reader should refer to Appendix G.

Treatment System

The existing Alexandria wastewater management system includes two treatment plants: The East plant and the West plant. The East plant is designed to provide secondary treatment while the West plant, which is still under construction, will provide primary treatment. For a complete description of these facilities and recommended improvements, the reader is referred to Appendices E and F for the East and West plants, respectively.

East Treatment Plant. The East plant receives flow from the No. 11 East pump station. The plant is located near the Hydrodrome in the Smouha area, and its discharge is to the Qala Drain leading to Lake Maryout. The plant capacity at average flow is 45 Ml/day and at peak flow is 70 Ml/day.

The East plant is designed to provide primary clarification and activated sludge secondary treatment. The incoming wastewater is screened by a manually cleaned bar screen in the influent channel preceding flow measurement by a Venturi flume. Next, grit is removed in six aerated grit chambers and spread on the land around the plantsite. The wastewater then is pre-aerated in six pre-aeration tanks for removal of scum and other floating debris by flotation and skimming. It then receives primary clarification and activated sludge secondary treatment prior to being discharged to the Qala Drain.

Scum from pre-aeration is disposed of in a landfill, and scum and sludge from the primary clarifiers are air-dried and made available as a soil amendment to area farmers. Sludge from the primary clarifiers includes both primary and waste activated sludge.

West Treatment Plant. The West plant, located on the north shore of Lake Maryout, is designed to provide primary treatment to an average

wastewater flow of 85 Ml/day. This plant is presently under construction and has not yet operated. When completed it will provide pretreatment by grit removal, screening or comminuting, and pre-aeration; and primary treatment by sedimentation. The incoming wastewater flow will be measured by a Venturi meter and then comminuted. Alternatively, it may be screened by manually cleaned bar screens. The flow will then be degritted in aerated grit chambers and pre-aerated for scum removal. The wastewater will then be treated by primary sedimentation prior to the planned, temporary discharge to Lake Maryout. Scum and grease removed in pre-aeration are to be disposed of by landfilling. Primary scum and sludge will be air-dried on sand drying beds, and dried sludge will be made available to local farmers as a soil amendment.

Existing Plant Conditions and Deficiencies. Major deficiencies exist at the East plant that severely limit its capability to perform as intended. These major shortcomings include:

- Lack of appropriate screening at the headworks
- Lack of adequate grit handling capabilities
- Inability to control sludge concentration from the primary clarifiers
- Insufficient plant air supply
- Insufficient plant air piping
- Inappropriate sludge withdrawal mechanism in the secondary clarifiers
- Inadequate return and waste activated sludge pumping and control capabilities
- No effluent disinfection
- Insufficient sludge drying bed and dried sludge storage area
- Inappropriate or inoperable electrical and instrumentation and control systems

Even though the West plant has not yet operated, several major deficiencies have been noted that will limit its intended function. These include:

- Inadequate grit handling capabilities

- Improper sludge hopper design in primary sedimentation basins
- Improper sludge transport capabilities to sludge drying beds
- Insufficient sludge drying bed and dried sludge storage area
- Inappropriate or inoperable electrical and instrumentation and control system

CURRENTLY PLANNED PROJECTS

Sewers, Pump Stations, and Force Mains

In August of 1977, Special Report No. 4 of the Alexandria Wastewater Facilities Development Programs was published as a part of the 1978 Master Plan Study to identify top priority projects (TPP). These are urgently required works for which design should be commenced immediately and construction or implementation undertaken at the earliest possible opportunity. Following is a list of the TPP that were identified in Special Report No. 4:

- Institutional and Operational Changes
  - Implementation of the sewer use law
  - Collection and disposal of wastes
  - Sewer cleaning program
  - Vehicle replacement
  - Prohibition of building in Dekheila
- Repairs and Replacements in the Existing System
  - Contracted sewer cleaning and inspection
  - Collector general relief sewer
  - Western Harbor system
  - Reconstruction of collapsed sewers
  - Manhole cover replacement

- Smouha stormwater drain
- Drainage relief--Mohamed Ali Square
- Sporting pump station and force main
- Bisiry force main extension
- Hospitals pump station and force main
- Rehabilitation of eastern zone pump stations

o New Service to Unsewered Areas

- Ras El-Soda system
- Siouf Keblia system
- Mex-Dekheila system
- Nouzha system
- Abu Qir system

The above projects are in various stages of development. None have been completed, but most recommended construction/renovation projects have a completed design report. The TPP and implementation of the original Master Plan recommendations will correct most of the deficiencies in the existing system. The remainder will be corrected by improvements discussed in this chapter and in Appendices G and H.

Treatment Plants

Besides completion of construction at both the East and West plants, the only planned treatment plant improvements are the installation of new compressors at the East plant. These compressors are intended to supply air for both pretreatment and secondary treatment processes, and will replace failed, smaller units. Completion of the compressor project is anticipated to be mid-1981.

RECOMMENDED SHORT-TERM IMPROVEMENTS

Sewers

Of the deficiencies listed in Appendix H, Table H-2, only two will not be remedied by the completion of the top priority projects or the 1978 Master Plan recommendations. These concern the Ibrahimia and the

Gabbary collectors. Only routine maintenance is required to correct problems in these sewers; therefore, the only additional short-term needs are to emphasize those TPP that are critical for the efficient use of the existing collection system.

As suggested in Appendix H, an intense sewer cleaning program must be implemented. This is also a recommended top priority project. Other nonstructural top priority projects that should be emphasized are collection and disposal of solid wastes, and implementation of a sewer use law.

Implementation of these latter projects will help prevent extraneous and incompatible wastes from entering the sewerage system where they presently cause serious operational and structural problems.

Pump Stations and Force Mains

The immediate need for major pump station upgrading is not excessive, but minor repairs and improvements in housekeeping techniques are needed in all stations. Pump stations appear to be in better condition than was reported following the 1977 inspection done as part of the Alexandria Wastewater Facilities Development Program. However, safety is still an item that needs to be emphasized.

The structural condition of most of the stations is fairly good and no major work is required. The electrical systems of nearly all pump stations are deficient in the area of fault current protection. Most of the equipment is in fairly good condition, but the failure of the d.c. power supplies has rendered the protective relay systems inoperative. Mechanically, a few stations require major changes to meet present needs. These include undersized stations along the coast where the increase in summertime population and the resulting increases in flow far exceed the flows produced by the resident population and wintertime rainflow.

Recommended improvements to the existing force mains consist of repairing leaks and collapsed pipes, and installation of new alternate lines and valving to render diversion capabilities operable. Appendix G contains a complete listing of all recommended short-term pump station and force main improvements.

Treatment Plants

East Plant. Recommended modifications to the East plant include rehabilitation and installation of new equipment to improve :

- Screening facilities

- Flow measurment
- Grit removal and handling
- Pre-aeration system
- Primary clarifiers
- Aeration basins
- Secondary clarifiers
- Plant air supply system
- Sludge pumping facilities
- Electrical and instrumentation and control systems

Also additions to the sand drying beds and installation of disinfection facilities are recommended. Appendix E contains a detailed description of all recommended improvements. These are needed regardless of the final wastewater disposal plan.

West Plant. Recommended improvements to the West plant are based on providing primary treatment to an average flow of 85 Ml/day, which is the present intention. This would be the case for at least 5 years if land application is the selected wastewater disposal option. That period of time would allow implementation of the land application system. After that, the plant would become an industrial waste pretreatment facility with the effluent being land applied. If sea disposal is selected, then the plant would be expanded to accommodate an average flow of 395 Ml/day from the western and central zones. Such improvements would be of a long-term nature and are discussed in Appendix J.

For providing primary treatment to : average flow of 85 Ml/day, modifications are recommended to the following plant systems:

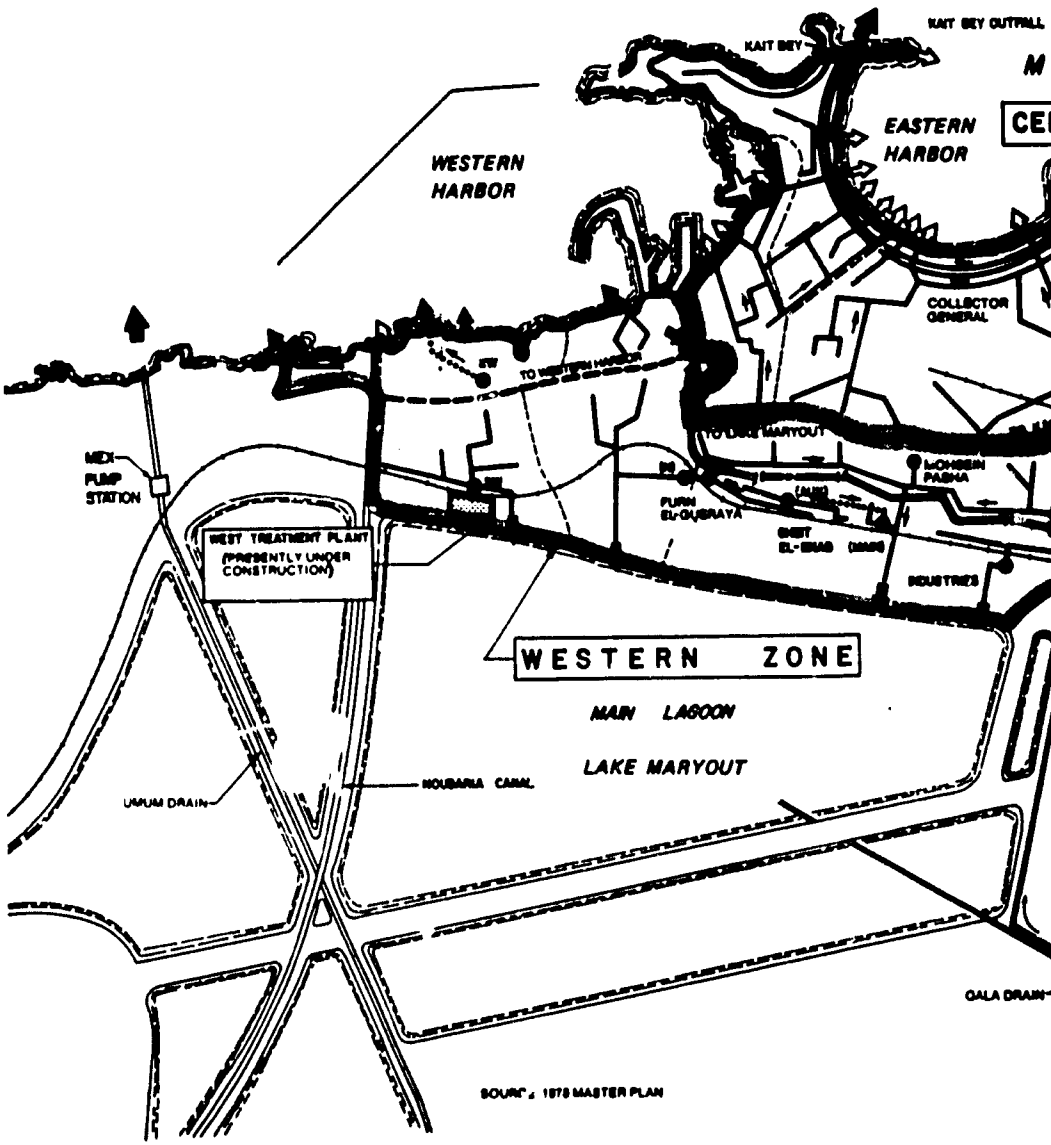
- Flow measurement
- Grit removal and handling
- Comminuting and screening
- Pre-aeration
- Primary clarifiers

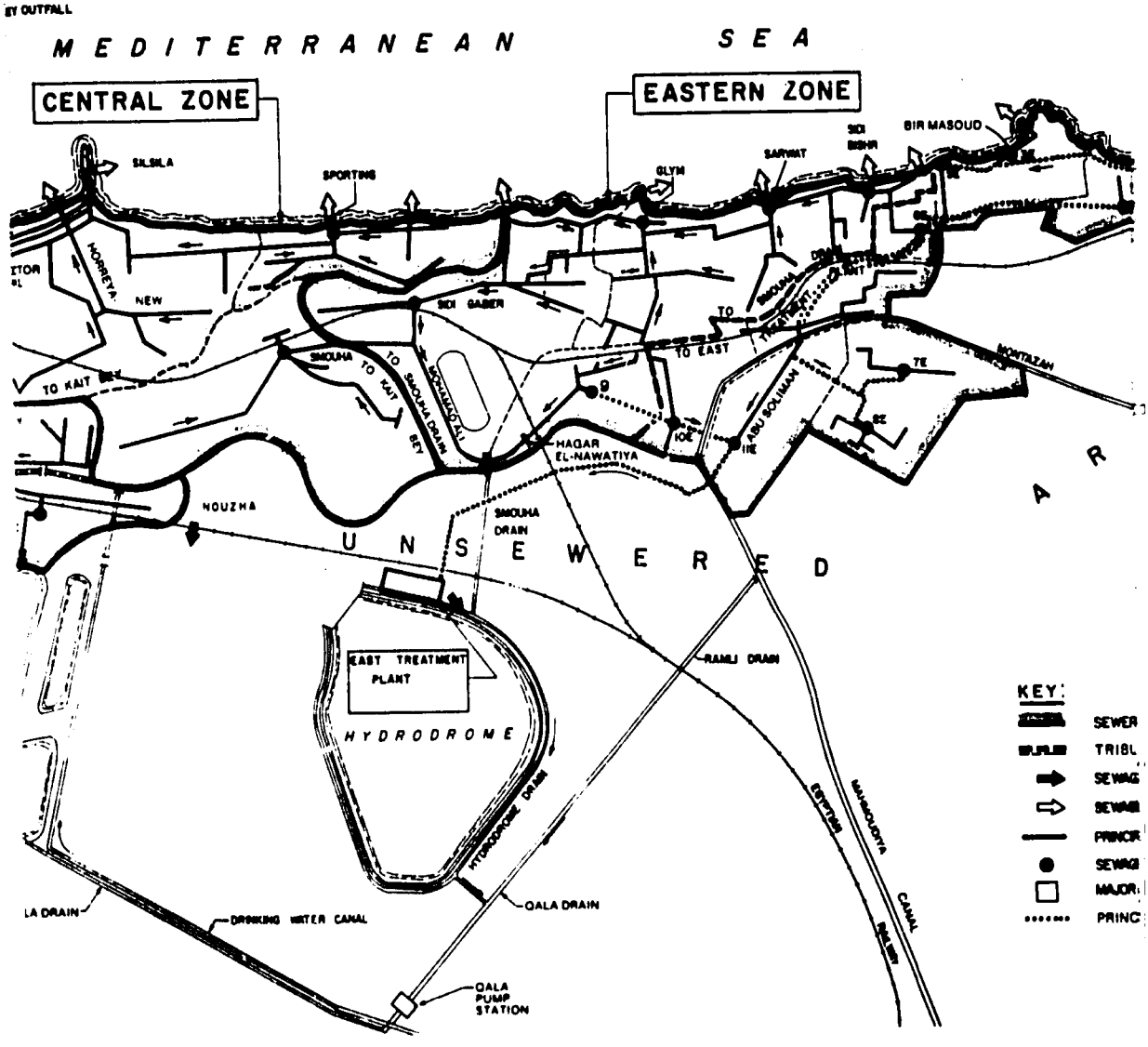
- Sludge transport
- Electrical systems and instrumentation and control

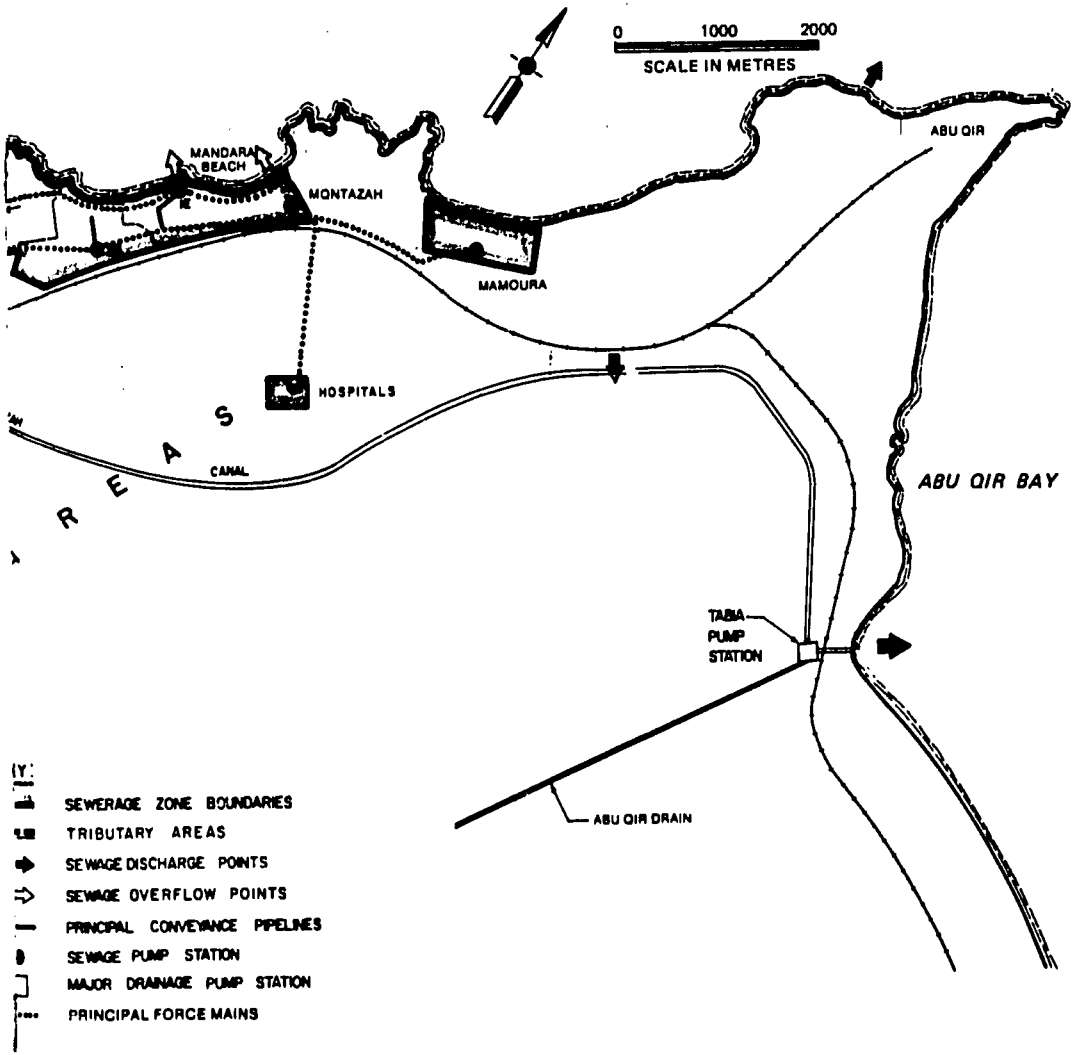
Further additions to the sand drying beds are recommended. Appendix F contains a detailed description of the recommended improvements.

Costs of Recommended Short-Term Improvements.

Estimated capital costs for the recommended short-term improvements are listed in Appendices E, F, G, and H.


KAIT BEY OUTFALL
KAIT BEY
EASTERN HARBOR
WESTERN HARBOR
COLLECTOR GENERAL
TO WESTERN HARBOR
TO LAKE MARYOUT
MIDI PUMP STATION
WEST TREATMENT PLANT (PRESENTLY UNDER CONSTRUCTION)
PUMP EL-GUSRAYA
LOHSSIN PASHA
INDUSTRIES
WESTERN ZONE
MAIN LAGOON
LAKE MARYOUT
NOUBARIA CANAL
UMUM DRAIN
QALA DRAIN
SOURCE: 1979 MASTER PLAN


MEDITERRANEAN SEA
CENTRAL ZONE
EASTERN ZONE
SILSILA
SPORTING
GLYM
SARWAT
BIR MASOUD
NEW
SIDI GABER
SMOUHA
TO KAIT BEY
TO EAST
HAGAR EL-NAWATIYA
ABU SOLIMAN
MONTAZAH
NOUZHA
SMOUHA DRAIN
UNSEWERED
EAST TREATMENT PLANT
HYDRODROME
RAMLI DRAIN
HYDRODROME DRAIN
QALA DRAIN
DRINKING WATER CANAL
QALA PUMP STATION
MAHMOUDIYA CANAL
KEY:
SEWER
PRINC
MAJOR



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE 3-1

EXISTING SEWERAGE SYSTEM

A/GOSD WWCG W/ECC

CHAPTER 4

SEA DISPOSAL ALTERNATIVE IB

INTRODUCTION

The 1978 Master Plan recommended sea disposal of sewage treatment plant effluent through a 10-km outfall at Kait Bey and a 10-km outfall in the vicinity of Sidi Bishr. The proposed sewered area was divided into three zones:

- The central zone with a treatment plant at Kait Bey
- The western zone with an enlarged treatment plant at the existing West plant site
- The inner and outer eastern zone with treatment plant near the Ras El-Soda area

The treatment plants were to provide preliminary treatment before discharge. Additionally, the East plant was to provide secondary treatment to flows from the Nouzha area prior to their discharge through the Qala Drain system to Lake Maryout.

Following publication of the 1978 Master Plan, an environmental impact statement recommended primary treatment and effluent disinfection prior to sea disposal. Accordingly, one of the purposes of this study was to reexamine a system to discharge effluent into the sea and to determine the possible impacts.

This review included an examination of the following changes to the data and proposed alternative in the 1978 Master Plan.

- Updated flows resulting from the planning study as discussed in Chapter 2
- Different locations for treating flows from the central and eastern zones flows as discussed in Appendix J
- Alterations in the wastewater conveyance system as discussed in Appendix M
- The addition of primary sedimentation, effluent disinfection, and sludge management processes as discussed in Appendix J

- Adjustments to the preliminary designs of the sea outfall as discussed in Appendix L

A complete system using sea disposal of primary effluent would consist of the following components:

- Wastewater collection as described in Appendix H
- Wastewater conveyance as described in Appendix M
- Wastewater treatment as described in Appendix J
- Effluent and sludge disposal as described in Appendix L

The reexamination, cost review, and economic assessment of the sea disposal alternative presented in this chapter considered the conveyance, treatment, and disposal components as integral parts of one total system. That is, costs and other factors were considered together. Wastewater collection system costs and impacts are not considered in this chapter as the collection system is common to all conveyance, treatment, and disposal alternatives considered.

Principal components comprising the sea disposal alternative are depicted on Figure 4-1. The central zone flows would be conveyed to the Central pump station where they would be pumped to the West plant. Western zone flows would be collected and pumped to the existing plantsite and combined with the central zone flows. The inner and outer east zone flows would be collected and pumped to the Ras El-Soda plant.

The West plant would be a primary treatment facility having an average dry weather flow capacity of 395 Ml/d. It would be expanded from the existing (under construction) 85 Ml/day primary plant. This facility would be upgraded at the same time capacity is added. Upgrading should include all recommendations for this facility presented in Appendix F, West Plant Evaluation, except that no additional sludge drying beds would be added. Sludge from the full 395 Ml/d plant would be mechanically dewatered prior to composting.

Figure 4-2 is a preliminary layout of the proposed 395 Ml/day facility. The effluent would be disinfected and pumped to the Central pump station near the Eastern Harbor. From there it would be pumped through an outfall 8 km long, terminating 40 m below sea level in a diffuser 450 m long. The West plant, its influent conveyance facilities, and effluent disposal facilities would be constructed in one stage. Construction in two or more stages is not appropriate because the 1991 flows are expected to be almost 86 percent of the design year 2000 flows. Staged construction would not be cost-effective in this situation.

The flows conveyed to the Ras El-Soda site would receive treatment at a new primary plant having an average dry weather flow capacity of 585 Ml/d. Figure 4-3 is a preliminary layout of the proposed facility. The effluent would be disinfected. An onsite pump station would pump the effluent to and through the Sidi Bishr outfall. This outfall would be 2600 mm in diameter and 10 km long, and would terminate in a diffuser 400 m long located 55 m below sea level. Construction of the plant would be staged by delaying construction of 25 percent of the primary sedimentation tanks, 12.5 percent of the chlorination facilities and 20 percent of the employee housing until 1990. Year 1991 flows are estimated at 75 percent of the design year 2000 flows. No year 1990 capacity staging of the influent conveyance or effluent disposal facilities is recommended because such staging would not be cost-effective.

Processing of solids removed in the primary sedimentation step at the West and Ras El-Soda plants would be by onsite vacuum filtration. Dewatered solids would be composted onsite at the West plant. The resulting stabilized solids would be available to local farmers for use as a fertilizer/soil conditioner. Solids dewatered at the Ras El-Soda site would be transported by truck 10-15 km to a composting site south of the plant near the Delta Highway to Cairo. Composted sludge would be available for use by local farmers.

The East plant and its influent conveyance system are common components of both the sea disposal and land application alternatives. In each case, the East plant would be upgraded to an average daily 45 Ml/d capacity and would provide activated sludge secondary treatment. Necessary plant modifications are described in Appendix E. A preliminary plant layout is presented on Figure 4-4.

The East plant currently receives its influent from the No. 11 East pump station, as shown in Figure 4-1. Under the sea disposal alternative, these flows would ultimately be discharged to the Ras El-Soda plant once it is operational. The East plant would then serve the Nouzha area and would receive flows from the new Nouzha pump station (70 Ml/d firm capacity).

The nitrified effluent from the upgraded East plant is expected to be of high quality, containing approximately 20 and 30 mg/l BOD and suspended solids, respectively. This effluent would be initially discharged to the existing drain around the Hydrodrome. This drain discharges through the Qala pump station to the Qala Drain, which in turn discharges to the Lake Maryout main basin. Once the plant's ability to produce a high quality effluent is demonstrated, effluent reuse by industry or agriculture in the Hydrodrome vicinity should be encouraged.

Sludge produced at the East plant would be dewatered and dried onsite in drying beds. The dried sludge would be stored for approximately 1-1/2

years to allow for pathogen decay. Following onsite storage, the aged sludge would be made available to local farmers for use as a much-needed soil amendment.

The year 2000 capacity plant upgrade would be constructed in a single stage. Single-stage construction is justified by the need to provide maximum service for flows from the No. 11 East pump station until the Ras El-Soda plant and its associated sea outfall are in service. Single-stage construction of the influent conveyance facilities is also justified on the basis of cost-effectiveness.

FEASIBILITY

The types of wastewater conveyance, treatment, and disposal facilities to be provided under the sea disposal alternative have long histories of successful construction and operation. While many of the works would be massive, the technological bases used in design, construction, and operation are well understood. Applying existing technology in Alexandria would likely be challenging for several reasons in the following order of descending difficulty:

1. Assurance of adequate industrial pretreatment to avoid unacceptable environmental impacts on the sea and prevent operating problems at the East plant

2. Successful activated sludge process control and general operation of the East Treatment Plant, even when assuming adequate pretreatment of industrial discharges to the plant

3. Construction of the outfalls at Kait Bey and Sidi Bishr

4. Operation and maintenance of sludge dewatering facilities at the West and Ras El-Soda treatment plants

5. Construction of the West Treatment Plant expansion and the Ras El-Soda plant

6. Operation and maintenance of the chlorine feed facilities serving the West, Ras El-Soda, and East plants

Construction, operation, and maintenance of system components not mentioned above should be easily accommodated, based on the best demonstrated construction, operation, and maintenance practices evident in the Alexandria area.

Industrial Pretreatment

Much of the sea disposal system's ultimate success in protecting the public health and general environment depends upon its ability to prevent discharge of toxic substances into the environment where they can become concentrated in the aquatic food chain and create hazards. This in turn depends on adequate control and pretreatment of industrial wastewater discharged into the sewer system.

Industrial pretreatment is not overly complex. It has been successfully implemented by many industries (in the United States and Europe) in each category of toxic substances discharge. The greatest challenge in Alexandria will not be acquisition of technology, but institutional controls to assure adherence to required pretreatment performance, both as to schedule and degree. Achievement of adequate pretreatment (or exclusion) of toxic substances will also require that attention be given to financing the pretreatment facilities.

Industrial pretreatment must not stop with control of toxic substances discharged to the sewers. Systems must also be provided to prevent discharges of incompatible wastes into the sewers. Included in this category are wastes having an excessively high or low pH and those containing gross solids, oil, grease, flammables, and excessively high fiber and settleable solids concentrations. Such wastes must be pretreated to avoid (1) stoppages in the collection and conveyance system, causing raw sewage overflows and bypasses, (2) destruction of collection, conveyance, treatment, and disposal facilities by acid attack or closure through scaling or adherence, and (3) overloading of treatment unit processes to the extent treatment is compromised and treatment unit bypass is made necessary.

East Plant Process Control

Successful treatment at the East plant will largely depend on process control and operation of the activated sludge unit process. The activated sludge process design at the East plant can best be described as "high rate". Careful and expert process control will be mandatory. This can best be assured through employment of at least two process control engineers, each meeting educational requirements and having several years of successful operational control at activated sludge facilities similar to those at the East plant. These process control engineers should have full authority over the activated sludge unit process operation.

Outfall Construction

Constructing the sea outfalls will be a challenging undertaking, but operating the outfalls should be relatively easy. Outfall placement is expected to present some problems for even large experienced contractors. The greatest concern centers on quality attainment and control in outfall pipe manufacture. Stringent quality control procedures must be enforced in pipe manufacture, outfall placement, and outfall protection.

Sludge Dewatering

Preliminary design criteria, layouts, and cost estimates assume mechanical dewatering of raw primary sludge by coil media vacuum filters. Should Alternative IB be selected for implementation, sludge dewatering by continuous belt press equipment should be considered as an alternative to vacuum filter dewatering during preparation of the basis of design reports on the West and Ras El-Soda plants. At that time, primary sludge from the East plant could be tested on pilot devices as necessary for final selection between these equipment types and finalization of design criteria.

Operation and maintenance of mechanical sludge dewatering facilities require skill and dedication on the part of the operation and maintenance staffs. Operation would involve polymer mixing and feeding, sludge pumping, sludge conveyance by belt conveyor, vacuum filter operation, and operation of ancillary vacuum pumps, filtrate pumps and numerous drives, and other mechanical devices. The sludge flow to dewatering would have to be maintained at least 16 hours a day. Failure to keep sludge feed concentrations at or above four percent solids, properly operate and maintain equipment, or properly mix and feed polymers can disrupt sludge dewatering. Any prolonged disruption (a day or more) would result in the need to bypass treatment in the primary sedimentation tank, which would significantly reduce disinfection effectiveness. These concerns could best be addressed by retaining at least two sludge dewatering supervisors with extensive vacuum filter experience at each primary plant. These supervisors could then assist in training other staff members.

Treatment Plant Construction

Constructing the Ras El-Soda plant and expanding the West plant would require modern construction techniques, sophisticated construction scheduling, and skilled labor. Of these, acquisition of sufficient skilled labor to install/construct the mechanical, electrical, and control systems would likely present the greatest challenge. Such skilled labor is currently in short supply. However, this shortage could be mitigated through use of expatriate labor having the necessary skills.

Chlorine Feed Facilities

Proper operation and maintenance of chlorine feed facilities would be required to protect the public health. Use should be made of skilled and experienced chlorine systems operations and maintenance personnel to assure continuous and appropriate chlorine feed.

ENVIRONMENTAL IMPACTS

The sea disposal alternative provides for acceptable treatment with a minimum amount of land dedicated to wastewater conveyance, treatment, and disposal systems. Chlorinated primary effluent would be introduced into the Mediterranean Sea at two discharge locations, 8 and 10 km offshore from Alexandria.

Effluent disposal at these distances and at depths of 40 and 55 m should ensure adequate dilution and dispersion of the wastewater with little or no direct impact on people, provided the concerns previously discussed are adequately addressed. Summarized below are the anticipated environmental impacts associated with the conveyance, treatment, and disposal portions of this alternative. This summary is based on the environmental impact sections presented in the appendices dealing with treatment alternatives (J), sea disposal and outfall review (L), and conveyance system alternatives (M).

Conveyance

Construction within limited spaces is required to convey central zone flows to the West Treatment Plant and from the plant to the Kait Bey outfall. This will result in short-term major adverse impacts. The proposed force main route follows existing roadways through densely populated neighborhoods between the West plant and Kait Bey. Approximately 1 km of the route traverses an area of high density commercial uses in the downtown shopping area. Near the existing Kait Bey pump station, the roadway is narrow and congested. Thus, construction will further aggravate existing transportation problems and severely disrupt activities at nearby businesses. However, we do not anticipate the need to demolish buildings.

Construction of the effluent force main between the Ras El-Soda plant and the shoreline will be less difficult. However, some relocation of existing residences will be necessary along this route.

Treatment

Large primary treatment plants would be constructed at the West and Ras El-Soda sites. The existing East plant would remain in operation but would discharge secondary effluent to the Qala Drain.

The long-term adverse environmental impacts associated with each of the three sites include:

- Potential hazards associated with chlorine storage and transport in densely populated areas
- Potential odor problems near the sites

In addition, construction on the Ras El-Soda site would preclude existing alternative plans for multifamily housing.

Effluent Disposal

Effluent discharges to the Mediterranean Sea should not cause major adverse environmental impacts. Effluent dilutions of 1000 to 1 within 1 km of the outfall diffusers and on the order of 4500 to 1 at the Alexandria beach areas should, in conjunction with effluent disinfection, adequately protect public health. Dissolved oxygen reductions near the outfall diffusers should not exceed 10 percent of ambient levels off the Alexandria coast.

An insignificant positive impact on fish production can be expected as a result of wastewater nutrient discharges to the Mediterranean Sea. These nutrient discharges are estimated, in the 1978 Master Plan, at approximately 10 percent of the nutrient load discharged by Nile floods prior to completion of the Aswan High Dam. The Nile floods discharged large quantities of nutrients in short periods of time, enhancing algae blooms and increasing fish production. Unfortunately, the much smaller year-round discharges from the outfalls will be continuously diluted. Increased algae production will be small and highly diluted. The low algal concentration cannot be expected to significantly increase fish production. While any impact on fish, however small, should be positive, no wastewater impact, positive or negative, will be expected on local shellfish production. This 's largely because no shellfish beds are known to exist in the outfall area. Further quantification of wastewater impacts on fish and shellfish will require additional field investigation.

Some minor adverse impacts will be unavoidable. An area encompassing approximately 33 km^2 may be impacted by sediment deposition rates

greater than ten times the normal background rate. Slight increases in the density and dominance of a few benthic organisms may be noted in this area. However, no noticeable change is expected in the local fisheries off the Alexandria coast. The potential for localized algal blooms at the discharge sites exists, but these should have a negligible impact due to their remote offshore location.

CONSTRUCTIBILITY

Constructibility of some sea disposal system elements has been previously discussed. Other than outfall and treatment plant construction at the West and Ras El-Soda sites, no construction problems are anticipated to require solutions beyond local (Egyptian) capability. The following sections briefly review the constructibility of the major system components.

Conveyance System

Pumping and pipeline elements comprising the sea disposal conveyance system would be constructed with less difficulty than would elements of the treatment and outfall facilities. Local contractors are familiar with the local materials and construction techniques to be used.

Control of ground water will likely be the greatest problem encountered in construction of pipelines, particularly deep gravity interceptors. Sophisticated ground water control will be important to permit pipe laying; it will also be necessary to avoid structural damage to adjacent structures.

Materials and equipment supply problems in construction of pump stations and pipelines could exist. These problems can be readily overcome, however, by prepurchasing large pipe, pump motors, and electrical gear for subsequent installation by the various contractors. In many cases, tenders for materials and equipment contracts can be sought several months in advance of design completion on a given system element.

Treatment Plants

Lack of a large local skilled labor pool would likely inhibit timely treatment plant construction. This can be mitigated by use of skilled expatriate labor where the local labor supply is inadequate.

Sophisticated construction scheduling and management techniques would be needed. These modern construction methods would be needed to complete

construction in a reasonable amount of time (3 years). Each plantsite should, for instance, be provided with its own concrete batch plant. Concrete "gang forming" techniques should be employed using reusable, lined panel forms. Concrete must be poured mechanically not manually. All major treatment plant equipment should be prepurchased to avoid adverse impacts on the construction schedule and to provide the maximum applicability and quality of equipment incorporated into the work.

Outfalls

The outfalls would pose the greatest construction challenge associated with the sea disposal alternative. Very few contractors are experienced in this type of construction. Similarly, the labor skills needed are relatively unique and specialized. Construction hazards are higher than those associated with onshore construction, and thus risks are also greater. These challenges would be translated into higher costs, since the contractors must add contingency costs and higher profit margins to cover the risks taken. These additional costs are reflected in the estimated outfall costs.

Quality control is generally difficult, but achievable, in casting the large diameter reinforced concrete outfall pipe and placing it in the sea. Inspection and testing of pipe before it is placed will be an absolute necessity, as will be rejection of inferior pipe and joints.

SCHEDULE

Figure 4-5 presents the anticipated schedule for sea disposal system implementation. This schedule was based on several assumptions:

- The magnitude of work as it can be defined currently
- Use of modern construction, construction scheduling, and construction management techniques
- Importation of expatriate labor to supplement local labor supplies where these are inadequate by reason of numbers or skill level
- Time of construction required for similar facilities in the United States, adjusted to perceived individual project element construction conditions in Alexandria at the time of construction

The outfall construction is on the critical path of total system completion. This should be kept in mind during review of the subsequent discussion on principal system element schedules.

East Plant Upgrade

This project is common to both the sea disposal and land application alternatives. Appendix E recommends specific facility additions and modifications that need to be effected to render the plant functional in its intended service. These additions and modifications should be constructed with all due haste. Construction completion and successful operation will reduce pollutant loads to the receiving drains and Lake Maryout, and the effluent disinfection will benefit public health.

Design of the East plant upgrade is shown to begin in June 1981, with design completion in April 1982. Major equipment (pumps, air blowers, air diffusers, and some piping and electrical gear) would be prepurchased. Construction is anticipated to require between 18 and 19 months, concluding by the end of March 1984. Full operation could begin upon construction completion. The plant would treat flows from the No. 11 East pump station until either the Nouzha area wastewater conveyance system is operational or the Ras El-Soda plant and Sidi Bishr outfall are operable.

Conveyance System

The conveyance system design is shown to begin in July 1981 and be complete by the end of February 1983, allowing 19 or 20 months for design. Major equipment, to include pumps, motors, electrical switchgear and large diameter pipe, should be prepurchased as indicated. Construction is shown to begin in April 1983 and conclude in February 1986, for a total construction time of 34 to 35 months. Construction could begin at an earlier date with earlier than scheduled completion of design on one or more conveyance project elements. Construction completion of those conveyance system elements delivering wastewaters to the Ras El-Soda and East plants could also be delayed. The rationale behind later completion would be the late, June 1988, startup date on the Ras El-Soda plant and Sidi Bishr outfall. Until that plant and outfall are operational, wastewater influent to the East plant could continue to be from the existing No. 11 East pump station.

Any modification of the conveyance system construction schedule from that shown on Figure 4-5 should be preceded by a study of benefits to be derived by early completion of system components discharging to the Ras El-Soda and East plants. Temporary discharge of some Ras El-Soda area flows to the West collector or other local drains near the plantsite would be an improvement over the uncontrolled local discharges and overflows that would otherwise occur.

West Plant Expansion and Upgrade

Early construction of the West plant upgrade and expansion would provide some benefits by alleviating local sewage overflow and by eliminating dry weather discharges to the beach areas in the central and western zones. Once the West plant and its associated conveyance facilities are operational, chlorinated effluent discharge could be into Lake Maryout until the Kait Bey outfall is completed. Plant effluent could also be pumped to the existing Kait Bey outfall during the winter as a means of reducing organic loading to Lake Maryout.

The schedule shown on Figure 4-5 indicates design of needed West plant work commencing in July 1981, with design completion anticipated 19 months later in February 1983. Major equipment and critical long-delivery-time materials would be prepurchased during design for subsequent installation by the contractor(s). Construction is shown to require 36 months. This is approximately 150 percent as much time as would be required to construct the facilities in the United States. Plant startup and staff training would begin in February 1986 and be concluded in June of that year. Full plant operation would begin in April 1986.

Ras El-Soda Plant

The Ras El-Soda plant first-stage construction is scheduled to be completed at the time the Sidi Bishr outfall is completed. This allows considerable undesignated time in the Ras El-Soda schedule. Design and construction could be moved forward in time by 2 years should interim discharge of chlorinated plant effluent to the Weqf collector or other local drain system be deemed appropriate. Any substantial discharges to the Weqf or other local drain would, however, likely require expansion of the drain and effluent pumping capacities.

One substantial benefit in delaying the Ras El-Soda plant design and construction, as presented on Figure 4-5, would to spread out the total resource commitments for the West plant and Ras El-Soda plant design and construction. Design of the Ras El-Soda plant would follow that of the West plant, reducing the number of staff needed at any time. This should provide opportunities for design economy. Peak construction materials and labor needs on the Ras El-Soda plant would not occur until after the West plant labor and materials input reached a low level or ceased altogether. As a result, less expatriate labor would need to be employed during construction, and less stress would be placed on local materials supply.

The Ras El-Soda plant design would, by recommended schedule, begin in July 1983 and be completed 19 months later in February 1985. As with the West plant expansion and upgrade major equipment and critical materials would be prepurchased during design. First-stage construction time is expected to take 36 months. While this facility is larger than the West plant project, the plant will be totally new and a large area is available for construction staging. These considerations should make the 3-year, first-stage Ras El-Soda plant construction schedule feasible. Plant startup and staff training would begin in February 1988 and conclude in June 1988. Plant operation would be expected to begin in June 1988, after construction on the plant and the Sidi Bishr outfall is completed.

Opportunities exist for staged construction at the Ras El-Soda plant. The second and final stage of construction would need to be in place at the end of 1990 to avoid exceeding the design capacity of the primary sedimentation tank under projected 1991 design flows. The design, equipment prepurchase, construction and startup schedules for this second-stage project are depicted on Figure 4-5. Design is shown to require only 7 months. This design effort will be relatively minor, consisting of updating the original design for equipment, materials and construction detail changes, and preparation of prepurchase and construction tender documents.

Construction is shown to require only 18 months, an adequate amount of time given the construction involved.

Outfalls

Appendix L presents a detailed discussion of studies that need to be undertaken prior to outfall design. Also described are the outfall construction methods assumed as a basis of this report. The schedule shown on Figure 4-5 for outfall studies, design, and construction reflects the study needs and construction methods outlined in Appendix L.

The recommended outfall schedule shows design beginning in January 1982, only 6 months after the needed additional studies are started, and ending in May 1983, only 6 months after these additional studies are complete. This "fast track" approach is necessary because the outfalls are on the critical path of the implementation schedule for the sea system.

Outfall construction and testing is estimated at almost 5 years, beginning in July 1983 with completion in June 1988. Both outfalls would be constructed simultaneously. One pipe casting plant would be

established in Alexandria to service the two outfall projects. Actual outfall construction in the sea is expected to be practical only two shifts per day, for as few as 150 days per year due to visibility and wave conditions. For estimating purposes, construction during two shifts per day over 240 days per year was assumed. This constraint, together with the long outfall lengths involved, makes the 4-year, 11-month construction schedule necessary.

Since the Kait Bey outfall is shorter, its construction will likely be completed first. Upon completion, the West plant discharge could be directed through this outfall to the sea.

CAPITAL COST

The estimated capital costs to fully implement the sea disposal alternative are summarized in Table 4-1. Detailed capital costs for the individual components comprising the total system are presented in the following appendices:

- Conveyance facilities, Appendix L
- Treatment facilities, Apppendices E, F, and J
- Outfalls, Appendix L

The costs in Table 4-1 are presented in local (LE), foreign exchange ($) and total (LE + 0.7 x $) Egyptian pound categories. All costs are at end-of-1980 values.

Salvage values for all sea disposal system components at the end of year 2000 are presented in Table 4-1 as negative numbers. The salvage values are presented for the purpose of present worth calculations presented later in this chapter. Salvage values indicated are based on straight-line depreciation from full value (in end of 1980 LE) to zero value when the end of useful service life is reached. Expected service lives for the various system components are listed in Table 4-2.

OPERATION AND MAINTENANCE COSTS

Estimated annual operation costs, (end-of-1980 LE values) are detailed for conveyance facilities in Appendix M, treatment facilities in Appendix J, and outfalls in Appendix L. Costs are presented at 1980, 1990, and year 2000 flow rates in each of these appendices. Operation and maintenance (O&M) costs estimated for each principal system component during the time period from the initial part-year of operation

through 1989 are obtained by interpolation between 1980 and 1990. Likewise, annual costs for the years 1991 through 1999 are estimated by interpolation between 1990 and 2000. The sum of the individual system component O&M costs are presented in Tables 4-3 through 4-9. Also indicated in these tables are the quantities of consumables estimated to be used as related to the estimated O&M cost by category.

Annual O&M costs in Tables 4-3 through 4-9 are presented for years:

- 1984/85--the 2 years when only the East plant is operating, receiving plant capacity flows from the No. 11 East pump station (Table 4-3)
- 1986--the first partial year of conveyance facilities and West plant operation (Table 4-4)
- 1978--the first full year of conveyance facilities and West plant operation (Table 4-5)
- 1988--the first partial year of Ras El-Soda plant and outfall operation (Table 4-6)
- 1989--the first full year of Ras El-Soda plant and outfall operation (Table 4-7)
- 1990--the first year of 11 successive years through year 2000 in which annual O&M costs are expected to increase in a straight-line fashion (Table 4-8)
- 2000--the last year of the present worth analysis (Table 4-9)

Estimated annual O&M costs for the full sea disposal system in years 1991 through 1999 can be obtained by straight-line interpolation between the values shown in Tables 4-8 and 4-9.

TECHNICAL ASSESSMENT

This section addresses the sea disposal alternative as it pertains to the following parameters:

- Operability
- Maintainability
- Reliability

- Flexibility
- Expandability
- Potential for upgrading treatment
- Energy consumption

Evaluation of the sea disposal system by each parameter assumes complete system operation (conveyance, treatment, and disposal).

Operability

System operation should be relatively simple, with the following few exceptions:

- The East plant activated sludge operation would be relatively difficult and would require special attention and process control staffing.
- Mechanical sludge dewatering at the West and Ras El-Soda plants would also require special operational attention and staffing with experienced key operators.
- Chlorine feed facilities at all three plants would require some experienced key staffing.

The rationale behind these special operational considerations has been previously discussed.

Operability of all other system components, other than the outfalls, has been either directly or indirectly demonstrated in Alexandria. Outfall operation would be simple, requiring little operational attention.

Maintainability

The most difficult aspect of system maintainability will likely be in relation to the conveyance facilities after maintenance staffs for the plants and conveyance facilities have been trained.

A/GOSD personnel have become well acquainted with the difficulty of maintaining collection and conveyance systems. Future maintenance will be made less difficult if implementation and enforcement of the appropriate industrial waste pretreatment requirements and ordinances is accomplished. This, together with exclusion of incompatible wastes and

improved facilities design, will result in fewer problems with pump and pipe stoppages, equipment and materials degradation, and pipeline breaks.

Proper maintenance of pump stations and treatment plants will largely depend upon the acquisition and training of key staff. This will be most critical in the area of electrical and control specialists. Adequate spare part stocks will be necessary to complement the ability of qualified maintenance staffs in keeping all mechanical, electrical, and control systems online.

Annual inspections of the outfall and periodic outfall maintenance would be subcontracted to marine specialists. These inspection and maintenance requirements are detailed in Appendix L.

Reliability

Physical reliability of the sea disposal alternative would be expected to be quite good, provided conditions and concerns presented under the previous sections concerning operability and maintainability are satisfactorily addressed.

Failure of individual items of machinery at large treatment plants is quite probable. For this reason, the anticipated design incorporates redundant units for all critical elements and a dual power supply system. Specific design criteria related to reliability are discussed in detail in Appendix J. The sea portion of the two outfalls does represent one area where redundant facilities are simply not feasible. Here reliance must be placed on the best available design and construction practices rather than on providing redundancy. Well designed and constructed outfalls have proven to be remarkably trouble-free throughout the world and this should also be true for Alexandria. Conceivably, a major undersea break could occur due to national catastrophe, major maritime accident, or sabotage. In such a case, A/GOSD could not be expected to possess the capability to effect repairs using their own forces and equipment. Rather, a contractor experienced in outfall construction would be required. This would result in unavoidable delay during which the only possible action on the part of A/GOSD would be to increase operating vigilance and provide increased levels of disinfection. Depending on the time of year, type and location of failure, and other conditions, these actions may very well be sufficient to avoid potential beach contamination.

The certainty of meeting pollution control and public health objectives could be compromised by lack of success in excluding or pretreating or incompatible toxic wastes. If toxic wastes continue to be discharged to

the sewers, the toxic substances would become concentrated in aquatic sea life and returned to Alexandrian and other Egyptian residents through the food chain. Primary treatment does little to remove toxic materials, and chlorine disinfection can actually enhance toxicity.

Successful operation of the East plant will be dependent upon elimination of most toxic discharges to its influent flow. Failure to adequately pretreat or remove these wastes could render the activated sludge facilities inoperable and primary effluent would be discharged.

Flexibility

Flexibility in operation of the sea disposal system would be minimal. The one exception is the flexibility to use the outfalls for discharge of stormwater overflows as well as primary effluent. The outfalls would have significant excess capacity during early years of operation to enable this practice. On the negative side, the outfalls represent a significant investment and will have no flexibility in regard to alternative points of discharge or effluent reuse without a loss of investment.

Inflexibility is a characteristic of primary treatment. Any subsequent decision to productively reuse the wastewater, as in the land application alternative, could include continued primary plant operation and the consequent elimination of pond treatment at a land application site. Such a decision would, however, be burdened by considerable additional costs. The anaerobic pond system would have much lower capital and O&M costs than would the mechanical primary treatment facilities with their associated sludge management systems.

Secondary treatment facilities at the East plant will provide for some operational flexibility. Proximity of the discharge to potential industrial and agricultural water users should provide for flexibility in future effluent use once the ability to obtain a high quality effluent has been demonstrated.

Expandability

The wastewater conveyance system components (Appendix M) are configured to accommodate flows through year 2030. Capacity to convey flows greater than those projected through year 2000 would only require additional pumps and associated mechanical and electrical work at pumping stations and some parallel pipelines where two-stage construction for required flows was determined cost-effective. The year 2030 service area population is expected to be at or very near the saturation level.

Therefore, the conveyance facilities are given a very high expandability rating.

The three treatment plants would be designed to handle year 2000 flows. Expansion of the plants to accommodate flows at service area saturation would be possible given the following conditions:

- Plants are expanded by like or less land consumptive processes.
- No effluent upgrade, other than that provided by "enhanced primary treatment" (e.g., adding chemicals to the wastewater before the primary sedimentation process), is included in the expansions.
- The entire 41 ha is acquired for the Ras El-Soda plant site
- Sludge compost operations at the West plant are moved offsite.
- Mechanical sludge dewatering of a portion of the East plant sludge is provided onsite, with the dewatered sludge being composted at one of the offsite compost facilities.

The outfalls should be designed to accommodate year 2030 peak flows.

Potential for Upgrading Treatment

Sea discharge could conceivably require secondary treatment or "enhanced primary treatment" in the future. Interim permission to discharge wastewaters to the Pacific Ocean off the west coast of the United States is being granted on the condition that enhanced primary treatment be afforded. This generally involves chemical (polymer, lime, alum or ferric chloride) addition ahead of primary sedimentation. The chemically coagulated and flocculated wastewater is then settled in conventional primary sedimentation tanks. The chemical addition results in significantly increased toxic removals and some decrease in effluent BOD and suspended solid concentrations. Sludge production is increased by virtue of the chemical addition and increased solids removal efficiency.

Current United States practice is to permit ocean discharge only after enhanced primary treatment in locations where it is presently believed no significant environmental problems will result. Otherwise, secondary treatment will be required in the future.

Conversion of the West and Ras El-Soda facilities to enhanced primary plants could be accommodated on the proposed sites. Expansion of

enhanced primary treatment facilities to serve saturated service area flows would be possible if the constraints previously mentioned are appropriately addressed.

Both the West and Ras El-Soda plants could also be upgraded to provide biological tower secondary treatment to year 2000 flows as indicated in Appendix J. Offsite composting of West plant sludge would be required. The entire 41-ha Ras El-Soda site would be occupied by treatment and ancillary works and would not accommodate secondary treatment of saturated service area flows. The West plant site could be expanded into Lake Maryout to accommodate secondary treatment of these maximum future flows.

Energy Consumption

Estimated annual consumption of electricity (kWh) and liquid fuel (litres) is presented in Tables 4-3 through 4-9. Total estimated annual energy consumption in each of the years 1984 through 2000 is displayed in millions of kilocalories (kcal x 10^6) in Table 4-10. Energy consumption is shown in three categories: (1) fuel, (2) electrical power directly used, and (3) electrical power indirectly used in production of chlorine for disinfection. Power for chlorine manufacture is included in Table 4-10 since it would constitute a significant portion of the energy commitment made to system operation.

The conversion of electrical consumption (kWh) to kcal x 10^9 assumes the power will be produced by generators fired by fossil fuels operating at an efficiency of 32 percent electrical energy produced from the fuel energy consumed. This assumes Alexandria's wastewater conveyance treatment, and disposal projects will require new fossil-fueled power generators because Egypt's low cost hydropower is, or will soon be, committed to meet other necessary or productive demands.

PRESENT WORTH ANALYSIS

The single most important consideration in evaluating any proposed wastewater management system is generally the system's impact on financial resources. The present worth technique is useful in this evaluation and is used here.

Table 4-11 displays end-of-1980 present worth costs, in end-of-1980 pounds, by principal system component. Separate present worth amounts are presented for estimated capital, and operation and maintenance (O&M) costs to be expended through year 2000. A 6-percent discount rate was used. The present worth of each principal component's year 2000 salvage value is shown as a negative number in Table 4-11.

All prices used in calculating O&M present worth amounts are end-of-1980 Egyptian market prices. O&M costs and their present worths amounts would likely be higher if price controls and subsidies did not impact the labor, energy, and consumables markets.

Table 4-1
SUMMARY OF ESTIMATED CAPITAL COSTS
SEA DISPOSAL ALTERNATIVE 1B
(costs in end-of-1980 prices)

System Component	Midpoint of Capital Expenditure[a]	Local Cost (10^6 LE)	Foreign Exchange Cost	Total Capital Cost[b] (10^6 LE)	End of Year 2000 Salvage Value (10^6 LE)
East Plant Upgrade	3/26/83	6.7		16.1	9.6[c]
Conveyance System	7/1/84	95.5	6[illegible].4	162.3	103.2
West Plant Expansion & Upgrade	7/31/84	27.5	49.8	62.4	33.4[c]
Ras El-Soda Plant, 1st Stage	7/31/86	35.4[d]	60.0	77.4[d]	39.1[d]
Ras El-Soda Plant, 2nd Stage	3/14/90	4.4	9.3	10.9	7.8
Outfalls	10/1/85	108.9	45.0	140.4	105.2
Total	---	278.4	272.9	469.5	298.3

[a] For midpoint of capital expenditures see Figure 4-5

[b] Includes local costs (LE) plus foreign exchange costs at $1.00 US = LE 0.7

[c] Includes a salvage value of LE 8 million to account for the salvage value of existing construction and LE 1.6 million to account for new construction envisioned under Alternative 1B.

[d] Includes land cost at LE 600,000.

Table 4-2
SERVICE LIFE PARAMETERS
SEA DISPOSAL ALTERNATIVE 1B

Type of Component	Service Life
Land	Permanent
Wastewater conveyance and outfall structures (including outfall pipes, interceptors, force mains, and tunnels)	50 years
Other structures (including plant buildings, concrete process tankage, basins, lift station structures)	40 years
Process equipment (including major process equipment such as sedimentation tank mechanisms, vacuum filters; steel process tanks and chemical storage facilities)	15 years
Auxiliary equipment (including instruments and control facilities; sewage pumps, and electrical motors; mechanical equipment such as compressors, aeration systems, and chlorinators; power generating facilities)	15 years

Table 4-3
SUMMARY OF ESTIMATED 1984 AND 1985 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE 1B
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	0	0	0	0	0	0	0	0	0	0
West Plant	0	0	0	0	0	0	0	0	0	0
Ras El-Soda Plant	0	0	0	0	0	0	0	0	0	0
Outfalls	0	0	0	0	0	0	0	0	0	0
Total	11.12	72	29	256	12	1	93	35	506	870

Table 4-4
SUMMARY OF ESTIMATED 1986 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE 1B
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	11.08	72	15	134	0	0	0	0	204	410
West Plant	22.05	143	9	80	148	5	1,231	1,153	188	1,569
Ras El-Soda Plant	0	0	0	0	0	0	0	0	0	0
Outfalls	0	0	0	0	0	0	0	0	0	0
Total	44.25	287	53	470	160	6	1,324	1,188	898	2,849

Table 4-5
SUMMARY OF ESTIMATED 1987 ANNUAL O&M COSTS
SEA DISPOSL ALTERNATIVE IB
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	15.83	103	22	227	0	0	0	0	291	621
West Plant	31.94	207	13	116	223	8	1,959	1,834	273	2,438
Ras El-Soda Plant	0	0	0	0	0	0	0	0	0	0
Outfalls	0	0	0	0	0	0	0	0	0	0
Total	58.89	382	64	599	235	9	2,052	1,869	1,070	3,929

Table 4-6
SUMMARY OF ESTIMATED 1988 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE IB
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	41.30	268	34	357	0	0	0	0	514	1,139
West Plant	32.38	210	13	119	227	8	1,997	1,870	277	2,484
Ras El-Soda Plant	20.56	134	7	64	319	11	1,252	1,134	192	1,535
Outfalls	0.09	9	0	0	0	0	0	0	79	88
Total	105.45	693	83	796	558	20	3,342	3,039	1,568	6,116

Table 4-7
SUMMARY OF ESTIMATED 1989 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE 1B
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	63.00	410	45	530	0	0	0	0	704	1,644
West Plant	33.82	220	14	122	232	8	2,036	1,906	281	2,537
Ras El-Soda Plant	40.13	261	14	124	627	22	2,458	2,227	269	2,903
Outfalls	0.16	16	0	0	0	0	0	0	146	162
Total	148.23	979	102	1,032	871	31	4,587	4,168	1,906	8,116

Table 4-8
SUMMARY OF ESTIMATED 1990 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE 1B
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	63.00	410	47	549	0	0	0	0	704	1,663
West Plant	33.26	217	14	125	236	8	2,074	1,942	285	2,577
Ras El-Soda Plant	42.19	275	15	126	663	24	2,597	2,353	383	3,161
Outfalls	.16	16	0	0	0	0	0	0	146	162
Total	149.73	990	105	1,056	911	33	4,764	4,330	2,024	8,433

100

Table 4-9
SUMMARY OF ESTIMATED 2000 ANNUAL O&M COSTS
SEA DISPOSAL ALTERNATIVE 1B
(10^3 LE, end-of-1980 prices)

System Component	Labor		Power		Fuel		Chemicals		Materials	Total
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost	Tonnes	Cost	Cost	Cost
East Plant Upgrade	11.12	72	29	256	12	1	93	35	506	870
Conveyance System	63.00	410	59	620	0	0	0	0	704	1,734
West Plant	35.36	230	16	146	280	10	2,458	2,285	320	2,991
Ras El-Soda Plant	50.36	327	21	178	917	33	3,527	3,230	467	4,235
Outfalls	.16	16	0	0	0	0	0	0	146	162
Total	160.00	1,055	125	1,200	1,209	44	6,078	5,550	2,143	9,992

Table 4-10
SUMMARY OF ENERGY REQUIREMENTS
SEA DISPOSAL ALTERNATIVE 1B
(kcal x 10^9)

Year	Liquid Fuel	Electricity[a]	Chlorine Manufacture[a,b]	Total
1984	-	77	1	78
1985	-	77	1	78
1986	1	141	15	157
1987	2	170	23	195
1988	4	223	38	265
1989	7	273	49	329
1990	7	280	50	337
1991	7	286	52	345
1992	7	291	54	352
1993	8	296	55	359
1994	8	301	56	365
1995	8	306	58	372
1996	8	312	60	380
1997	8	317	62	387
1998	9	322	63	394
1999	9	328	65	402
2000	9	333	66	408
Total	102	4,333	768	5,203

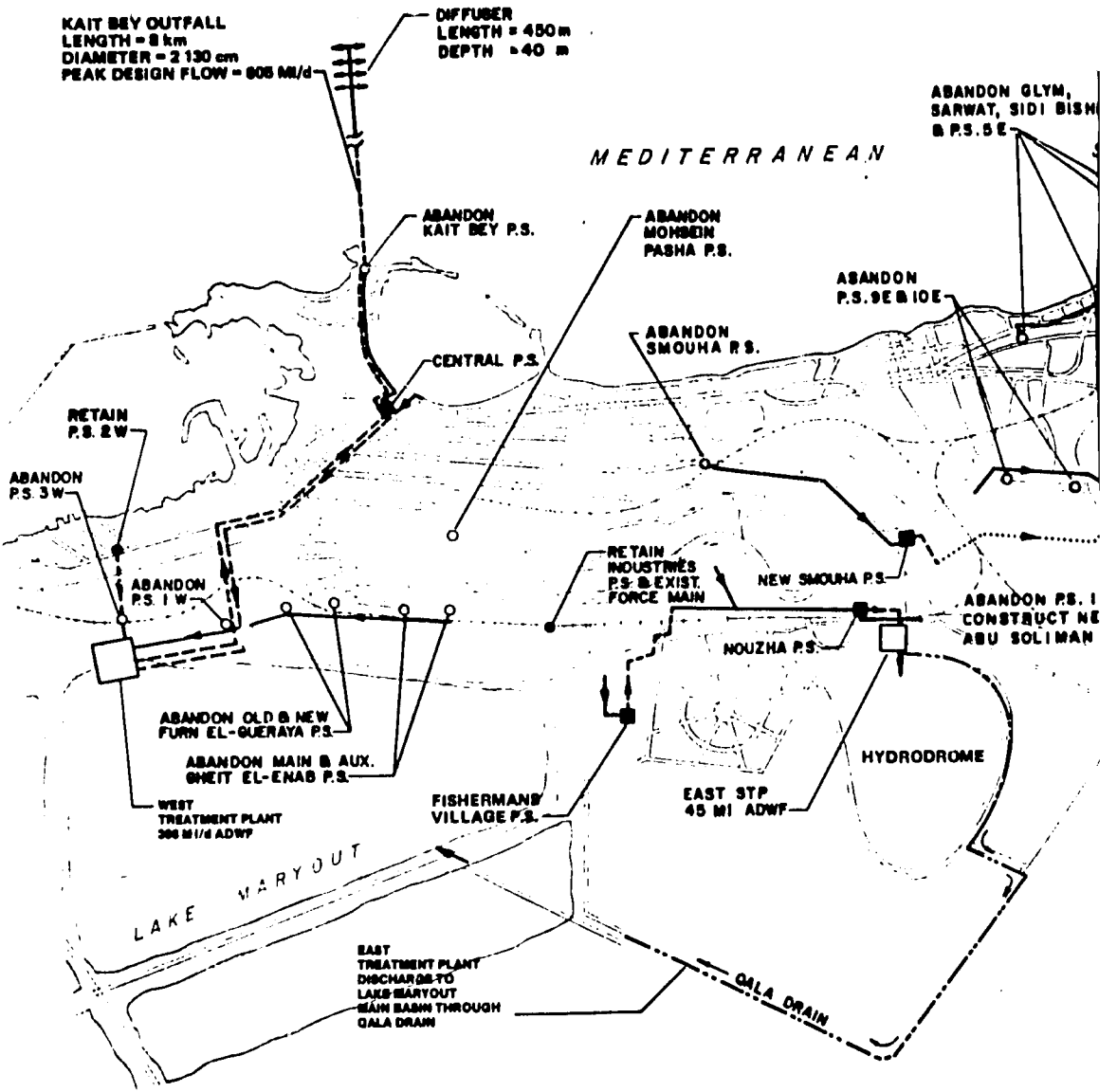
[a]Assumes power is produced by fossil-fueled generators operating at fuel-to-electricity conversion efficiencies of 32 percent.

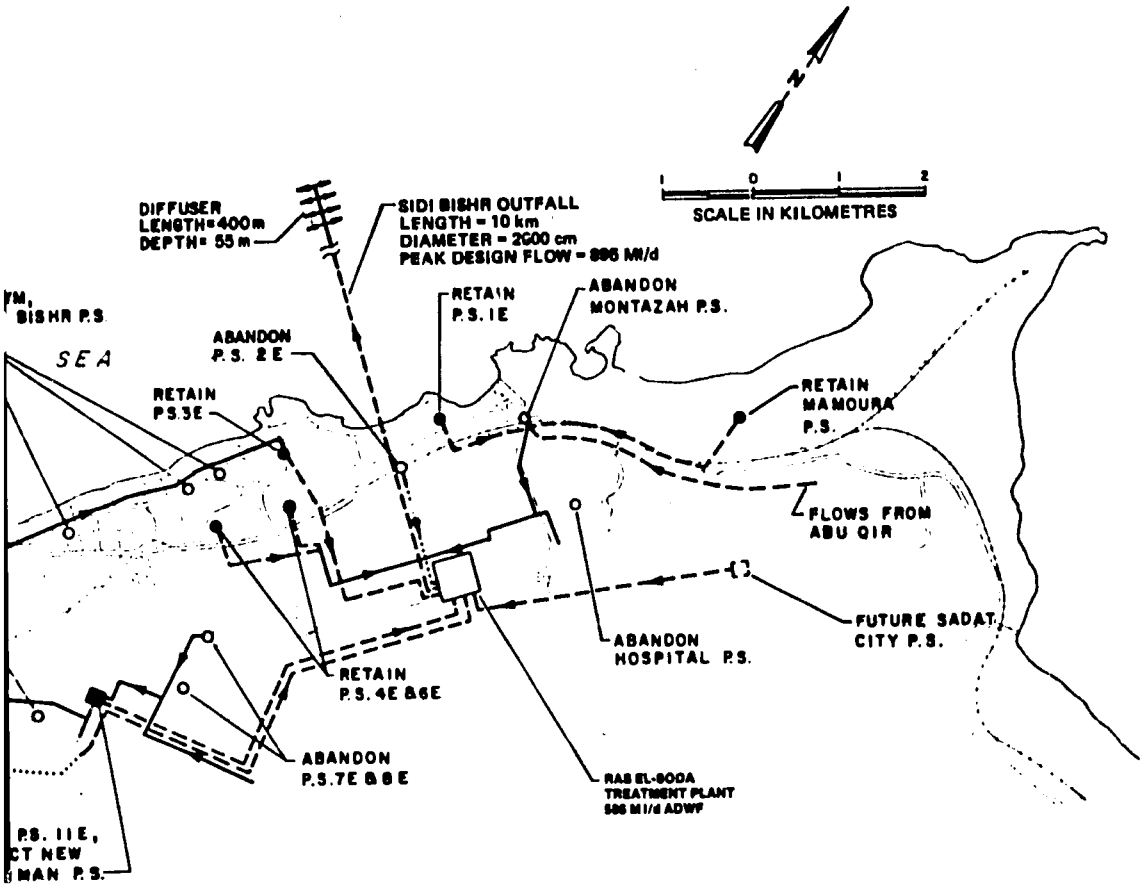
[b]Electricity to produce chlorine used for effluent disinfection is included as an indirect commitment of resources. Assumes 50 percent of the power used in producing chlorine and caustic in a diaphragm or membrane cell plant is assigned to production of chlorine (1500 kWh/tonne).

Table 4-11
ESTIMATED PRESENT WORTH
SEA DISPOSAL ALTERNATIVE IB
(LE, end-of-1980 prices)

System Component	Present Worth (10^6 LE)[a] Capital		O&M		Salvage Value		Total
East Plant	14.1	+	7.9	-	3.0	=	19.0
Conveyance System	132.4	+	10.6	-	32.2	=	110.8
West Plant	50.7	+	19.2	-	10.4	=	59.5
Ras El-Soda Plant[b]	62.3	+	20.2	-	14.6	=	67.9
Outfalls	106.4	+	1.0	-	32.8	=	74.6
Total	365.9	+	58.9	-	93.0	=	331.8

[a]Discount rate of 6 percent used
[b]Includes Stage 1 and Stage 2 construction, capital, and salvage values.


KAIT BEY OUTFALL
LENGTH = 8 km
DIAMETER = 2 130 cm
PEAK DESIGN FLOW = 605 Ml/d
DIFFUSER
LENGTH = 450 m
DEPTH = 40 m
MEDITERRANEAN
ABANDON GLYM, SARWAT, SIDI BISH & P.S. 5 E
ABANDON KAIT BEY P.S.
ABANDON MOHSEIN PASHA P.S.
ABANDON P.S. 9E & 10E
ABANDON SMOUHA P.S.
CENTRAL P.S.
RETAIN P.S. 2W
ABANDON P.S. 3W
ABANDON P.S. 1 W
RETAIN INDUSTRIES P.S. & EXIST. FORCE MAIN
NEW SMOUHA P.S.
ABANDON P.S. 1 CONSTRUCT NE ABU SOLIMAN
NOUZHA P.S.
ABANDON OLD & NEW FURN EL-GUERAYA P.S.
ABANDON MAIN & AUX. GHEIT EL-ENAB P.S.
HYDRODROME
WEST TREATMENT PLANT 300 Ml/d ADWF
FISHERMANS VILLAGE P.S.
EAST STP 45 Ml ADWF
LAKE MARYOUT
EAST TREATMENT PLANT DISCHARGE TO LAKE MARYOUT MAIN BASIN THROUGH GALA DRAIN
GALA DRAIN



NOTE:
SLUDGE COMPOST SITES NOT SHOWN. NOTATION ABANDON P.S. INDICATES ABANDONMENT FOR SEWAGE. SEE APPENDIX G FOR DESCRIPTION OF P.S. TO BE RETAINED FOR STORM-WATER PUMPING.

LEGEND

—·—·—	DRAIN
———	GRAVITY SEWER
— — —	PRESSURE SEWER
·····	EXISTING SEWERS
ADWF	AVERAGE DRY WEATHER FLOW
STP	SEWAGE TREATMENT PLANT

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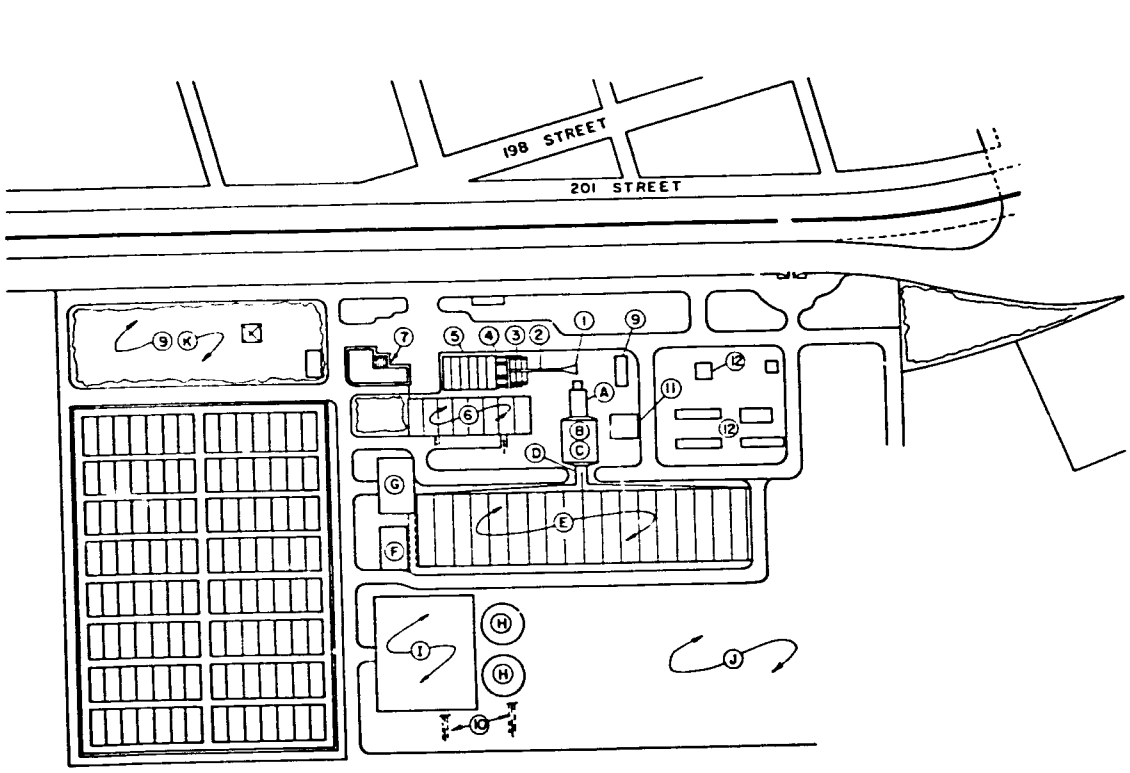
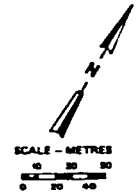
FIGURE 4-1

ALTERNATIVE 1B FACILITIES





UNDER CONSTRUCTION

1. INLET WORKS
2. VENTURI FLUME
3. AERATED GRIT BASINS
4. COMMINUTOR BASINS
5. PRE-AERATION BASINS
6. PRIMARY SEDIMENTATION
7. BLOWER BUILDING
8. SLUDGE DRYING BEDS
9. SLUDGE STORAGE
10. EFFLUENT CHANNELS
11. ADMINISTRATION AND LABORATORY BUILDING
12. HOUSING FOR STAFF

PROPOSED ADDITIONS

A BAR SCREENS
B AERATED GRIT BASINS
C FLOTATION BASINS
D PARSHALL FLUMES
E PRIMARY SEDIMENTATION
F CHLORINE BUILDING
G EFFLUENT PUMP STATION
H LIQUID SLUDGE STORAGE
I VACUUM FILTERS
J COMPOST AREA
K COMPOST STORAGE

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FIGURE 4-2

WEST TREATMENT PLANT SITE PLAN
ALTERNATIVE 1B

A/GOSD WWCG W/ECG

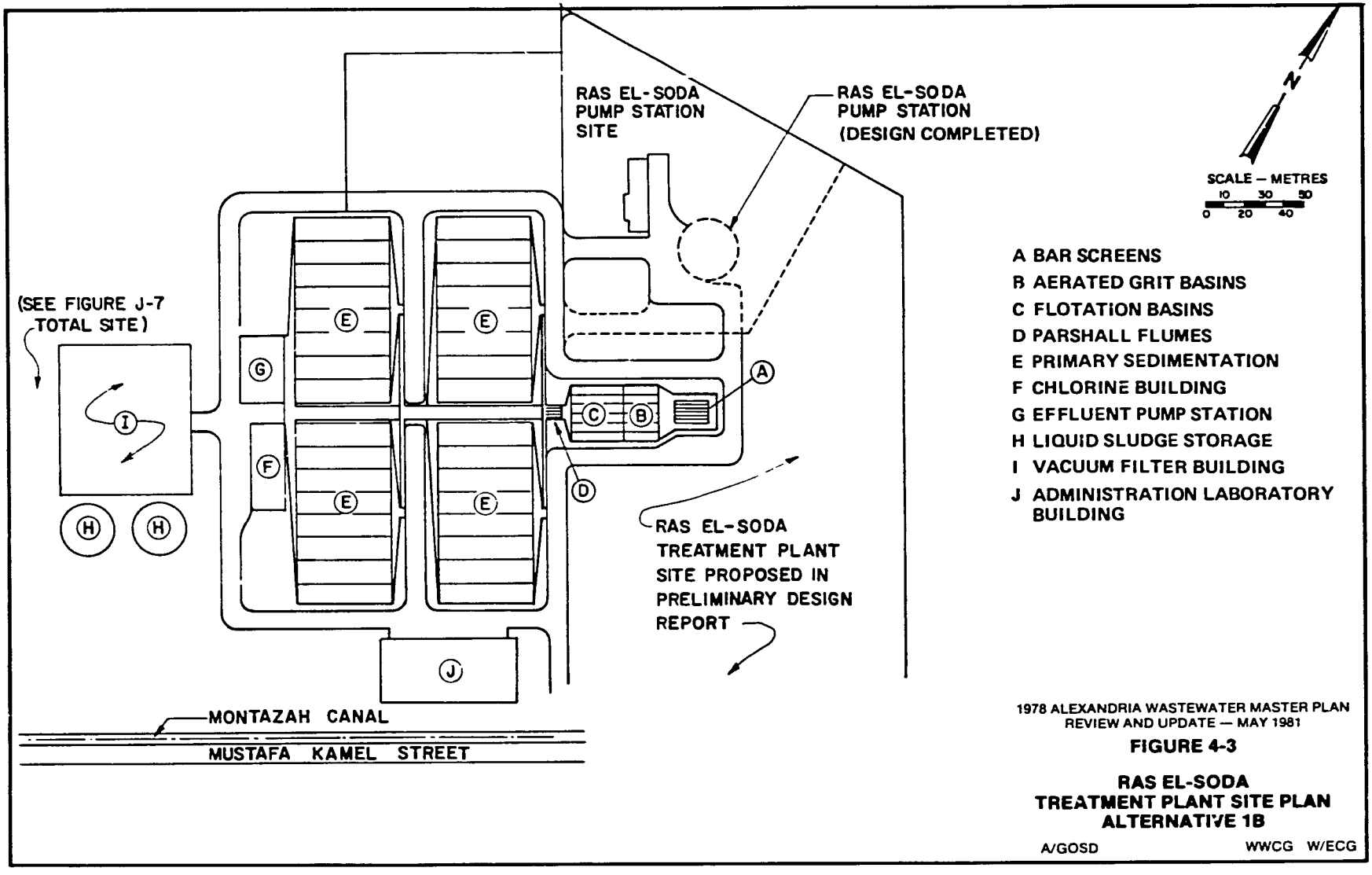

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FIGURE 4-3

RAS EL-SODA TREATMENT PLANT SITE PLAN ALTERNATIVE 1B

A/GOSD WWCG W/ECG

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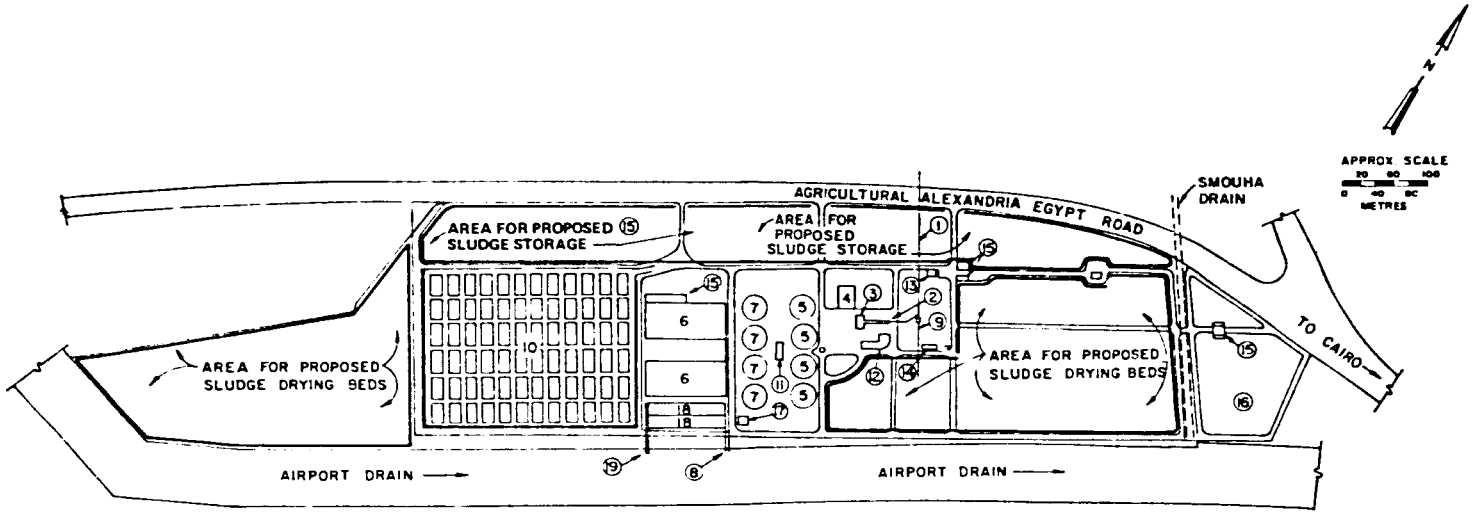

LEGEND:

1. INFLUENT FORCE MAIN
2. INLET WORKS
3. AERATED GRIT BASIN
4. PRE-AERATION BASINS
5. PRIMARY SEDIMENTATION
6. AERATION BASINS
7. SECONDARY SEDIMENTATION
8. EFFLUENT LINE, EXISTING
9. BYPASS TRAIN
10. SLUDGE DRAIN BEDS
11. PUMP HOUSE
12. MACHINERY BUILDING
13. LABORATORY AND ADMINISTRATION BUILDING
14. INDUSTRIAL LAB
15. STORAGE BUILDING
16. HOUSING
17. CHLORINE BUILDING, PROPOSED
18. CHLORINE CONTACT BASIN, PROPOSED
19. EFFLUENT LINE, PROPOSED

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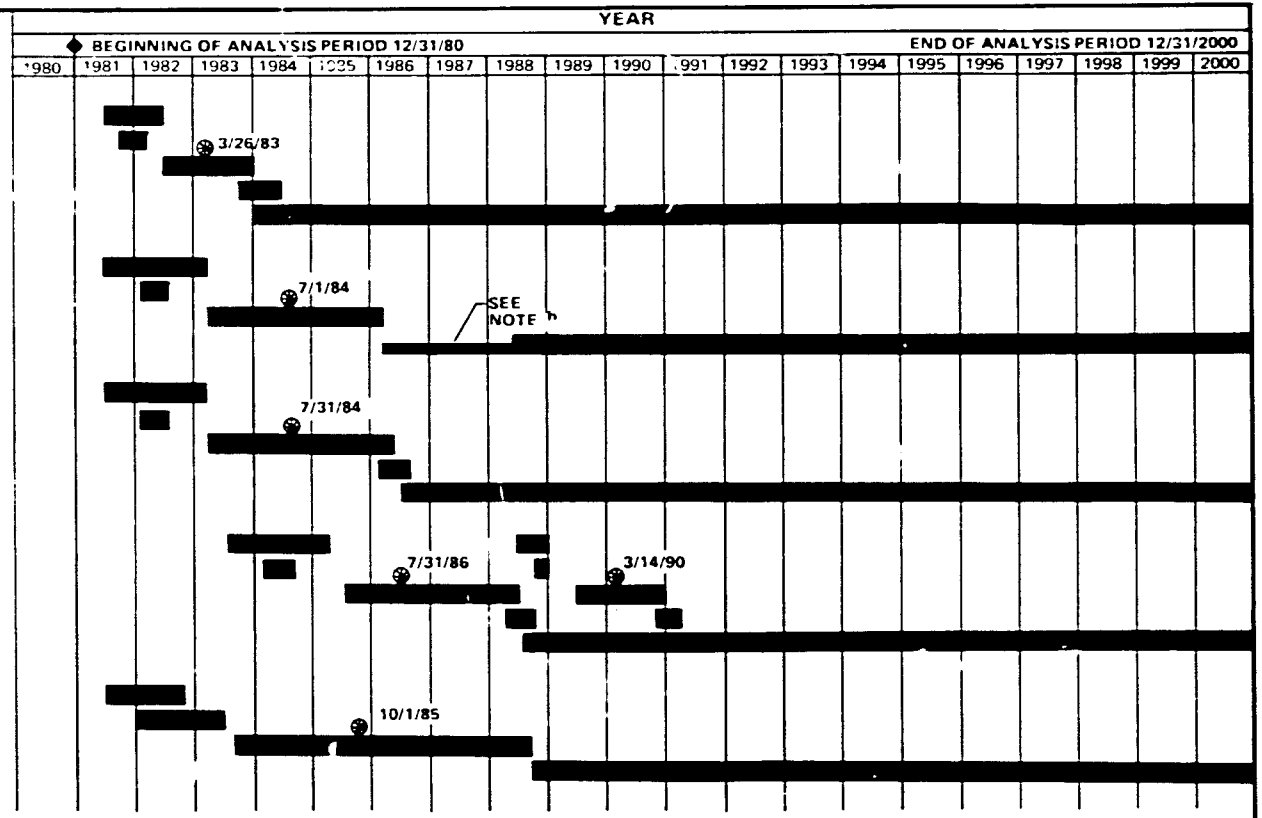
FIGURE 4-4

EAST TREATMENT PLANT SITE PLAN
ALTERNATIVE 1B

A/GOSD WWCG W/ECG



[a] Includes basis of design reports.

[b] That portion of the conveyance system serving the Ras El-Soda plant will not be placed in full operation until the Sidi Bishr outfall and Ras El-Soda plant are operational.

[c] Design and construction of Ras El-Soda plant delayed so finish date of plant and outfall are same.

LEGEND

◆ Point in time to which all capital expenditures are indexed and present worth established

● Midpoint of capital expenditure for phase of system component (design plus purchase plus construct plus startup).

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FIGURE 4-5

IMPLEMENTATION SCHEDULE FOR SEA DISPOSAL ALTERNATIVE 1B

A/GOSD WWCG W/ECG

CHAPTER 5

LAND APPLICATION ALTERNATIVES

In the 1978 Master Plan, land application of the wastewater was considered and found not to be feasible when compared with the sea outfall alternative. Since that time, however, some of the basic assumptions and interpretations used in that analysis have changed. As a consequence, one of the main objectives of the Master Plan Review was to reexamine the possibility of land application for effective treatment in combination with beneficial reuse of the water and nutrients.

As a result of a reconnaissance investigation, two viable land application alternatives were developed. These alternatives have been designated as IIIA and IIIB and are presented in detail in Appendix K. The related conveyance systems (discussed in Appendix M) and treatment systems (discussed in Appendix J) are combined in this chapter with land application systems to form Alternatives IIIA and IIIB.

As in Chapter 4, wastewater collection costs and impacts are not considered because they are common to all the alternatives.

DESCRIPTION OF ALTERNATIVES

Alternatives IIIA and IIIB consist of conveying wastewater to the desert, providing pretreatment, and using it to irrigate farm crops. These major project features are shown on Figures 5-1 and 5-2. Once the East and West treatment plants are rehabilitated and/or completed, they will be used to treat wastewater flows up to their full capacities.

The difference between Alternatives IIIA and IIIB lies in the level of pretreatment and the cropping pattern. The conveyance system and the use of the East and West plants would be the same in each alternative. Alternative IIIA provides a higher degree of pretreatment than IIIB. This would allow the wastewater to be used on high value crops. Alternative IIIB provides only primary level pretreatment; consequently only forage, fiber, and seed crops could be irrigated with the effluent. High water- and nutrient-using forage crops are proposed; this significantly reduces land requirements and capital costs.

As described in Appendix K, suitable irrigation sites were investigated throughout the greater Alexandria area. The site finally selected for the land application alternatives is located as shown on Figures 5-1 and 5-2 and is identified as the East Site.

Alternative IIIA

Conveyance System. The major facilities in the conveyance system are the East, West, and Desert pump stations and their related force mains. The remaining in-city facilities would be generally the same as for Alternative IB and are discussed in detail in Appendix M.

The East pump station would have a design capacity of 895 Ml/day at a total dynamic head of 92 m. It would require four 4000-hp pumps plus two redundant pumps. All pumps would have variable speeds.

The West pump station's design flow of 650 Ml/day would require three 3500-hp pumps with two additional pumps in reserve. The Desert pump station would include flow equalization basins to reduce pipe and pump sizes. It would be designed to pump 1255 Ml/day, against a 92-m head, using six 4000-hp pumps with two additional pumps in reserve. The flow equalization would be provided in circular concrete tanks provided with conical bottoms. The wastewater would enter and exit the tanks through drains in the bottoms. Pumps would divert flows in excess of 1255 Ml/day from the Desert pump station inlet to the tanks. Discharge from the tanks would be by gravity through flow control gates back to the pump station inlet, maintaining pump station flows at or near 1255 Ml/day until the equalization tankage is empty. The equalization tank contents would be mixed by diffused air to minimize (1) sedimentation in the tanks and associated cleaning requirements and (2) odor release. Final design may not preclude release of unacceptable odors. For that reason the design would provide for subsequent addition of tank covers and odor scrubbing of exhaust air when and if needed.

The force mains from the East pump station to the Desert pump station would consist of two pipelines 1750 mm in diameter and 38 km long. The force mains from the West pump station would consist of two pipelines 1450 mm in diameter and 33 km long. Three pipelines 1850 mm in diameter would convey the wastewater the final 44 km from the Desert pump station to the land application site.

Land Application System. Alternative IIIA is a system which would consist of pretreatment by anaerobic lagoons, followed by rapid infiltration for additional high-level treatment and ground water storage. The soil and ground water conditions at the East Site are such that all of the excess wastewater flows can be stored in the aquifer during the winter. The stored water would then be recovered for use during the peak irrigation season in the summer. Except for dissolved minerals, the recovered ground water would be of high quality and could be used for irrigation of most crops, including high value crops that would result in significant economic returns.

Based on a conceptual design for the year 2000 wastewater flows, major features of the system would include:

- 210 ha of anaerobic lagoons with an average detention time (in service) of 3 days
- Production and agricultural reuse of 254 million kg/year (dry weight basis) of dewatered sludge
- 1540 ha of rapid infiltration basins, loaded at a rate of 27 m/year
- Storage of 19 percent of the annual wastewater volume in the ground water system beneath the site
- 223 ground water recovery wells
- A combination of concrete-lined canals and pressure pipelines for distribution of the treated effluent to the irrigation system
- 444 center pivot irrigation machines
- 23,000 ha (54,160 fd) of cultivated land, irrigated at an average rate of 1.5 m/year
- Total land area of 31,000 ha (73,000 fd)
- Administration and support facilities located on the site

East Treatment Plant. The East plant could be used in the same way as proposed for the sea disposal alternative. That is, it could be upgraded to an activated sludge secondary treatment plant with a capacity of 45 Ml/day. Chlorinated effluent would then be discharged to the Qala Drain.

West Treatment Plant. A phased approach is recommended to make use of the West plant. Construction would first be completed for primary treatment of 85 Ml/day of domestic wastewater for discharge to Lake Maryout. This would provide some interim relief to the lake. Additional sludge drying bed area would be provided over that in the original design.

When the desert land application system is completed, the domestic wastewater flow would be diverted to the land application system. The West plant could then be modified for industrial wastewater pretreatment. The pretreated industrial flows could then also be safely sent to

the land application system. Since the plant, as modified for industrial pretreatment, would not be a part of the domestic sewerage system per se, the cost of operating the plant (after irrigation commences) has not been considered in the cost analysis. Also, an estimated LE 16.7 million salvage value of the West plant has been credited to domestic wastewater treatment.

Alternative IIIB

The conveyance system and East and West Treatment Plants would be the same as described for Alternative IIIA. The only differences would be in the design and operation of the land application systems.

In Alternative IIIB, irrigation would follow primary treatment in anaerobic ponds. Although significantly less capital-intensive than Alternative IIIA, it would also provide fewer benefits and less revenues to offset the annual cost of the system. Pretreatment would again be by anaerobic lagoons but with an additional emergency storage capacity. No seasonal storage would be provided, however, so crops would be overirrigated in the winter and some land taken out of production in the summer. With only primary treatment, the selection of crops would be limited to forage and other non-food crops for public health reasons.

Based on a conceptual design for the year 2000 wastewater flows, the major features of this land application system would include:

- 520 ha of anaerobic treatment/storage lagoons with an average detention time of 8.75 days
- 5 days' emergency storage capacity
- Production and agricultural reuse of 340 million kg/yr (dry weight basis) of dewatered sludge
- A combination of concrete-lined canals and pressure pipelines for distribution of the effluent to the irrigation system
- Screening of the wastewater before irrigation
- 276 center pivot irrigation machines
- 14,700 ha (34,240 fd) of cultivated land, irrigated at an average rate of 2.4 m/year
- Total land area of 18,500 ha (44,500 fd)

- Administration and support facilities located on the site

ANALYSIS OF ALTERNATIVES

The land application alternatives are analyzed in the following sections for techrical feasibility, environmental and social concerns, constructibility, scheduling and staging options, and costs.

Technical Feasibility

All of the elements of the land application alternatives, taken separately and in combination, are feasible. The conveyance system consists of several pipelines and pump stations. The size and quantity of the facilities may be relatively large, especially the Desert pump station and force mains, but only conventional and well-proven design, construction, and operation practices would be required.

Anaerobic lagoons, which are proposed for both alternatives, are simple, inexpensive, and reliable for primary level treatment of high-strength wastes. This method will satisfy the treatment requirements for the subsequent rapid infiltration and/or irrigation systems.

The rapid infiltration system of Alternative IIIA is also a well-proven technology, which is capable of providing very high levels of treatment if properly employed. It is a method of treatment that has received an increasing amount of interest in the United States and has been the object of numerous research and observation projects in recent years. Based on these studies, design and operation guidelines have been established which can help ensure reliable performance. The soil and ground water conditions of the East Site appear to be very favorable for rapid infiltration and for storing water in the aquifer. However, detailed geologic and ground water studies will be required to optimize the design for basin layout and effluent recovery.

The effluent recovered from the rapid infiltration system will be suitable for irrigation of most crops except those that are highly salt-sensitive. Pathogens in the raw wastewater would be almost completely removed by the rapid infiltration process, so public health concerns should not restrict crop selection. In fact, wells located down gradient from rapid infiltration systems in the United States are commonly and safely used for domestic water supplies.

Alternative IIIB does not include the high level of treatment and the seasonal storage provided by the rapid infiltration system of Alternative IIIA. It utilizes primary effluent for irrigation of non-

food crops with safeguards that are consistent with standards currently employed by the World Health Organization and various agencies in the United States. The concept of overirrigating the forage crops in the winter months is also quite feasible if water-tolerant varieties are selected and the soils are well drained (which is the case over most of the selected site).

Environmental Impacts

Both land application alternatives provide for acceptable wastewater treatment and disposal with overall net benefits to the public health of urban Alexandria. In addition, each alternative conserves water and nutrients for agricultural production. This is an important consideration in a rapidly growing country where less than 3 percent of the total land is arable. Thus, the large land area requirements of each alternative are not necessarily an adverse impact; rather, the proposed land use is both compatible with nearby uses and even desirable. Summarized below are the significant environmental impacts, both beneficial and adverse of each land application alternative. Mitigation measures that may be necessary to minimize some of the adverse impacts are discussed. This summary is based on the environmental impact sections presented in the appendices dealing with Treatment Alternatives (J), Land Application of Wastewater (K), and Conveyance System Alternatives (M).

Alternative IIIA. The beneficial environmental impacts associated with this alternative include:

- Improving the public health of Alexandria's residents area due to the provision of an adequate sewerage system
- Reusing wastewaters and sludge to increase agricultural production
- Using otherwise undevelopable land to produce high value crops
- Encouraging the eventual development of rural decentralized communities
- Providing employment during construction and operation
- Reducing the consumption of fertilizers for crop production

Potential adverse impacts include:

- A localized risk, which exists with any irrigation system, that parasitic diseases, such as schistosomiasis, could become endemic at the land application site

- Increased ground water flow from the land application site towards Wadi Natroun
- Possible contamination of downstream water supplies if the desert conveyance pipeline fails near major canal crossings
- Energy requirements for pumping flows to the land application site
- Localized odor problems near the anaerobic lagoons and the Desert pump station
- Disruption of potential archaeologic resources along the conveyance route and at the land application site

With proper planning, most of these adverse impacts can be minimized or avoided. Integrating public health considerations early into the planning and design of the land application and irrigation systems can minimize the potential risk to worker health and productivity from parasitic diseases that are common to irrigated lands in Egypt. Incoming workers should be screened for parasitic diseases and educated in proper hygiene. The system should be designed to minimize worker contact with surface waters and to control the propagation of intermediate hosts and disease vectors. The rapid infiltration system will provide a high level of protection against passage of wastewater pathogens to the irrigation system and, subsequently, crops produced. Appendix K describes the natural mechanisms present in the rapid infiltration beds providing this protection.

Increased ground water flows would result from the percolation of a portion of the water applied for irrigation. Much of this would flow in a southerly direction, affecting ground water levels in the northern part of Wadi Natroun and increasing the area of existing saline lakes. This impact would be similar for any type of reclamation project in the area and should be addressed as part of a comprehensive geohydrological investigation.

Contamination of the Drinking Water Canal, and the Nasser and Noubaria Canals due to potential leaks along the conveyance pipeline can be eliminated if the crossings are designed with this potential problem in mind. Pipeline crossings of the major canals could be accomplished by either submarine or overhead means. Submarine crossings would be less subject to sabotage but much more difficult to repair if ruptured. Overhead crossings, if used, would simplify inspection for leakage and could be designed to contain leakage and discharge by gravity to a point away from the canal. Localized odor problems near the anaerobic lagoons can be minimized to a certain extent by accounting for prevailing winds

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in the final site layout and landscape design. Reconnaissance archaeological surveys should be conducted in collaboration with local experts to minimize and avoid, if possible, the destruction of any significant archaeological resources.

Alternative IIIB. The beneficial and adverse environmental impacts anticipated with this alternative are similar to those described for Alternative IIIA. Somewhat greater public health risks are expected since the irrigation system relies on primary effluent. In addition, fewer benefits will be expected with agricultural production. Energy requirements are only slightly less than for Alternative IIIA.

Constructibility

As discussed in Appendix M, the most difficult construction required for the land application alternatives will be the conveyance system. Parts of the 38-km and 33-km legs of the pipelines from the East and West pump stations will cross existing and reclaimed wetlands. This will require construction on elevated pile supports. Also, the Noubaria Canal crossing will have to be carefully designed and constructed so as not to interfere with barge traffic and irrigation water supplies.

The 44-km section of the pipeline from the Desert pump station should be easier to construct. It will be buried and require only local dewatering. Our cost estimates assume that a foreign company would establish a plant in Egypt to manufacture the large diameter pressure pipe needed for the system.

The East and West pump stations will probably require dewatering during construction. However, this should not be a major hindrance since no deep excavation will be needed. These two pump stations may also require pile foundations.

The magnitude and nature of the conveyance system construction will require large, established contractors with proper equipment and experience. However, capable contractors should not have major problems and should be able to finish the work in an acceptable time period.

Construction of the pretreatment and irrigation facilities should present no problems in implementation. Little of the construction, earthwork, well drilling, onsite pipeline and canal installation, and center pivot assembly requires special skills or technology not now available in Egypt.

Schedule/Staging

Figure 5-3 presents the estimated implementation schedule and staging possibilities for the land application alternatives. The timing would be essentially the same for IIIA and IIIB. The scheduled initial operation of the land application system was delayed until July 1988, which is the earliest possible operation date of the sea disposal alternative. This was done to facilitate comparison of the land application and sea disposal alternatives in the present worth cost analysis. In reality, the land application system could be constructed sooner and start operating as early as September 1986.

The conveyance system, especially the long pipelines to the land application site, will require the most time for construction. As discussed in Appendix M, several months would be required for a foreign contractor to set up a pipe production plant in Egypt. Then another few years would be required to manufacture the pipe. Therefore, the conveyance system will be the controlling component of the implementation schedule. The land application system could be constructed in a shorter time period than shown, but it could not start operation until the conveyance system is completed.

Processes at the West Treatment Plant would be changed from domestic sewage treatment to industrial pretreatment when the land application system begins operation. However, the plant should be completed in the interim to reduce discharge of untreated domestic sewage flows to Lake Maryout as soon as possible. The East plant upgrading can proceed independently. It should also be completed as soon as possible to help improve the quality of Lake Maryout. It is on the same schedule as for Alternative IB.

The land application components of Alternatives IIIA and IIIB can be staged to match increases in wastewater flow. A 5-year staging interval was used for the present worth analysis, although shorter or longer intervals could just as well be used. The conveyance system might include some minor staging by the addition of pumps as flows increase. However, the structures would be completed for the year 2000 design flow capacity. No staging was assumed in the present worth analysis because the pumps represent only a small portion of the overall cost. All of the barrels of the pipelines must be completed to handle the peak flows expected by 1988.

Capital Costs

The estimated capital costs of the land application alternatives, as they would occur in a staged construction program, are presented in

Tables 5-1 and 5-2. Detailed capital costs for the individual components comprising the total system are presented in the following appendices: conveyance facilities (M), land application facilities (K), and treatment facilities (E, F, and J). The costs are presented in local, foreign exchange, and total categories as in Chapter 4.

The year 2000 salvage values and the value of the West Treatment Plant in 1988 are shown as negative costs in the tables. Expected service lives for the various system facilities are listed in Table 5-3. These were used with the straight-line depreciation method to determine salvage values.

The land area used in the land application system was assumed to have a salvage value of LE 2200 per feddan. This does not include the salvage value of the general site improvements and irrigation system, as those are included separately on Tables 5-1 and 5-2. The salvage value of LE 2200 per feddan is based on a total value for the land, with improvements, of LE 3000 per feddan less the value of the improvements, which are about LE 800 per feddan.

Typical land values in the area, after 10 to 15 years of reclamation, are LE 5000 or more per feddan. In economic terms, this figure could be said to represent the present worth value to the farmer of his future net agricultural returns. For this analysis, a more conservative value of LE 3000 was used because, without wastewater, the site might not have a water supply. Still, the land would have a greater value than unreclaimed land because salts would be leached from the root zone, wind breaks would be established, organic matter would be increased in the soil, and it is possible that ground water or Noubaria Canal water supplies could be developed. Thus, development of the wastewater irrigation system would significantly increase the value of the land.

The total land value was based on data provided by Dr. M. A. Khalil of the University of Alexandria, Agricultural Economics Department, plus other data sources. The area used for the land application system was assumed to have essentially no initial cost since it has very limited economic value without an irrigation water supply.

Operation and Maintenance Costs

The sum of the annual operation and maintenance costs for each of the system components and the net crop income for the two land application alternatives are presented in Tables 5-4 through 5-12. Annual costs for the farm operation (the production and handling of crops) are contained in the net crop income and are not included in the quantities or costs for labor, power, fuel, chemicals, and materials. The annual costs are

presented in more detail for each of the component systems in their respective appendices. As in Chapter 4, the annual costs are presented for each year when a significant facility is added, and values for intermediate years can be computed by interpolation.

The net crop income, also shown in Tables 5-4 through 5-11, is assumed to increase from a net cost in the year of initial startup (1988), to full income after 5 years (1993). After that, the net income will be proportional to the irrigated land area (which is proportional to wastewater flow). As discussed in Appendix K, full crop production is not to be expected from reclamation projects until salts are leached from the root zone and some organic matter is added to the soil.

Annual O&M costs in Tables 5-4 through 5-12 are presented for Alternatives IIIA and IIIB for the following years:

- 1984--the 1 year when only the East plant is operating (Table 5-4)
- 1985 through 1987--the 3 years when both the East and West plants are operating (Table 5-5)
- 1988--the year the conveyance system to the desert and the land application systems start operations and the West plant ceases operation for domestic sewage treatment (Table 5-6)
- 1989--the first full year of operation of the conveyance and land application systems and the East Treatment Plant (Table 5-7)
- 1990--The first year of 11 successive years through year 2000 in which annual O&M costs are expected to increase in a straight line with flow (Table 5-8)
- 1991--the third year in the 5-year period required for the cropland to reach full production (Table 5-9)
- 1992--the fourth year in the 5-year period required for the cropland to reach full production (Table 5-10)
- 1993--the last year in the 5-year period required for the cropland to reach full production (Table 5-11)
- 2000--the last year of the present worth analysis (Table 5-12)

TECHNICAL ASSESSMENT

Operability

Each of the individual components of both land application systems is straightforward and would not be complicated to operate. The conveyance system does not require that the pumping rates be matched perfectly to the incoming wastewater flow. The pretreatment and storage systems at the land application site provide a large daily flow equalization opportunity between the conveyance and irrigation systems; therefore, matching flow between those two systems will not be critical. Anaerobic lagoons are very simple to operate; the major operational requirements are control of the outgoing flow rate to maintain the desired water level in the lagoons, and periodic draining and sludge removal.

Normal operation of the rapid infiltration system in Alternative IIIA consists only of directing the wastewater to the basins in the proper sequence for flooding and drying. The water table levels must be monitored so that the ground water mound can be properly managed and the most efficient use made of the wells.

Operation of the distribution and irrigation systems will be like that for any conventional center pivot system, except that the irrigation schedule must be more carefully coordinated with both the water supply and the farming operations. This is particularly true of Alternative IIIB as no seasonal storage of wastewater would be provided.

The operability of the East Treatment Plant would be as described for the sea disposal alternatives. Operation of the West plant, as proposed in Alternatives IIIA and IIIB, would be simpler than the greatly expanded and more complicated plant proposed in Alternative IB.

Maintainability

If corrosive industrial and other incompatible wastes are kept out of the sewerage system, the conveyance and land application facilities should be easily maintained. Although they cover a large area, all of the facilities will be very accessible. Most of the equipment, with the exception of the center pivots, is relatively common and does not require highly trained field maintenance crews. The alignment controls on the pivots are highly technical, but with a few trained staff, could be efficiently maintained.

Maintainability of the East Treatment Plant would be the same as for Alternative IB. The smaller West plant would be simpler and easier to maintain than the larger plant proposed in the sea disposal alternative.

The conveyance system pump stations will be extraordinarily large. This does not necessarily create a difficult maintenance condition. The proposed design would include overhead cranes and lifting devices to aid maintenance crews. Two redundant pumps per station would be provided so routine service could be performed regularly. The larger pump passageways should preclude plugging. Skills required to adequately maintain such large pump stations are locally available as is evidenced by the many large pump stations in operation on the widespread Egyptian canal and drain systems. The pipelines should require very little maintenance except for routine, regular servicing of the vacuum and air relief valves. These will require frequent service by trained technicians. Access to those located in wetland crossings would be facilitated by causeways or catwalks connecting to the access road as needed.

Reliability

The conveyance system is the least reliable aspect of the land application alternatives. However, its reliability is similar to that of the conveyance systems necessary for the sea disposal alternative. Additionally, if a failure did occur, such as a pipeline break, the public health impacts would be much less for land application where the conveyance system is located mostly in remote areas.

The anaerobic lagoons are very reliable for providing the level of pretreatment required for land application. The facility includes little mechanical equipment that could fail. Subsequent discharge from the lagoons to the rapid infiltration basins (Alternative IIIA) or directly to the irrigation canals (Alternative IIIB) would be by gravity flow. Therefore, power outages would not directly affect the operation.

The rapid infiltration system of Alternative IIIA is conservatively designed so that it should provide very reliable treatment and operation. Very little mechanical equipment and no pumping would be required to spread the wastewater on the basins. Pumping the treated effluent from the ground, which is an integral part of the rapid infiltration system, would be reliable due to the large number of wells incorporated in the design. Mechanical failure of a few wells would affect only a small percentage of the rapid infiltration basins. Power outages lasting several days could occur with no adverse impact on the ground water.

The major concern would be power outages that could last for more than a few days. If this very unlikely event were to occur, especially during hot weather, many crops would be damaged or killed because the soil has a low moisture-holding capacity. The risk of crop damage due to a long-term power outage is the same as for other nearby irrigation projects.

However, the land application site should have a relatively reliable power supply because of the major double circuit 220-kV transmission line that crosses the site. All of the required transformers and distribution systems would be newly constructed as part of the land application project. For this reason their reliability should be good.

The onsite wastewater distribution and irrigation systems should be very reliable, especially taken as a whole. The system is composed of many separate units--pump stations, sections of canals, and center pivots--that can all be isolated if they fail. This would allow the remainder of the system to continue operation.

The reliability of the East and West plants would be the same as in the sea disposal alternatives.

Alternatives IIIA and IIIB would have about the same overall reliability. Alternative IIIB may have slightly greater problems in irrigation because the wastewater will have received a lower level of pretreatment and some remaining solids may plug the irrigation equipment. However, this alternative would also involve less land area and no intermediate treatment process (i.e., the rapid infiltration system of Alternative IIIA), which makes it a simpler system.

Flexibility

Both the conveyance and land application systems have some flexibility to adapt to changing future conditions. The conveyance system end point will be fixed at the East Site. However, future increases in wastewater flow could be withdrawn at other points along the route, treated and used for crop irrigation. The multiple pipeline arrangement also increases the opportunity to pump at a wider range of flow rates.

The land application systems have flexibility in pretreatment, staging, crop selection, and farm management. Initially, Alternative IIIA could be built similar to IIIB, with the rapid infiltration system being added at a later date. The facilities of both land application systems can be staged, which allows them to be more closely matched to actual wastewater flow increases.

The cropping pattern of both systems can be adjusted to a degree to match market conditions and maximize income. The management of the irrigation and farming systems can be accomplished by a range of options, from totally public to totally private enterprises. Alternative IIIA would be somewhat more flexible in crop selection and farm management than IIIB, due to the higher quality water for irrigation and the provision of seasonal storage.

The East plant flexibility options would be the same as for Alternative IB. The West plant could be kept as originally planned for primary treatment and discharge, be upgraded for better domestic sewage treatment, or be used for industrial pretreatment.

Expandability

The conveyance system could be expanded by extending its length or adding branch pipelines, increasing flow equalization capacity, adding larger or more pumps, and by adding additional parallel pipelines. Suitable land is available in the area adjacent to the East Site that could be used if the system were to be expanded. Crop irrigation could also be expanded by serving lands adjacent to the conveyance system route. Both the East and West plants could be expanded or upgraded to a limited degree as discussed in Chapter 4 and Appendix J.

Potential for Upgrading Treatment

Alternative IIIB could be upgraded at a later date by adding rapid infiltration, which would essentially be implementing Alternative IIIA. If during design a decision were made to accommodate such an option, the irrigation system could be designed to leave a suitable area for later addition of rapid infiltration basins.

Energy Consumption

The estimated annual consumption of electricity and fuel for each of the systems is presented in Tables 5-4 through 5-12. The total energy usage, expressed in the common units of billions of kilocalories (kcal x 10^9) for each of the years of operation from 1984 through 2000, is presented in Table 5-13. As in Chapter 4, the energy value of electricity is computed using a 32-percent efficiency rate for converting fossil fuel to electric power.

PRESENT WORTH ANALYSIS

The same present worth analysis method described in Chapter 4 was used for the land application alternatives. Table 5-14 presents the combined present worth costs for Alternatives IIIA and IIIB, which include capital costs, operation and maintenance costs through the year 2000, and the salvage values at the end of the period. The present worth of Alternative IIIA is LE 346.8 million, while that for Alternative IIIB is about 7 percent higher, at LE 368.8 million.

As with sea disposal, operation and maintenance costs would likely be higher if price controls and subsidies did not impact the labor, energy, and consumables markets. Also, depending on the crops grown, the net farm income could be higher or lower than the values used in the analysis, depending on government controls. Sensitivity to such changes is discussed in Chapter 6.

Table 5-1
SUMMARY OF ESTIMATED CAPITAL COSTS
LAND APPLICATION ALTERNATIVE IIIA
(costs in end-of-1980 prices)

System Component	Midpoint of Capital Expenditure[a]	Local Cost (10^6 LE)	Foreign Exchange Cost (10^6 $)	Total Capital Cost[b] (10^6 LE)	End-of-Year 2000 Salvage Value[c] (10^6 LE)
Site and Route Investigations and Environmental Analysis	12/31/81	2.3	3.4	4.7	0
Conveyance System	1/15/87	542.9[d]	139.0	640.2[d]	450.4[d]
Land Application System					259.9[e]
1st stage:	2/6/87				-
Treatment[f]		31.1	13.5	40.5	
Irrigation[g]		67.9	57.1	107.9	
2nd stage:	7/1/90				-
Treatment		3.4	0.7	3.9	
Irrigation		7.1	5.1	10.7	
3rd stage:	7/1/95				-
Treatment		3.2	0.3	3.4	
Irrigation		8.8	5.3	12.5	
East Plant Upgrade	3/26/83	6.7	13.4	16.1	9.6
West Plant Upgrade	3/6/84	5.8	6.0	10.0	16.7[c]
					16.7 in 1988[c]
Total		679.2	243.9	849.9	719.9 in 2000

[a]For midpoint of capital expenditures see Figure 5-3.

[b]Includes local costs (LE) plus foreign exchange cost at $1.00 US = LE 0.7.

[c]The salvage value of the West plant will occur in 1988, rather than in 2000.

[d]Includes LE 2.5 million land cost.

[e]Includes the salvage value from all three stages of construction, plus the land salvage value of LE 152 million.

[f]Includes anaerobic lagoons, rapid infiltration, and a portion of the administration facilities.

[g]Includes irrigation, drainage, and a portion of the administration facilities.

Table 5-2
SUMMARY OF ESTIMATED CAPITAL COSTS
LAND APPLICATION ALTERNATIVE III[illegible]
(costs in end-of-1980 prices)

System Component	Midpoint of Capital Expenditure[a]	Local Cost (10^6 LE)	Foreign Exchange Cost (10^6 $)	Total Capital Cost[b] (10^6 LE)	End-of-Year 2000 Salvage Value[c] (10^6 LE)
Site and Route Investigations and Environmental Analysis	12/31/81	1.5	2.2	3.0	0
Conveyance System	1/15/87	542.9[d]	139.0	640.2[d]	450.4[d]
Land Application System					165.2[(5)]
1st stage:	2/6/87				-
Treatment[f]		13.0	3.7	15.6	
Irrigation[g]		61.7	26.9	80.5	
2nd stage:	7/1/90				-
Treatment		1.5	0.3	1.7	
Irrigation		6.7	2.1	8.2	
3rd stage:	7/1/95				-
Treatment		1.6	0.4	1.9	
Irrigation		7.1	3.2	9.3	
East Plant Upgrade	3/26/83	6.7	13.4	16.1	9.6
West Plant Upgrade	3/6/84	5.8	6.0	10.0	16.7[c] 16.7 in 1988[c]
Total		648.5	197.2	786.5	625.2 in 2000

[a]For midpoint of capital expenditures see Figure 5-3.

[b]Includes local costs (LE) plus foreign exchange cost at $1.00 US = LE 0.7.

[c]The salvage value of the West plant will occur in 1988, rather than in 2000.

[d]Includes LE 2.5 million land cost.

[e]Includes the salvage value from all three stages of construction, plus the land salvage value of LE 95.3 million.

[f]Includes anaerobic lagoons, rapid infiltration, and a portion of the administration facilities.

[g]Includes irrigation, drainage, and a portion of the administration facilities.

Table 5-3
SERVICE LIFE PARAMETERS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB

Type of Component	Service Life
Land	Permanent
Water conveyance structures (including pipelines, canals, and drains), earthwork, and windbreaks	50 years
Other structures (including buildings, concrete structures, canal linings, and pump stations) and wells	40 years
Electrical systems and domestic water system	30 years
Auxiliary equipment (including instrument and control facilities; pumps and electrical motors, canal gates, and water screens), vehicles, center pivots and other sprinkler systems, and fences	15 years

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Table 5-4
SUMMARY OF ESTIMATED 1984 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor		Power		Fuel		Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost			
Conveyance System	-	-	-	-	-	-	-	-	-
Land Application System	-	-	-	-	-	-	-	-	-
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	11.12	72	28.8	256	12	1	541	-	870

[a]Includes chemicals for East plant operations.
[b]Net farm income equals gross crop income less crop production costs; see Appendix K.
[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-5
SUMMARY OF ESTIMATED 1985-1987 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor		Power		Fuel		Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost			
Conveyance System	-	-	-	-	-	-	-	-	-
Land Application System	-	-	-	-	-	-	-	-	-
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	8.0	53	0.4	4	11	1	482	-	540
Total	19.12	125	29.2	260	23	2	1,023	-	1,410

[a]Includes chemicals for East plant operations.
[b]Net farm income equals gross crop income less crop production costs; see Appendix K.
[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-6
SUMMARY OF ESTIMATED 1988 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	49.76	324	92.2	816	-	-	819	-	1,959
Land Application System:									
Treatment & Rapid Infiltration	4.23	61	17.8	185	-	-	84	-	330
Irrigation & Agriculture	14.82	176	29.1	303	-	-	487	1,950	2,916
Administration & Support	3.85	59	-	-	215	7	38	-	104
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	4.00	27	0.2	2	6	1	241	-	271
Total	87.78	719	168.1	1,562	233	9	2,210	1,950	6,450
Alternative IIIB									
Conveyance	49.76	324	92.2	816	-	-	819	-	1,959
Land Application System:									
Treatment	2.21	36	-	-	-	-	12	-	48
Irrigation & Agriculture	11.82	142	26.6	196	-	-	417	1,200	1,955
Administration & Support	3.27	52	-	-	233	8	36	-	96
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	4.00	27	0.2	2	6	1	241	-	271
Total	82.18	653	147.8	1,270	251	10	2,066	1,200	5,199

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production; costs see Appendix K. In the first year, no crops are harvested; therefore, cost of crop production is incurred with no offsetting crop sales.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-7
SUMMARY OF ESTIMATED 1989 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	99.52	647	192.0	1,691	-	-	1,638	-	3,976
Land Application System:									
Treatment & Rapid Infiltration	8.80	127	37.0	385	-	-	176	-	688
Irrigation & Agriculture	30.79	366	60.6	630	-	-	1,012	-2,647	-639
Administration & Support	8.00	123	-	-	448	15	79	-	217
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	158.23	1,335	318.4	2,962	460	16	3,446	-2,647	5,112
Alternative IIIB									
Conveyance	99.52	647	192.0	1,691	-	-	1,638	-	3,976
Land Application System:									
Treatment	4.61	75	-	-	-	-	25	-	100
Irrigation & Agriculture	24.62	296	55.4	408	-	-	869	-972	601
Administration & Support	6.81	108	-	-	485	17	75	-	200
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	146.68	1,198	276.2	2,355	497	18	3,148	-972	5,747

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production costs; see Appendix K.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-8
SUMMARY OF ESTIMATED 1990 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	99.52	647	199.7	1,753	-	-	1,638	-	4,038
Land Application System:									
Treatment & Rapid Infiltration	9.13	132	38.4	399	-	-	182	-	713
Irrigation & Agriculture	31.96	380	63.0	655	-	-	1,053	-5,500	-3,412
Administration & Support	8.30	128	-	-	465.1	16	82	-	226
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	160.04	1,359	329.9	3,063	477.1	17	3,496	-5,500	2,435
Alternative IIIB									
Conveyance	99.52	647	199.7	1,753	-	-	1,638	-	4,038
Land Application System:									
Treatment	4.78	78	-	-	-	-	26	-	104
Irrigation & Agriculture	25.56	308	58.0	424	-	-	902	-2,019	-385
Administration & Support	7.06	112	-	-	503	18	78	-	208
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	148.04	1,217	286.5	2,433	515	19	3,185	-2,019	4,835

[a]Includes chemicals for East plant operations.
[b]Net farm income equals gross crop income less crop production costs; see Appendix K.
[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-9
SUMMARY OF ESTIMATED 1991 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor		Power		Fuel		Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
	1000 Mandays	Cost	10^6 kWh	Cost	1000 Litres	Cost			
Alternative IIIA									
Conveyance	99.52	647	205.6	1,799	-	-	1,638	-	4,084
Land Application System:									
Treatment & Rapid Infiltration	9.36	135	39.4	409	-	-	187	-	731
Irrigation & Agriculture	32.77	390	64.5	671	-	-	1,079	-8,501	-6,361
Administration & Support	8.51	131	-	-	477	16	84	-	231
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	161.28	1,375	338.3	3,135	489	17	3,529	-8,501	-455
Alternative IIIB									
Conveyance	99.52	647	205.6	1,799	-	-	1,638	-	4,084
Land Application System:									
Treatment	4.90	80	-	-	-	-	27	-	107
Irrigation & Agriculture	26.20	315	59.5	435	-	-	924	-3,120	-1,446
Administration & Support	7.24	115	-	-	516	18	80	-	213
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	148.98	1,229	293.9	2,490	528	19	3,210	-3,210	3,828

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production costs; see Appendix K.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-10
SUMMARY OF ESTIMATED 1984 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	99.52	647	211.5	1,845	-	-	1,638	-	4,130
Land Application System:									
Treatment & Rapid Infiltration	9.59	138	40.4	419	-	-	192	-	749
Irrigation & Agriculture	33.56	399	66.1	688	-	-	1,105	-11,644	-9,452
Administration & Support	8.72	134	-	-	488	17	87	-	238
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	162.51	1,390	346.8	3,208	500	18	3,563	-11,644	-3,465
Alternative IIIB									
Conveyance	99.52	647	211.5	1,845	-	-	1,638	-	4,130
Land Application System:									
Treatment	5.02	82	-	-	-	-	28	-	110
Irrigation & Agriculture	26.83	323	60.9	445	-	-	947	-4,279	-2,564
Administration & Support	7.42	118	-	-	528	19	81	-	218
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	149.91	1,242	301.2	2,546	540	20	3,235	-4,279	2,764

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production costs; see Appendix K.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

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Table 5-11
SUMMARY OF ESTIMATED 1993 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	99.52	647	217.3	1,888	-	-	1,638	-	4,173
Land Application System:									
Treatment & Rapid Infiltration	9.82	142	41.3	429	-	-	196	-	767
Irrigation & Agriculture	34.37	408	67.7	704	-	-	1,131	-14,974	-12,731
Administration & Support	8.93	138	-	-	500	17	89	-	244
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	163.75	1,407	355.1	3,277	512	18	3,595	-14,974	-6,677
Alternative IIIB									
Conveyance	99.52	647	217.3	1,888	-	-	1,638	-	4,173
Land Application System:									
Treatment	5.14	84	-	-	-	-	28	-	112
Irrigation & Agriculture	27.48	331	62.4	456	-	-	969	-5,500	-3,744
Administration & Support	7.60	120	-	-	541	19	84	-	223
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	150.85	1,254	308.5	2,600	553	20	3,260	-5,500	1,634

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production costs; see Appendix K.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-12
SUMMARY OF ESTIMATED 2000 O&M COSTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Labor 1000 Mandays	Labor Cost	Power 10^6 kWh	Power Cost	Fuel 1000 Litres	Fuel Cost	Mat'ls[a] Cost	Net[b] Farm Income	Total[c] Cost
Alternative IIIA									
Conveyance	99.52	647	256.9	2,206	-	-	1,638	-	4,491
Land Application System:									
Treatment & Rapid Infiltration	11.44	165	48.0	499	-	-	228	-	892
Irrigation & Agriculture	40.04	475	78.7	818	-	-	1,315	-17,807	-15,199
Administration & Support	10.40	160	-	-	581	20	103	-	283
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	172.52	1,519	412.4	3,779	593	21	3,825	-17,807	-8,665
Alternative IIIB									
Conveyance	99.52	647	256.9	2,206	-	-	1,638	-	4,491
Land Application System:									
Treatment	5.98	98	-	-	-	-	33	-	131
Irrigation & Agriculture	31.98	384	72.0	530	-	-	1,127	-6,538	-4,497
Administration & Support	8.84	140	-	-	629	22	97	-	259
East Plant Upgrade	11.12	72	28.8	256	12	1	541	-	870
West Plant Upgrade	-	-	-	-	-	-	-	-	-
Total	157.44	1,341	357.7	2,992	641	23	3,436	-6,538	1,254

[a]Includes chemicals for East plant operations.

[b]Net farm income equals gross crop income less crop production costs; see Appendix K.

[c]Total is sum of labor, power, fuel, and materials, less farm income.

Table 5-13
SUMMARY OF ENERGY REQUIREMENTS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(kcal x 10^9)

	Alternative IIIA				Alternative IIIB			
Year	Liquid Fuel	Electricity[a]	Chlorine Manufacture[b]	Total	Liquid Fuel	Electricity[a]	Chlorine Manufacture[a,b]	Total
1984	-	77	1	78	-	77	1	78
1985	-	78	1	79	-	78	1	79
1986	-	78	1	79	-	78	1	79
1987	-	78	1	79	-	78	1	79
1988	2	451	1	454	2	396	1	399
1989	4	853	1	858	4	740	1	745
1990	4	884	1	889	4	768	1	773
1991	4	907	1	912	4	788	1	793
1992	4	929	1	934	4	807	1	812
1993	4	952	1	957	4	827	1	832
1994	4	974	1	979	4	846	1	851
1995	4	996	1	1,001	4	865	1	870
1996	4	1,018	1	1,023	5	883	1	889
1997	4	1,039	1	1,044	5	902	1	908
1998	4	1,061	1	1,066	5	921	1	927
1999	4	1,083	1	1,088	5	940	1	946
2000	5	1,105	17	1,111	5	959	1	965
Total	51	12,563	17	12,631	55	10,953	17	11,025

[a]Assumes energy conversion efficiencies of 32 percent.

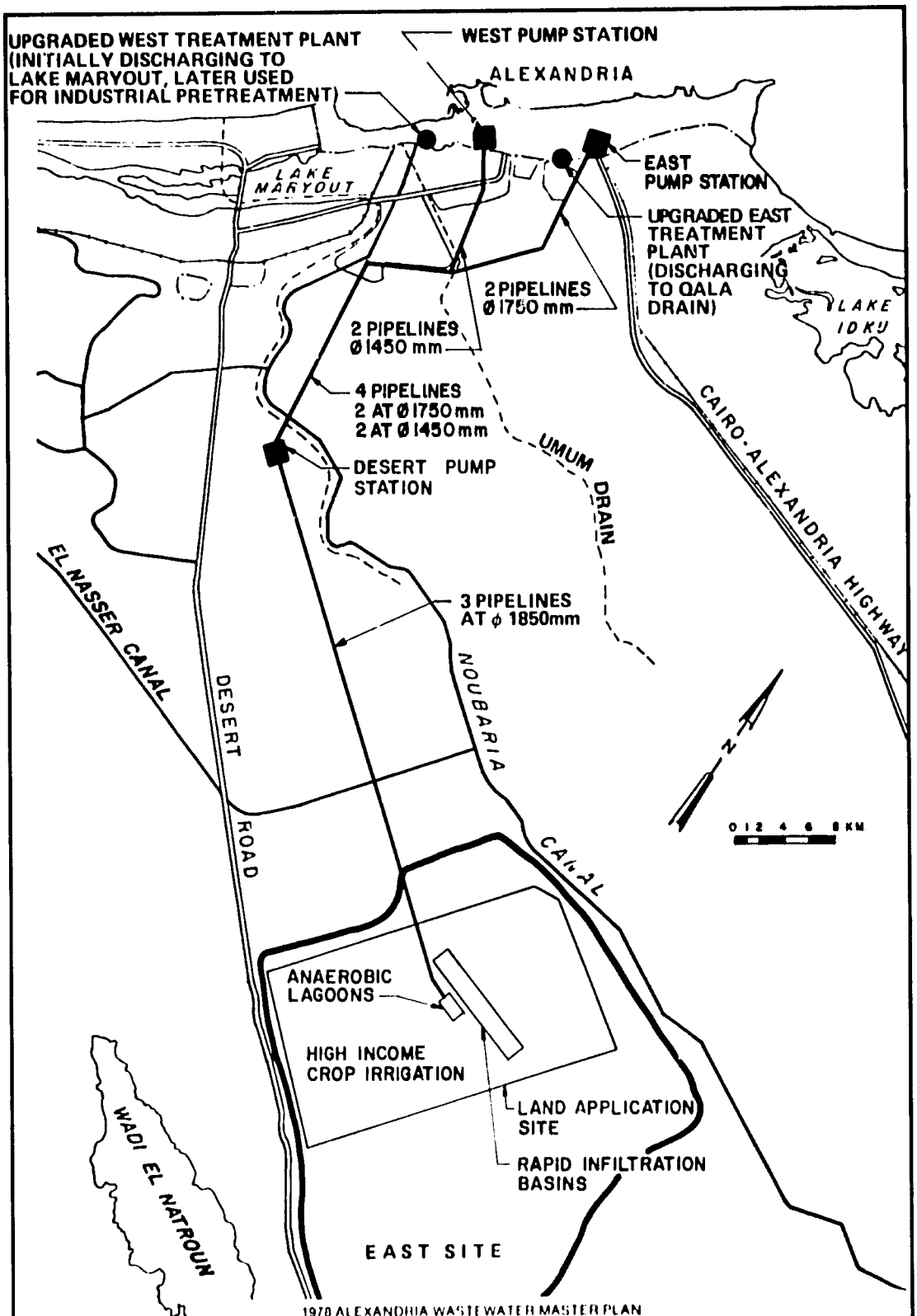
[b]Electricity to produce chlorine is included as an indirect commitment of resources. Assumes 50 percent of power used in producing chlorine and caustic in a diaphragm or membrane cell plant is assigned to production of chlorine (1500 kWh/tonne).

Table 5-14
ESTIMATED PRESENT WORTHS
LAND APPLICATION ALTERNATIVES IIIA AND IIIB
(10^3 LE, end-of-1980 prices)

System Components	Capital		O&M		Salvage Value		Total
Alternative IIIA							
Site and Route Investigations and Environmental Analysis	4.5	+	0	-	0	=	4.5
Conveyance System	450.4	+	24.1	-	140.6	=	333.9
Land Application System[b]	119.2	+	-46.9	-	81.8	=	-9.5
East Plant Upgrade	14.1	+	7.9	-	3.0	=	19.0
West Plant Upgrade	8.3	+	1.4	-	10.8	=	-1.1
Total	596.5	+	-13.5	-	236.2		346.8
Alternative IIIB							
Site and Route Investigations and Environmental Analysis	2.9	+	0	-	0	=	2.9
Conveyance System	450.4	+	24.1	-	140.6	=	333.9
Land Application System[b]	77.9	+	-12.3	-	51.5	=	14.1
East Plant Upgrade	14.1	+	7.9	-	3.0	=	19.0
West Plant Upgrade	8.3	+	1.4	-	10.8	=	-1.1
Total	553.6		21.1		205.9		368.8

[a]Discount rate of 6 percent used.

[b]Includes Stage 1, 2, and 3 capital and salvage values.



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE 5-1

ALTERNATIVE IIIA FACILITIES

A/GOSD WWCQ W/ECQ

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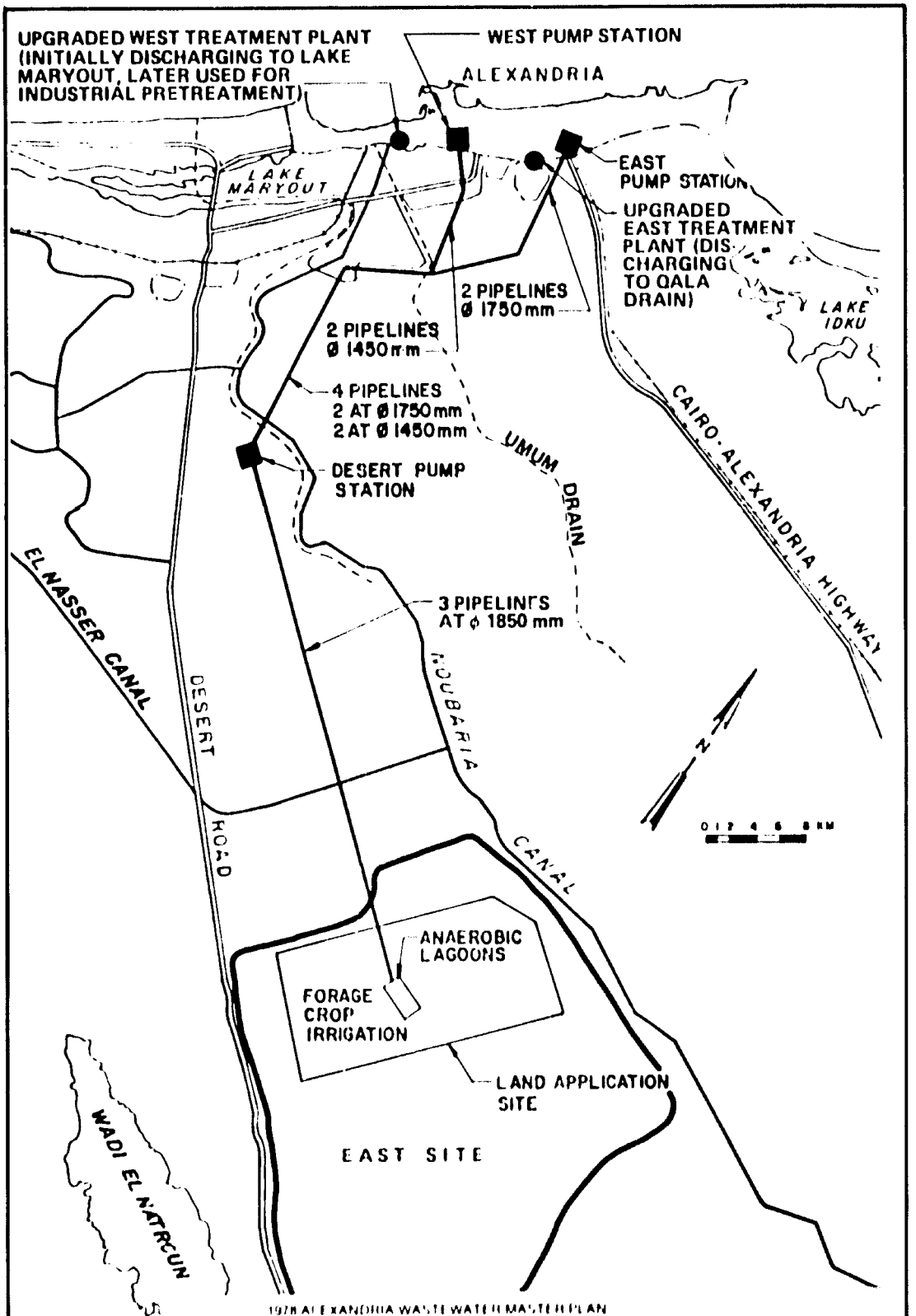
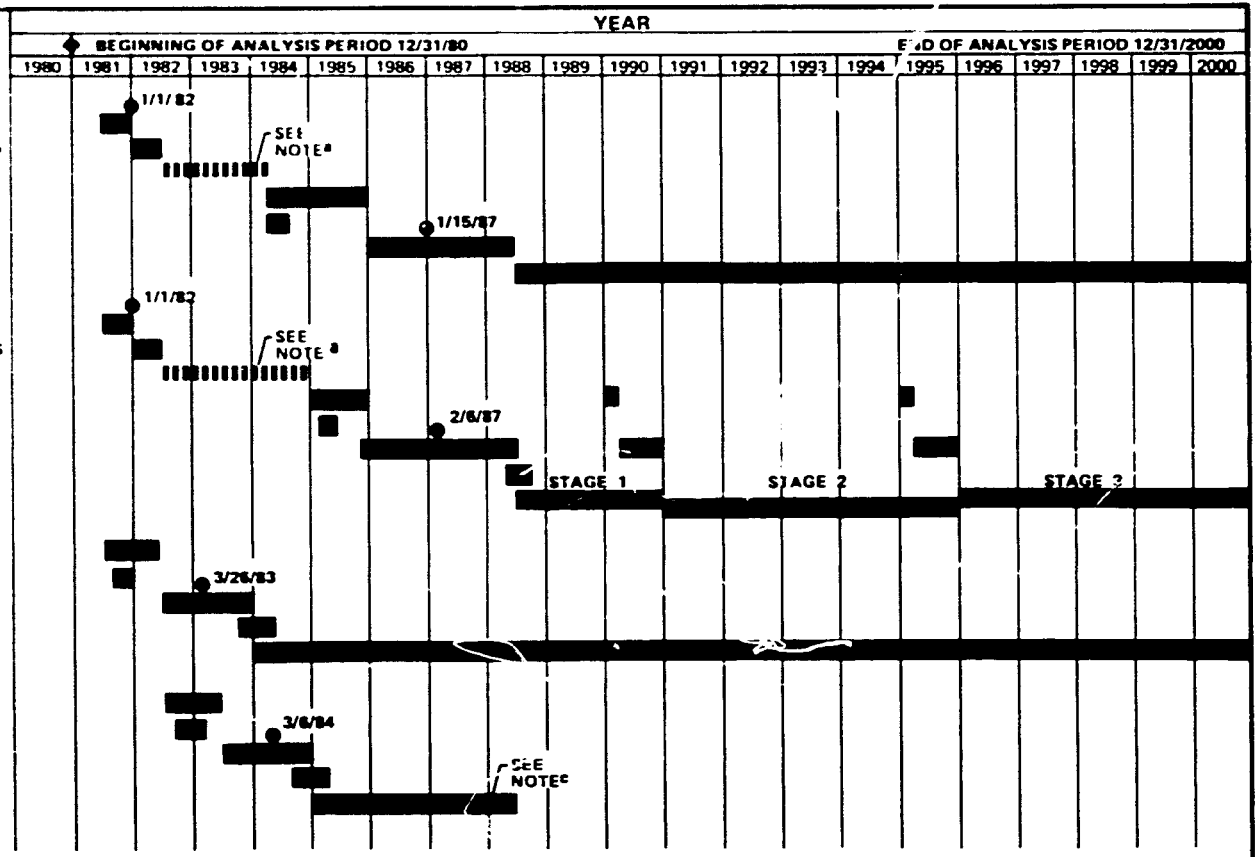


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE 5-2

ALTERNATIVE IIID FACILITIES

A GOSD WWCG WECG



[a] Undesignated time was inserted to delay operation until 7/88, the scheduled startup time for Alternative IB. This was done to make the present worth analysis comparable. In reality, the land application system could begin operations in 9/86.

[b] Includes basis of design reports.

[c] The West plant would be abandoned for domestic sewage treatment and converted to industrial pretreatment when the land application system begins operation.

LEGEND:

◆ Point in time to which all capital expenditures are indexed and present worth established.

● Midpoint of capital expenditure for phase of system component (site investigations plus environmental analysis) or (design plus purchase plus construct plus startup).

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE 5-3

IMPLEMENTATION SCHEDULE FOR LAND APPLICATION ALTERNATIVES IIIA AND IIIB

A/GOSD WWCG W/ECG

CHAPTER 6
EVALUATION OF ALTERNATIVES

INTRODUCTION

This chapter evaluates and compares the three most viable alternatives for wastewater treatment and disposal which have been developed based upon a reconnaissance level investigation. These alternatives are:

- o IB Sea Disposal with Two Primary Plants
- o IIIA Land Application with High Income Crops
- o IIIB Land Application with Forage Crops

The USAID Handbook has been used as the basic guideline for the evaluation of alternatives. The Handbook does not specify a rigid methodology. Rather, it permits the exact methodology to vary so the most credible and relevant approach can be used for a particular project.

The guidelines provide for either cost-effectiveness or cost-benefit analyses to be used in evaluating alternatives. A cost-effectiveness analysis is usually the more appropriate technique when the project's output cannot realistically be assigned monetary values. A cost-benefit analysis is usually more appropriate when the project's outputs are primarily marketable items or when they can be valued in monetary terms.

Wastewater treatment projects have traditionally been evaluated using a cost-effectiveness analysis because pollution control benefits are largely nonmonetary in nature. A cost-effectiveness analysis is an effective tool for considering both quantitative and qualitative factors. Therefore in comparing the three most feasible alternatives for Alexandria wastewater treatment and disposal, a cost-effectiveness analysis was used as the basic tool for comparison. However, the land application alternatives provide measurable monetary benefits in addition to pollution control benefits. Therefore, a cost-benefit analysis was performed to consider whether the additional cost of either land application alternative is justifiable on the basis of the additional benefits that would accrue.

The cost-effective analysis is presented in the following section. The cost-benefit analysis is presented in the subsequent Economic Analysis section.

COST-EFFECTIVENESS ANALYSIS

This evaluation considers both quantitiative and qualitative factors. Quantitative factors are items that can be estimated in strictly numerical terms. These quantitative factors are listed below:

- Capital costs (foreign exchange, local, total)
- Operation and maintenance costs
- Present worth (total capital, total O&M, total)
- Energy consumption

Qualitative factors are items that cannot be quantified, but that affect the suitability of the alternatives. The qualitative factors that are used are listed below:

- Reliability and flexibility
- Overall operability
- Environmental factors
- Socio-political factors

Determining an alternative's reliability is a subjective judgment that assesses the probability of continuous, acceptable operation and the cause/effect relationship among events that may interrupt continuous operation. Flexibility depends on the ability to adapt an alternative's various components to accommodate changed conditions such as wastewater quantity or quality, growth rate, and effluent quality requirements.

Overall operability of an alternative concerns the operator skill levels required, the ease or difficulty of maintenance, and the necessity for and obtainability of spare parts.

Included under the general heading of environmental factors are the relative adverse impacts to the terrestrial and aquatic environments, and the relative benefit/risk to public health.

Socio-political factors include the acceptability of the alternatives by government officials, the general public, and special interest groups.

Each alternative will respond differently to changes in the financial assumptions used for cost estimating or cost-effectiveness analyses. For example, an alternative that is capital-intensive but low in opera-

tion and maintenance costs will have a certain 20-year total present worth cost associated with a given discount rate. Another alternative that is not capital-intensive but is high in operation and maintenance costs may have the same 20-year present worth cost with the same discount rate. However, if the discount rate is changed, the two alternatives will then have different 20-year present worth costs. Sensitivity to such changes and the severity of economic impacts associated with these changes are important attributes of the alternatives. Determination of such impacts is termed a sensitivity analysis. This chapter presents sensitivity analyses for changes in the following quantitative factors:

- Capital costs
- Costs of labor for operation and maintenance
- Energy costs
- Net crop returns
- Foreign exchange rates
- Land values
- Infrastructure costs
- Discount rates
- Primary effluent chlorination during bathing season only

Selecting the most suitable alternative requires decisions based on quantifiable and non-quantifiable factors. Obviously, the best solution for Alexandria should be as economical as practicable--but is an economical solution with questionable reliability and flexibility better than a more costly, but also more reliable and flexible, system? The answer to such a question largely depends on the value one places on money and quality. The answer may also depend on one's willingness to invest present capital to avoid potential future operation and maintenance costs or vice versa. Depending on the value or significance given each quantitative and qualitative factor, different alternatives may be perceived as superior.

Methodology

Two basic criteria are paramount in selecting the most suitable solution for wastewater treatment/disposal for Alexandria. The alternative should be the least costly system capable of meeting the overall

objective; and, the alternative should be acceptable by social, political, environmental, engineering, and wise investment standards. The first of these criteria requires that alternatives be compared as to their cost-effectiveness. To perform a cost-effectiveness analysis properly, the time value of money must be considered. This is accomplished by expressing future expenditures in present costs. The results are the present worth of all costs and receipts over the analysis timeframe. This present worth methodology is used for all economic comparisons.

Qualitative factors do not lend themselves to such rigorously quantifiable expressions. They can, however, be subjectively grouped from best to worst in relation to their individual impacts.

Evaluation of Quantitative Factors

Commitment of resources must always be given prime consideration in choosing among acceptable alternatives that will achieve the primary goal. The following quantifiable factors related to resource commitment were considered in the evaluation of the three final alternatives.

- Capital cost
- Operation and maintenance cost
- Total present worth cost
- Energy consumption

The relative effects of these factors are shown in Tables 6-1 through 6-4.

Cost Estimating. The development and use of a cost data base for review and update of the 1978 Master Plan is as described in Appendix B. The cost data base is for end of year 1980, or just prior to the time of its preparation. It reflects rapidly advancing construction practices in Egypt to avoid escalating costs based on outdated construction practices.

Capital costs were estimated using the following techniques:

- Cost vs. size or capacity curves based on USA experience adjusted for (1) type of work (earthwork, process equipment, concrete) to reflect local costs and (2) timeframe of cost curve data base to reflect end-of-1980 costs.
- Cost vs. size or capacity curves prepared specifically for the Alexandria projects using end-of-1980 actual labor rates, labor

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productivity and materials costs obtained by WWCG's cost research in Egypt. Appendix B lists the principal material and labor unit costs used in developing construction, operation and maintenance costs. Locally based construction cost curves for pump station and sewer projects are also presented.

- Preliminary design, quantification of the construction and extension of these quantities using locally obtained unit costs, as described above, together with pricing information received from the USA on equipment and materials to be imported.

Construction contingencies, varying from 20 to 40 percent depending upon the nature of the work, are included in capital cost estimates as described in Appendix B. The specific construction contingencies assumed for each project are presented in the capital cost tables throughout this report. An additional amount of 15 percent of construction plus contingencies is included in all capital cost estimates to cover engineering, legal, and administrative costs.

Capital costs are presented by local cost, foreign exchange, and total cost categories. An end-of-1980 exchange rate of LE 0.70 = $1.00 U.S. is assumed for purpose of total cost presentation. All costs are categorized as either local or foreign. If the needed material, equipment, labor, or service can be obtained in Egypt, it was included as a local cost; if not, it is included in the foreign exchange category. The cost categories are defined more specifically as follows:

Local Cost Items

Site and earthwork
Concrete
Masonry
Metals
Wood and plastic (as available)
Thermal and moisture protection
Doors and windows
Painting and finishes
Special interior fixtures
Furnishings
Plumbing
Light HVAC
Lighting
Concrete, concrete cylinder, and glass reinforced plastic (GRP) and small diameter steel pipe

Foreign Exchange Cost Items

Process equipment
Process mechanical
Instrumentation & control
Hoisting and conveying equipment
Process electrical systems
Ductile iron, asbestos cement, and large diameter steel pipe

The use of Egyptian construction contractors is generally assumed.

Operation and maintenance costs are estimated in a fashion similar to that used for construction cost estimating. Principal unit prices used are presented in Tables B-1 and B-2 of Appendix B. Labor costs for all portions of Alternatives IB, IIIA and IIIB, with the exception of the land application portion of the last two alternatives, average approximately LE 6.50 per man per day in end-of-1980 costs. Land application system portions of Alternatives IIIA and IIIB were estimated at approximately LE 12.95 per man per day, reflecting a higher proportion of professionals on staff. Labor rates used include fringe and overhead burdens. Labor productivity was assumed at 70 percent of USA experience.

Principal commodities used in the operation and maintenance estimates include:

- Power at LE 0.01/kWh for the first 500 kW of demand, plus LE 0.005/kWh where demand would exceed 500 kW, plus a demand charge of LE 25.0/kW
- Chlorine at LE 0.30/kg
- Polymer at LE 4.50/kg dry weight

Present worth cost estimates are for the 20-year time period beginning at the end of 1980 and ending at the end of year 2000. They include capital, operation, maintenance, and replacement costs, and end-of-year 2000 salvage values based on straight-line depreciation. A 6-percent discount rate is used. Service life parameters used in calculating replacement costs included in present worth analyses are presented in Chapters 4 and 5, Tables 4-2 and 5-3, respectively.

Capital Costs. Most, if not all, of the capital costs occur in the early portion of a project's useful life. Projects that may be ideal by all other measures may be impractical because of their excessive demand for unavailable initial capitalization.

The alternative wastewater management plans available to A/GOSD vary greatly in their capital needs. Table 6-1 summarizes these needs. Alternative IIIA has the largest capital requirements; its cost in end-of-1980 costs is estimated to be nearly LE 850 million, or about 80 percent more than the least capital-intensive alternative.

Alternative IB, which is estimated to require a capitalization of nearly LE 470 million in end-of-1980 costs. Inflation through construction will result in a further disparity in capital needs.

Operation and Maintenance Costs. Long-term satisfaction with and performance of the wastewater management alternative selected will be strongly influenced by operation and maintenance (O&M) costs. Systems with high O&M are frequently neglected during periods of economic difficulty and fall into disrepair.

Operation and maintenance costs, estimated for each alternative through year 2000, are presented in Table 6-2. Land treatment with production of high value crops is by far the best alternative with respect to O&M. Net crop returns would more than offset O&M costs through year 2000. The cash flow from the project would actually be positive after 1991 when cropping is fully established. The second land application system alternative, where forage crops are grown, has a lower annual crop return and higher total O&M costs. The sea disposal system rates poorest in this evaluation with its total O&M costs of LE 124 million (based on market prices for power and fuel) through year 2000.

The present worth of the sea disposal O&M costs could be reduced by LE 5.4 million by eliminating chlorination of sea discharges during all but the 4-month summer bathing season. Information available for preparation of this report is inconclusive as to the need for year-round chlorination. Additional studies needed to reach a conclusion in this regard are recommended in Chapter 7, "Conclusions and Recommendations." Cost-effectiveness sensitivity to the need for year-round chlorination is discussed in the section, Sensitivity Analysis.

All O&M costs and present worth evaluations assume certain levels of effective crop production and farming efficiency. If these levels are not achieved during the actual farming operations, the net O&M costs will increase for Alternatives IIIA and IIIB and resulting present worth costs may favor Alternative IB.

Total Present Worth Costs. The usefulness of present worth cost evaluations has been fully discussed. The present worth analysis, as used in this effort, considered not only capital and O&M costs through year 2000, but also included salvage values and the time value of money as well.

From a total cost standpoint, the system having the lowest present worth cost is preferred. Total present worth costs associated with each alternative are summarized in Table 6-3. The present worths of the three alternatives are closely grouped while those of capital costs and O&M costs are widely separated. The high capital costs of the most

capital-intensive alternative are largely offset by a negative O&M cost and a high salvage value. Alternative IIIA is estimated to have a present worth cost 4.5 percent greater than that of Alternative IB. The Alternative IIIB present worth cost is estimated to be 11.3 percent greater than that of Alternative IB.

Energy Consumption. Energy consumption is an important evaluation criterion because it represents an irretrievable commitment of a scarce natural resource.

The impact of the commitment can be measured either by the quantitative measure of the energy amounts committed, or indirectly through inclusion of energy costs in comparison to total O&M costs. Energy costs at the current Egyptian market rate are included in the O&M costs previously discussed. Overall cost-effectiveness sensitivity to energy costs is further discussed in the section, Sensitivity Analysis.

Estimates of the amount of energy that will be consumed are shown in Table 6-4. These estimates assume all electrical energy consumed by the project will be produced in fossil-fuel-fired plants. This assumes that Egypt's low-cost hydroelectric power is or soon will be committed to other necessary or productive uses.

As shown in Table 6-4, Alternatives IIIA and IIIB require more than twice the energy than Alternative IB. Such a comparison is complicated by the fact that the end results obtained by the land application alternatives differ greatly from those achieved by sea disposal. All three alternatives effectively dispose of sewage; the land application alternatives also provide the means (and are therefore charged with the energy requirement) to irrigate up to 54,000 feddans of land for food production. If the energy requirements to irrigate a similarly sized parcel with Nile River water (assuming it was available) were added to the energy required for sea disposal, the sum would likely be very similar to the energy needs of the land application alternatives. If effective sewage disposal and increased food production are both high priority needs and represent national objectives, then directly comparing the amounts of energy consumed in each of the alternatives is invalid. Correspondingly, if sewage disposal, per se, is the prime objective, then a direct comparison is valid and provides a useful comparative parameter.

Evaluation of Qualitative Factors

While cost-effectiveness is a prime factor in evaluating alternatives, nonmonetary criteria are also appropriate tests. These additional factors are qualitative in nature and include the following:

- o Reliability and flexibility
- o Overall operability
- o Environmental factors
- o Socio-political factors

Table 6-5 summarizes each alternative's ability to meet the objectives associated with each qualitative factor.

Reliability and Flexibility. We believe both land application alternatives offer somewhat more reliability and flexibility than the sea disposal system. Their reliability is enhanced by system simplicity. Wastewater pumping would be the most complex and probably the least reliable element in each case. Raw wastewater pumping is common to all alternatives. The pipeline conveyance system employing three parallel force mains to the land application site would add to the reliability of the two land application alternatives.

Treatment in the land application alternatives would be by anaerobic pond system, with the only necessary pond function being a reduction in settleable solids concentrations. The sea disposal system would depend upon mechanical treatment and sludge dewatering facilities. These facilities would be more complex and less reliable. Reliability of the facilities following treatment is considered equal for the alternatives considered. These facilities (outfalls, infiltration basins, and irrigation systems) are all highly reliable.

System flexibility favors the land application system. Wastewater could be removed from the force main system upstream from the land application site at any point, treated, and reused to meet future agricultural and industrial needs along the way. Once treated at the land application site by rapid infiltration in Alternative IIIA, the recovered water could be pumped to the adjacent canal system carrying water from the Nile River.

Construction of the sea disposal system would require a large capital expenditure for outfalls, which would be suited to only one purpose--discharge of treated wastewaters to the sea. The primary plants could be converted to secondary treatment facilities to help meet future industrial and agricultural water needs; however, mechanical secondary treatment and associated sludge management would be costly. This cost, together with the costs for conveying treated effluent to reuse points would be much greater than the cost of either land application system, even when discounting the lost investment in outfalls.

Any future requirement for upgrading the effluent quality could result in the need to add chemical or secondary treatment to the sea disposal treatment plants. This would add greatly to capital and O&M costs. The land application systems should not be impacted by mandates for an upgraded effluent quality since no effluent would be released into the general environment.

All three of the alternatives are judged to be equal when considering flexibility in accommodating unanticipated (increased) flows. In each case, system expansion can be accommodated.

As shown in Table 6-5, all systems are given an acceptable rating for overall reliability and flexibility.

Overall Operability. Operability considerations favor the Land Application Alternative IIIA first, the Sea Disposal Alternative second, and the Land Application Alternative IIIB third. Alternative IIIA is ranked most acceptable because of its simplicity, low levels of operation and maintenance skill required, and the limited use of components subject to wear and breakdown. Operating the pump stations would be more demanding than operation of any other component. Raw sewage pumping is common to all alternatives. Very little operational attention will be required for the treatment ponds and infiltration basins. Operation of the irrigation wells and irrigation system would be relatively simple when compared to operation of mechanical treatment facilities such as those in the sea disposal system.

Total labor requirements for the three alternatives would be fairly similar. The projected total staffs for the year 2000 would be 615 for Alternative IB, 664 for Alternative IIIA, and 606 for Alternative IIIB. The figures for the land application alternatives include staff required for conveyance, east plant operation, and operation of the land application systems through irrigation. Labor requirements for farming are not included in these totals as they could become part of a separate organization outside of A/GOSD. In addition to the total staffs noted here, the management staff of A/GOSD would have to be considerably augmented to control the new operation. The extent of this augmentation should be defined as part of the ongoing management review.

While the sea disposal system is given a lower operability rating than the land application system in Alternative IIIA, its operability is judged superior to that of the land application system in Alternative IIIB. This is justified by the sea disposal system's use of well established technology with proven operating results. In the land application system where forage crops are grown, pretreatment would be by ponds and screens only. Concern exists that some solids may pass

the screen system, creating stoppages in pumping and application equipment. The sprinkler nozzles used to apply the partially treated wastewater would be particularly vulnerable to plugging and would require considerable operational attention. In addition, the lack of water storage would constrain the operation of the agricultural activities.

The above comparisons have excluded any consideration of the farming operation. The farm is an integral part of the total system of Alternatives IIIA and IIIB, especially regarding economic comparisons involving income from crop production. If operation of the 54,000 feddan farming operation is not as effective in crop production and resultant income generation as assumed, the effectiveness of Alternatives IIIA and IIIB in providing adequate effluent disposal will not be impaired; however, the present worth comparisons and resultant economic ranking of the land disposal alternatives will be significantly affected.

Environmental Concerns. Alternative IIIA is again favored when measured by environmental factors. This alternative would require the least contact between untreated wastewater and sludge on one hand and people on the other. Operators would have little or no reason to come into contact with the wastewater in the treatment ponds or in the infiltration basins. This is contrary to experience at mechanical treatment plants such as those associated with the sea disposal system. Once the wastewater would pass through the aerobic treatment zone below the infiltration beds in Alternative IIIA, it would be of tertiary or near drinking water quality except for nutrients and dissolved solids. Appendix K describes the natural treatment processes in the aerobic zone below the infiltration beds. Water pumped from the ground would exhibit a very low disease risk. Treatment, including disinfection, and discharge of wastewater to the sea (Alternative IB), will also greatly reduce the possibility of negative impact on public health.

Alternative IIIB might result in greater environmental impacts because partially treated wastewater would be applied to crops. Personnel working around the irrigation system would occasionally come into direct contact with the wastewater, and the crops themselves could act as an intermediary in exposing workers and others who might handle the crops to pathogens.

All alternatives must depend upon effective industrial pretreatment and toxic waste exclusion programs to avoid environmental damage. Aquatic life in the sea or crops grown on any land application system would serve to concentrate the toxics. The toxics would in turn be transmitted to the human population through food consumption.

Socio-Political Concerns. Project implementability often hinges on its social and political acceptability. Acceptability or degree of acceptance is measured by many standards and objectives, some of which have their roots in culture, others religion, and still others the political structure. Generally, the standards and objectives of concern do not have unanimous support. They do have a tremendous bearing on selection among alternatives nonetheless.

Socio-political considerations in Alexandria favor wastewater reuse. Both the land application alternatives are judged superior to sea disposal for the following reasons:

- Wastewater would be removed from the sea. This would remove constraints, real or imagined, in bathing at the many beaches in and adjacent to Alexandria. Tourist-related businesses might be expected to benefit from the assurance that the sea and its beaches were free from any and all sewage, treated or otherwise. Cultural, religious, economic, and political standards and objectives all have bearing here.

- The agricultural reuse of wastewater could provide economic benefits to Egypt well beyond the scope of this study. Water, not land, may be the critical ingredient to increased agricultural production, and the land application alternatives could add directly to Egypt's crop production. Nutrients in the reused water would have value in that the incremental increase in crop production would result from a less-than-incremental increase in consumption of manufactured fertilizers. The land application systems, particularly Alternative IIIA, would produce several hundred direct and indirect job opportunities over and above the employment that would result from the sea disposal system.

- The land application system has the potential to be cost-effective in the long run. While the cost-effectiveness analysis stops at the end of year 2000, the Alternative IIIA land application system does not start "paying out" until 1991. The users and taxpayers of the 21st century would be the primary financial beneficiaries. The desire to build a system promising this long-term benefit does have considerable merit in long-range economic planning, if initial capital requirements fit the current financial priorities and capabilities of Egypt.

While socio-political considerations favor land application over sea disposal, the two land application alternatives cannot be viewed equally since Alternative IIIA would provide significantly greater socio-political benefits.

Sensitivity Analysis

The cost-effectiveness analysis of alternatives depends to a large extent on the validity of the estimates on which it is based. Planning level estimates cannot be as accurate as estimates based on detailed construction plans. Therefore, a sensitivity analysis was performed to determine the effect of potential changes. Potential changes investigated included:

- A 20-percent capital cost increase
- A 50-percent O&M labor cost increase
- A 600-percent energy cost increase
- A 50-percent net crop income increase over current market prices
- Devaluation of the Egyptian pound from LE 0.7 to 0.9 = $1.00 U.S.
- LE 2000 per feddan increase in salvage values of irrigable land
- Inclusion of "social" infrastructure costs at LE 500 per feddan
- Changing the "opportunity" cost of money (present worth discount rate) from 6 percent to 3 and 9 percent, respectively
- Chlorination of Ras El-Soda and West Treatment Plant effluents during the 4 summer bathing months only

The individual change in the present worth of all estimated costs through the year 2000 caused by each of these possible changes is shown on Table 6-6.

As anticipated, capital and energy cost increases, discount rate increases, the additional infrastructure costs, and part-time chlorination adversely affect land application alternatives more than sea disposal. Operation and maintenance labor cost increases affect all alternatives essentially equally. The remaining changes affect the sea disposal alternative more adversely than the land application alternatives.

ECONOMIC ANALYSIS

Preceding sections of this chapter have concentrated on economic and qualitative comparison methods as they are normally employed in the pro-

cess of facilities planning in U.S. practice. Incumbent in such analyses are certain conditions:

- o Inflation is assumed to have an equal impact on all commodities except, in some cases, on energy.
- o Salvage values represent the value of the remaining useful life of a facility after the end year of the present worth analyses rather than the value of a "sale" of the asset.
- o Current actual commodity prices are used.

This type of analysis works well in situations where subsidies, price supports, and price controls are absent, and nationwide self-sufficiency in terms of all requisite project labor and materials is present. These requisites are not the case in Alexandria. Among other commodities, food, fuel, and electrical power are subsidized in Egypt; i.e., intentionally sold at less than world market prices. Labor and housing rental rates are likewise government-controlled, presumably at levels consistent with the subsidies employed. The Egyptian pound is not a "hard" currency. While the official rate of exchange vis-a-vis the U.S. dollar is approximately 82 pt. per dollar (up from 70 pt. per dollar in early 1981), there is every indication that if the pound were to be allowed to "float" against the dollar, the exchange would currently be 90 to 95 pt. per dollar.

Because such conditions as mentioned can represent very significant constraints to any overall solution to Alexandria's sewerage needs, USAID has asked that an analysis based on a "free market approach" be included in the comparison of alternatives. The accomplishment of a rigorous "free market approach" analysis would be an extremely complex undertaking and is outside the scope of work currently authorized for this report. The complexity arises because of the fact that within Egypt, as elsewhere in the world, commodity prices and costs, prevailing subsidies and controls, etc., are all interrelated. Therefore, a major change in one item inevitably leads to a chain reaction of pressures to force reactive and complementary changes in other items. A "free market approach" analysis is nevertheless an important comparative tool and so a preliminary analysis was conducted and is presented in Appendix N. The following is a summary of this analysis.

Study Approach

The economic analysis was conducted from the national viewpoint. Its purpose was to provide Egyptian and USAID officials with a basis for deciding which wastewater disposal alternative would provide the greatest net benefit to the economy of Egypt as a whole. Therefore,

opportunity costs and prices (often referred to as "shadow prices") were used in measuring benefits and costs. These shadow prices reflect the real cost of production inputs and outputs to the national economy with the influence of market distortions removed. Examples of distortions that have been removed include artificially low prices for some crops caused by government controls, and subsidized prices for some farm production items such as fertilizers, credit, and improved seeds.

The economic analysis is based on a useful life for project facilities of 50 years. However, the structures [illegible]zed for the year 2000 design flow capacity.

Data used in the analysis were obtained by a review of previous similar studies, published and unpublished information available from Egyptian government agencies, interviews with knowledgeable professionals, site investigations by the study team, and engineering cost estimates. The data are regarded as adequate for the purposes of updating the master plan. However, more detailed investigations will be required to confirm the economic feasibility of the selected alternative.

Project Benefits

The benefits of each project were measured on the basis of the "with and without principle." The future conditions that would result from the project were compared with conditions anticipated to occur without the project. Thus, the net improvements would be benefits attributable to the project.

The adverse environmental impacts associated with taking no action to improve the wastewater facilities of Alexandria are regarded as unacceptable. The public health consequences of increased population densities in nonsewered areas and increases in the discharges of untreated wastewater flows would be severe. The risks of contacting contaminated water would increase dramatically. In addition, the worsening of beach pollution could have a significant adverse impact on the important tourism economy of the Alexandria region.

Alternatives IB, IIIA, and IIIB would all provide the necessary protection to public health and the environment. However, the land application of reclaimed water would result in additional benefits for Alternatives IIIA and IIIB. Consequently, the benefit analysis was focused primarily on determining the incremental irrigation benefits for the land application alternatives.

Irrigation benefits were measured on the basis of a farm budget analysis. Direct irrigation benefits were defined as the return to water (net farm income) that would result from irrigating with the reclaimed

water. The return to water was calculated as a residual by estimating gross crop income and deducting crop production costs (including a return to farm labor) and returns on investment (6 percent) and to management (1 percent of gross income).

The return to water was calculated for representative crops expected to be grown in the irrigation service area. The anticipated cropping pattern was then used as a basis for calculating the weighed average benefit on an annual basis.

Direct irrigation benefits at full project development would amount to LE 42.0 million per year for Alternative IIIA and LE 5.8 million per year for Alternative IIIB. The computations of these benefits are shown in Tables 6-7 and 6-8.

For either land application alternative, full crop production would not be achieved for about 5 years after irrigation was begun. Crop production--and hence direct irrigation benefits--would be low during the first few years of irrigation. During this period, salts would be leached from the root zone and organic matter incorporated into the soil. This 5-year development period is reflected in scheduling the irrigation benefits from each land application alternative. The schedule of benefits also reflects the projected increase in wastewater flows up to the year 2000 design capacity.

In addition to the direct irrigation benefits, Land Application Alternative IIIA would provide important benefits in achieving the national goal of food security. The annual value expressed in 1980 shadow prices of food crops produced with the year 2000 wastewater flow would be LE 71.2 million at 1980 price levels.

Project Costs

A schedule of construction, operation and maintenance, and replacement costs, along with the monetary benefits and net benefits for Alternatives IB, IIIA and IIIB, are presented in Table 6-9 through 6-11, respectively. The costs are based on shadow price levels at the end of 1980. Since the costs are based on shadow prices, they differ to some extent from those based on anticipated market prices, as presented in other sections of the report.

Energy costs are an important consideration in evaluating the land application alternatives. The energy requirements for both land application alternatives would be substantially greater than for the sea disposal alternative. Energy costs (power at LE 0.056/kWh) in the economic analysis were based on marginal rates (shadow prices) which reflect the full cost of developing new sources of energy supply.

Comparison of Alternatives

The comparison of alternatives was made on the basis of an incremental analysis. Both of the land application alternatives would be substantially more costly in terms of initial capital cost than the sea disposal alternative. All three alternatives would achieve the basic purpose of maintaining public health and environmental standards. Therefore, the focus of the analysis was on whether the additional costs for land application of the wastewater would be justified on the basis of the additional economic benefits.

The internal rate of return (IRR) was computed as a measure of the economic worth of the land application increments. The IRR is the interest rate at which the present worth of the future flow of monetary benefits would be equal to the future flow of monetary costs. It indicates the discount rate at which the benefit-cost ratio would be 1.0.

The IRR for incremental costs and benefits for Alternative IIIA is 7 percent over a 50-year period of analysis at the foreign exchange rate of LE 0.70 = $1.00 U.S. For Alternative IIIB, incremental project costs would exceed incremental benefits even at an interest rate of zero.

A Sensitivity Analysis of the Internal Rate of Return

A sensitivity analysis was also conducted to determine whether the IRR would be significantly influenced by different assumptions about costs and benefits or variations from the estimates. Results of this sensitivity analysis for Alternative IIIA are presented in Table 6-12.

A delay of 5 to 10 years in the time required to attain full crop yields would reduce the IRR from 7.0 to 5.5 percent. If construction costs increase 10 percent above the estimates, the IRR would be 6.3 percent.

Substantial amounts of electricity would be used for pumping in Alternative IIIA. A marginal rate of LE .056 per kWh was used to compute energy costs in this economic analysis. If this rate is increased 10 percent, the IRR would be 6.6 percent. Please note that marginal rates are substantially higher than market rates for electricity.

If the shadow prices for crops produced on irrigated lands are increased 10 percent, the IRR would increase to 8.9 percent. This indicates the study results are significantly influenced by the level of crop prices.

The IRR decreased to 6.8 percent if the salvage values at the end of the 50-year period of analysis are excluded. Although the salvage values are substantial, they are heavily discounted in the analysis on a present worth basis.

The IRR increased from 7.0 percent to 7.7 percent when the foreign exchange rate was changed from LE 0.70 = $1.00 U.S. to LE 0.85 = $1.00 U.S. This indicates that the use of an artificially low exchange rate tends to understate the benefits in relation to the costs for Alternatives IIIA. If the trend toward devaluation of the Egyptian currency continues, land application of the wastewater under Alternative IIIA will become increasingly desirable.

In conclusion, it appears some delays in reaching full project development or minor variations in construction costs, electricity costs, or crop prices could occur without significantly influencing the economic feasibility of Alternative IIIA. Of those variables tested, changes in crop prices and the foreign exchange rate appear to have the greatest potential influence on study results. Another critical factor, of course, is the achievement of the assumed farming production outputs. This is related to the ability of the farm operator to implement and maintain farming practices at consistently high levels in order to achieve the assumed crop production.

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

This chapter sets forth the principal conclusions and recommendations reached from the reconnaissance-level studies and analyses described in the preceding six chapters and the appendices to this report. Emphasis is placed on conclusions and recommendations pertinent to the choice between the basic alternatives for wastewater management discussed in Chapters 4, 5, and 6. These are sea disposal and land application.

These alternatives have been described in detail in Chapters 4 and 5, respectively, and evaluated in detail in Chapter 6 of this report. The conclusions and recommendations presented in this chapter have their bases not only in this report and its accompanying volume of appendices, but in discussions and considerations involving A/GOSD, USAID, and WWCG that have taken place since the initial drafting of this report in May 1981.

The draft report, republished in final form after revisions to accommodate comments received and the addition of this chapter, a new Appendix N, and an executive summary, constitute the final report on the Review and Update of the 1978 Alexandria Wastewater Master Plan.

WWCG's contract for review and update of the 1978 Master Plan envisioned relatively quick adoption of a wastewater management plan following publication of the draft review and update report in May 1981. At present, a wastewater management plan has not been selected. Additional information has been developed and analyzed since the draft report was published. A part of that additional information base is represented by the addition of APPENDIX N, ECONOMIC ANALYSIS. This additional work expanded the economic analysis of the sea disposal and land application alternatives. These additional studies have reaffirmed that the two alternatives are nearly equal in total present worth cost terms. In terms other than total present worth costs, the alternatives are quite different. Major differences include operation and maintenance requirements, land requirements, and, most importantly, requirements for initial capital funding.

The delay in selecting a wastewater management plan has rendered schedules in this report outdated. Further delays beyond this writing will further outdate the schedules, resulting in the following two impacts:

- The population of Alexandria, Egypt, will continue to be impacted by the water pollution and public health impacts

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resulting from improper sewage disposal for an additional time equal to the delay (now 5 months and increasing).

- Capital costs of the wastewater management plan that is implemented will be increased an estimated 1 to 1-1/2 percent per month of delay because of inflationary pressure.

CONCLUSIONS

This review and update study was directed by the scope of the contract to investigate both sea and land based systems for the disposal of sewage from Alexandria. In accomplishing this task, efforts were to be made to identify and determine the feasibility of a disposal system which would make beneficial reuse of Alexandria's wastewater through irrigation. A technically feasible land application alternative has been identified. It, along with a sea disposal alternative, represents a sound choice for the solution of wastewater disposition in the Alexandria area.

Based on the information currently available, both of these alternatives appear technically feasible from an engineering standpoint. Both alternative wastewater management plans will functionally meet the principal wastewater mangement goals of safely disposing of Alexandria's wastewater and protecting the recreational beaches. Each will remove Alexandria's wastewater from the immediate area and either dispose of it or reuse it in a fashion recognized as technically adequate and safe. The choice between alternatives may, however, be dictated by capitalization requirements, by land availability, by results of future field investigations, or by some combination of these.

Implementation of either of these alternatives will require considerable additional study prior to undertaking final design. The required predesign work consists mainly of field studies and is outlined in more detail in the recommendations section of this chapter.

Sea disposal is much less capital-intensive than land application. Using LE 0.85 = $1.00 U.S., and end-of-1980 prices as a basis, sea disposal would require expenditures of about LE 510 million, and land application would require expenditures of about LE 886 million, a difference of LE 376 million.

If the impact of inflation is considered, this capital cost difference becomes much greater. For example, assuming an inflation rate of 15 percent on local costs and a variable rate (9 percent in 1981 decreasing to 5.7 percent in 1990) for foreign exchange costs, and an exchange rate related to inflation by the ratio of local cost escalation factor to foreign exchange cost escalation factor, the construction cost for sea

disposal would total LE 1011 million. On a similar basis, the construction cost for land application would total LE 2260 million. The cost differential, expressed in inflated (rather than constant) currency values, would be nearly LE 1250 million. Foreign exchange costs would be $388 million for sea disposal and $385 million for land application.

Based on 1980 prices, sea disposal will require significantly larger net outlays, from an economic viewpoint, for operation and maintenance than will land application. This is due to the income which will be received from the sale of crops produced by the land application system. The magnitude of this cost differential is dependent on the assumptions on which the calculation is based but is significantly large and favors land application regardless of the basis of computation. Using shadow prices as a basis, projected operating and maintenance costs associated with sea disposal for the period 1981 through 2000 total about LE 199 million, while with land application a net revenue (crop returns minus costs) of about LE 60 million would be received. On this basis the differential between the two alternatives is about LE 259 million, favoring land application.

The higher capital costs associated with the land application alternative are mainly due to the costs associated with conveyance of the wastewater to the nearest feasible application site which is located about 60 km south of Alexandria in an undeveloped area of sandy desert soils. All closer sites investigated were found to be unsuited for large-scale land application systems because of conflicts with existing land uses or unfavorable site conditions.

The anticipated cost-effectivness as measured by the total present worth cost of both alternatives, using 1980 prices, is essentially equal. Variations in the discount rate employed in the analysis do not change the near equality significantly. We conclude that, with the information presently at hand, the present worth costs of the two alternatives are not a decisive factor in determining which alternative to implement.

Because of the nearly equal cost-effectivness of the two alternatives, an analysis was made to determine the internal rate of return which might be realized if the additional capital (beyond that required for sea disposal) was spent to implement land application. Using shadow prices as a basis, a base exchange rate of LE 0.70 = $1.00 U.S., and a 50-year time span, this analysis indicated that the internal rate of return, (i.e., the discount rate at which the present worth of benefits received equals the present worth of costs incurred) would be 7 percent. This figure would increase to about 7.7 percent if an exchange rate of LE 0.85 = $1.00 U.S. were used. This finding indicates that land application will be economically advantageous if returns of 7 to 7.7 percent

or less are considered acceptable. If capital is limited, the financial merit of this alternative can be compared to other potential investment opportunities in the national economy of Egypt as a whole. If other investments are typically yielding returns lower than this, land application would be preferred.

We conclude that sea disposal is potentially less flexible than land application and about equal in its ability to effectively deal with safe disposition of Alexandria's wastewater.

A/GOSD and the A/GOSD Board of Directors favor the land application alternative if the questions of capital financing, land availability, and technical feasibility can be satisfactorily answered. They also believe that the option of land application or beneficial reuse of sewage effluent should be retained in any finally adopted long-range plan.

Both alternatives require that toxic industrial wastes be pretreated or excluded from the A/GOSD system. This is a critical need which must be accomplished regardless of the alternative selected.

While both alternatives are considered functionally acceptable, the benefits which would flow from the plans are not equal. The sea disposal plan would provide appropriate treatment and disposition of wastewater into the Mediterranean Sea. The land application alternative would, in addition to appropriate disposition of wastewater, provide other benefits. Principal among these would be the addition of about 54,000 feddans of irrigated land. Implementation of the land application alternative could help Egypt meet the following national goals:

- Self-sufficiency in food production
- Increased productive employment
- Decentralization of the population through the development of new small villages needed to accommodate the treatment and farming operations

Land application could be constructed in a time period approximately 2 years less than for sea disposal; however, needed field investigations which must precede either of the designs would require more time for land application than for sea disposal. Considering needed field studies, predesign, design, and construction as a whole, land application could potentially be operable about 1 year before sea disposal. Utilization of interim treatment and disposal strategies could substantially reduce the importance of such a delay for the sea disposal alternative.

Regardless of the wastewater management alternative selected, a critical need exists for field investigations necessary to further define design criteria related to the disposal elements of either alternative. This predesign related field work may reveal design constraints rendering either alternative more or less functional and more or less costly. Since fewer unknowns appear to be associated with the sea disposal than with the land application alternatives, field studies for sea disposal should require less time.

Necessary field work must span at least one summer season in the case of the outfall studies and at least one growing season in the case of the land application system. Implementation of the land application system will require completion of an environmental assessment and/or an environmental impact statement in order to qualify for USAID funding. USAID has indicated this work could be done concurrently with the other field work so as to not unduly delay design work.

Inflation costs associated with delaying an overall wastewater management system selection and implementation are at least LE 4.7 million per month for the sea disposal alternative and LE 8.5 million per month for the land application alternative. These delay costs are extremely significant and should be minimized to the fullest extent possible.

Further reevaluation of the currently available data and issues which affect the two alternatives is not likely to produce a result indicating clear superiority of one over the other. The basic conclusion, regardless of how the data is analyzed, is quite likely to remain that the salient factors are:

- Sea disposal would require significantly less capital expenditures, would be less flexible, and could require larger net outlays, from an economic viewpoint, for operation and maintenance.

- The validity of the assumptions made in this report concerning the engineering and financial viability of both alternatives cannot be completely confirmed without additional data obtainable only from actual field investigations and related studies. Fewer unknowns appear to be associated with the sea disposal alternative, however.

- Either alternative can effectively and safely dispose of Alexandria's wastewater.

- Delaying a decision to attempt to reanalyze existing information will prove fruitless and inconclusive.

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Delaying a decision to acquire additional information until a long-range wastewater management plan is selected and approved is unnecessary and will result in increased costs ranging from LE 4.7 to 8.5 million per month of delay. In addition, the current public health hazards caused by wastewater in Alexandria will worsen and the earliest possible time for correction will be delayed.

Given the inconclusive results of the analysis of currently available data, the probable social and economic cost of delay, and the need for additional study before definitive design can begin on all parts of either alternative, we conclude that Alexandria's best interests could be served by the adoption of a phased implementation plan.

This plan calls for the immediate identification of the financial and land acquisition constraints associated with the land application alternative. If financial constraints make initial implementation of the land application alternative not feasible, the option of land application and/or wastewater reuse in some form should be retained as a long-range goal in any wastewater disposal plan adopted.

The phased plan should consider maximizing the potential values of expanding the east and west plants in conjunction with full utilization of the natural treatment capacity of Lake Maryout and the replacement and extension of one outfall at the location of the existing Kait Bey outfall to reduce the treated sewage flows being discharged into Lake Maryout and to eliminate untreated sewage being discharged into the sea. The objective would be to provide primary sewage treatment service to the entire Alexandria population at the earliest possible time, while improving the physical and aesthetic qualities of Lake Maryout and the sea.

Significant advantages of the phased implementation plan are: the apparent availability of funds for both the local and foreign costs of the program; early resolution of critical flooding and pollution problems; reduction in pollution of drains, Lake Maryout, and the Mediterranean Sea; early service to outside areas (Abu Qir and Dekheila-Agamy); and improved collection and treatment of sewage flows to the year 2000.

RECOMMENDATIONS

Phased Implementation Plan

We recommend that A/GOSD adopt a phased wastewater management implementation plan consisting of the following:

1. Adopt a program to best provide interim treatment and disposal of Alexandria wastewater flows as soon as possible based on local and

foreign funds available at that time. The planned development should center on the roles the east and the west plants can play in minimizing the pollutional load to Lake Maryout and the sea. Consideration should be given to expanding both plants immediately to "high rate" primary plants to serve as much of the greater Alexandria area as possible. In addition, consideration should be given to supplementing the natural treatment capacity of a portion of Lake Maryout through selected dredging, supplemental aeration, and other means.

2. Conduct marine studies to determine the design data needed to construct the new pipeline at the existing Kait Bey outfall.

3. Identify possible capital financing constraints associated with the land application options. Conduct field studies needed to confirm the technical and economic feasibility of the land disposal system when funding sources have been identified.

4. Adopt land disposal as the goal for disposal of wastewater in Alexandria, and continue to investigate alternate means of financing. When funds are available, divert the sewage flow to land disposal on a phased program. In the event all flows can be diverted to the land, the Kait Bey outfall can be used for stormwater flows and emergency standby. Should difficulties be encountered during the phased implementation program that cannot be resolved with regard to land disposal, A/GOSD should reevaluate other alternatives presented in this report.

5. Continue with ongoing design and construction activities.

Other Recommendations

Additional recommendations as contained in Technical Appendices and in Chapters 1 through 6 of this volume are summarized in tabular form in Table 7-1. Table 7-2 lists the recommended field work associated with the sea disposal alternative. Similarly, Table 7-3 lists the recommended field work associated with the land application alternative.

Table 7-1
SUMMARY OF RECOMMENDATIONS

Recommendation	Report Discussion Location
During the preparation of the basis of design reports, measure actual wastewater quantities to ensure that the facilities will be adequate to meet future needs. Conduct a more extensive sewage sampling and analysis program before final design to determine the concentrations of toxic materials and the effects of treated wastewater and sludge disposal on land or sea.	Chapter 2, p. 2-8 and Appendix C, pp. C-10, 12, and 13
Accelerate a concentrated program to start immediate repairs on sewers needing repair or replacement--especially those in a collapsed or partially collapsed condition.	Chapter 3, p. 3-4
Emphasize those TPP's in the short-term that are critical for the efficient use of the existing collection system including an intense sewer cleaning program, a program for collection and disposal of solid wastes, and implementation of a sewer use law.	Chapter 3, p. 3-16 and Appendix H, pp. H-8 and H-9
Adopt and implement an effective industrial pretreatment program and a noncompatible waste program. Monitor the GOFI pretreatment program closely to ensure that adequate pretreatment of industrial wastes is accomplished prior to discharge into the Alexandria sewer system. Implement an industrial waste sampling program with more and longer sampling in order to adequately assess the pretreatment needs and best pretreatment methods.	Chapter 4, p. 4-5 and Appendix D, p. D-5
Implement an east plant upgrading as soon as possible, utilizing improvements shown in Appendix E. (This recommendation is now under review by A/GOSD with consultation from USAID and WMCG).	Chapter 4, p. 4-5 and Appendix E, pp. E-17 through E-19
All new sewer systems should be designed for sanitary sewers only with separate storm sewers and drainage systems employed for stormwater flows.	Appendix A, p. A-10

Table 7-1 (continued)
SUMMARY OF RECOMMENDATIONS

Recommendation	Report Discussion Location
Conduct a more detailed investigation of industrial wastewater flows for individual basis of design reports.	Appendix D, pp. D-2 through D-3
Implement west plant improvements as listed in Appendix F.	Appendix F, pp. F-9 through F-11
Implement existing pump station improvements as outlined in Appendix G.	Appendix G, pp. G-12 through G-21
Continue TPP projects identified in the 1977 Special Report No. 4 including, but not limited to, solid waste (I-2), sewer use (I-1), and manhole replacement (II-5). Regularly clean the Ibrahimia and Gabbary collectors.	Appendix H, pp. H-5 through H-9
Improve the environmental data base.	Appendix I, pp. I-14 and I-15
Conduct a reconnaissance level archaeological survey prior to final route selection for the land disposal alternative pipelines.	Appendix M, p. M-18

Table 7-2
RECOMMENDED FIELD WORK FOR
THE SEA DISPOSAL ALTERNATIVE

Item	Description
Wind and Wave Studies	Wind and wave data to establish design forces on the outfall and "construction windows"--One full year of continuous data is essential; longer records would be statistically beneficial. Studies funded by the UNDP-UNESCO are expected to provide much of the data required. Supplementary data should be collected at sea by specially installed wave and wind instruments.
Density Profile Studies	Seawater density profiles for summer conditions are definitely required and records for one complete year are preferred.
Current Data	Continuous current information for at least one summer and preferably for one year is required. Two metered arrays supplemented by drogue studies at each outfall location should be provided.
Dye Studies	A series of three dye drops should be made over several days' time during the summer to determine vertical and horizontal eddy diffusion rates.
Bottom Surveys and Bathymetry	The bottom should be resurveyed along each outfall alignment.
Geotechnical Studies	Geotechnical studies along each outfall route, with special studies in areas to be trenched, are required. These studies should include shallow drilling (to 10 m below the mudline) and laboratory testing of samples.
Water Quality Studies	Water column data on nutrients, chlorophyll, turbidity, light transmission, temperature, salinity, pH, and dissolved oxygen should be collected for 1 year.
Sediment Studies	Sediment samples should be collected for particle size analysis, percent volatile solids, BOD, and organic carbon. Settling traps should be used to measure settling rates and particle size distributions.

Table 7-2 (continued)
RECOMMENDED FIELD WORK FOR
THE SEA DISPOSAL ALTERNATIVE

Item	Description
Bacterial Die-off Studies	Measurements should be made of coliform bacteria death rates over time under ambient conditions.
Benthic Animal Studies	Benthic animals should be collected by divers or by grab samples. Collection should be a minimum of four times per year for at least 1 year and at several stations around each outfall site. Samples should be sieved through 1-mm screens and analyzed by accepted scientific methods.
Fish Studies	Fish should be collected in the area by otter trawls. Samples should be analyzed for stomach content, chlorinated hydrocarbons, and trace metals.
Beach Use Studies	A 12-month study of beach use should be undertaken to better establish the need for year-round chlorination of wastewater discharges.
Library Research	Detailed library research should be done to obtain available data on oceanography, marine biology, and water quality in the Alexandria area. This search should include universities and various UN and Egyptian agencies.
Project Delivery Analysis	A parallel study to the field work should be undertaken to develop a detailed project implementation schedule, determine needs for advanced procurement of materials such as a pipe plant, identify materials supply sources, identify qualified and interested construction contractors, and establish construction practices and techniques.
Preliminary Design	Preliminary design activities should refine design criteria and prepare preliminary layouts and budget level capital and O&M estimates based on the results of above studies.

Table 7-3
RECOMMENDED FIELD WORK FOR
THE LAND APPLICATION ALTERNATIVE

Item	Description
Site Surveys	Ground and aerial surveys of the proposed site should be conducted to produce a base map of the proposed site (1:2,500 scale with contours at 0.5 m).
Pipeline Route Surveys	Transmission pipeline routes should be surveyed and investigated, including soundings and bottom sampling of Lake Maryout and marsh crossings.
Geological Investigation	Geological investigations should include a seismic survey (approximately 24,000 m of profiles), and stratigraphic and shallow borings. Stratigraphic borings should be approximately 250 m deep, while shallow borings should extend to approximately 20 m below the existing water table.
Geohydrological Investigations	Geohydrological investigations should determine ground water aquifer properties by shallow aquifer infiltration and recovery tests, and well tests.
Ground Water Model	A mathematical model should be developed for ground water movement and quality studies. This model will form the basis for rapid infiltration system design and environmental impact studies.
Soil Survey	The agronomic and physical properties of surface soils should be surveyed at the proposed site (mapping to 1:25,000 scale).
Crop Trials	A crop trial program at or near the proposed site should be conducted for at least one growing season using water of a quality similar to that expected from the rapid infiltration system. The program should demonstrate responses of various crops to water quality, methods of irrigation, and management practices.
Agronomic Assessment	Detailed cropping patterns and methods should be planned, future yields should be estimated, and irrigation requirements should be identified. In addition, the market potential for crops and required infrastructure and processing facilities should be determined.

Table 7-3 (continued)
RECOMMENDED FIELD WORK FOR
THE LAND APPLICATION ALTERNATIVE

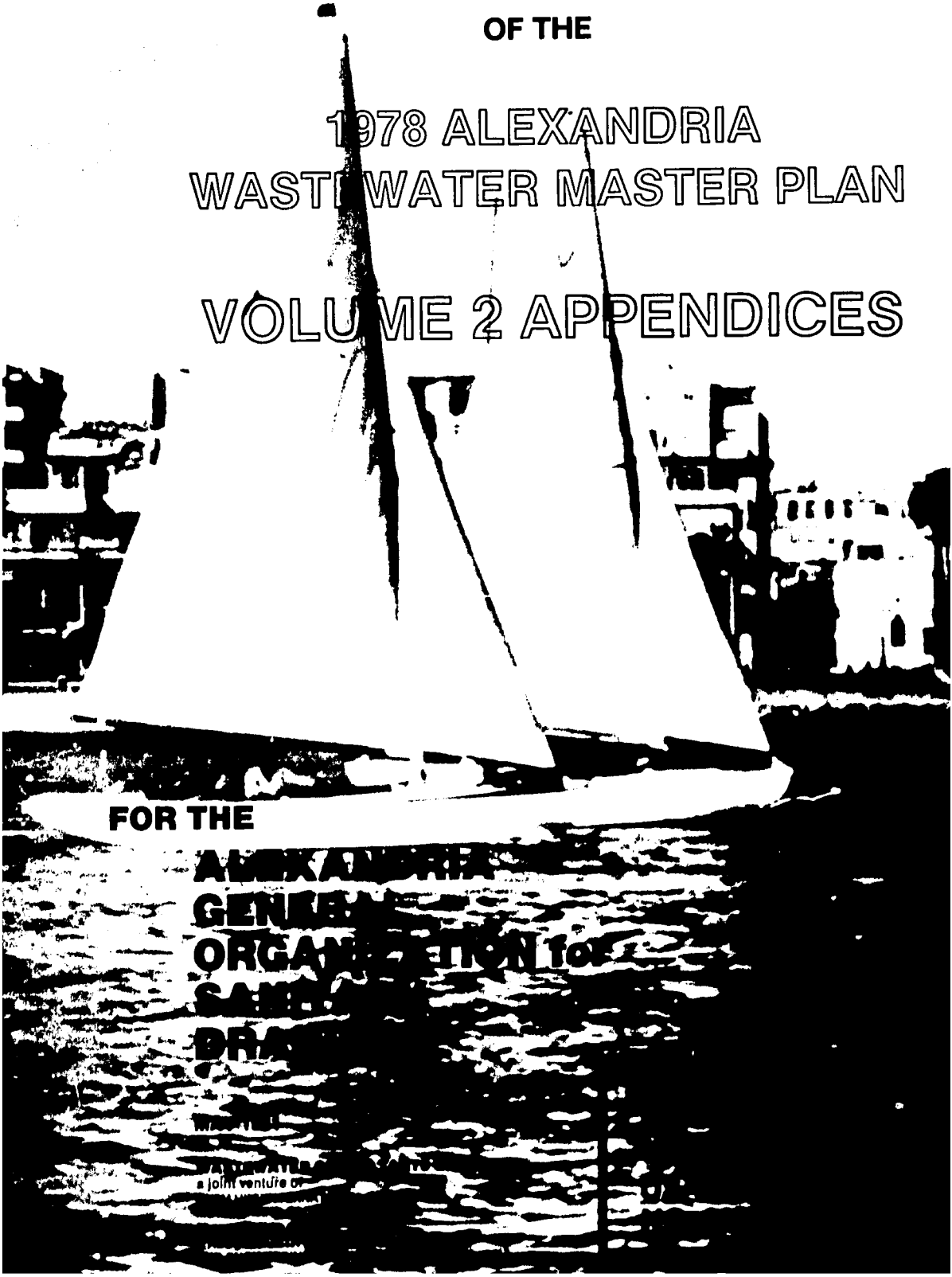
Item	Description
Environmental Assessment	Baseline environmental conditions should be reviewed and the impact of projected developments assessed.
Information Program	An informational program for governmental officials, the academic community, and the interested public should be conducted at various stages during the study. This program should include one workshop at which noted experts on various aspects of land application systems would be invited to make presentations and review progress on the project to date.
Project Delivery Analysis	A study, parallel to the field investigation, should be prepared to develop and present a schedule for advanced materials and equipment procurement and construction of facilities for the conveyance and land application systems. The study should also identify materials and equipment sources, define construction and advanced procurement contracts, identify qualified and interested suppliers and contractors, and establish construction practices and techniques.
Preliminary Design	Design criteria should be refined and preliminary layouts prepared of conveyance system, treatment, rapid infiltration, irrigation, drainage, farm operation, and community facilities. In addition, capital and operating costs should be estimated as well as resource and manpower requirements.

REVIEW AND UPDATE

OF THE

1978 ALEXANDRIA WASTEWATER MASTER PLAN

VOLUME 2 APPENDICES



FOR THE

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APPENDIX A

UPDATED MASTER PLAN CRITERIA

This appendix presents the general rationale and criteria used in the preparation of this updated Master Plan Report for Alexandria. These have been formulated through review of the 1978 Master Plan and studies of Egyptian, international, and United States practices relevant to wastewater facilities.

DESIGN PERIODS

Each element in a wastewater system is designed for a specific period, throughout which it is expected to serve the needs of that system. Design periods vary with the type of element and depend upon many factors, including resistance to deterioration, construction and operating costs, flexibility of performance under various conditions, and rate of obsolescence. Design periods recommended for the Alexandria facilities are shown in Table A-1. In general, these design periods are the same as the estimated service lives of facilities discussed in Appendix B.

Generally, the sizing of elements is based on the anticipated capacity required through the design year. Technical and economic considerations may warrant the installation of a smaller element initially, to be followed as the loading increases by a second or even additional elements. Such items as large diameter pipelines and force mains, require evaluation on a case-by-case basis.

WASTEWATER FLOWS

Domestic Wastewater

Domestic wastewater flows as projected in the 1978 Master Plan were estimated from a combination of data including the then present and anticipated future water consumption rates, load use projections, and anticipated population densities. This data and methodology resulted in the estimated per capita domestic wastewater flows shown in Table A-2. Study of anticipated future domestic and governmental water use provided the results shown in Table A-3. Comparison of Tables A-2 and A-3 is complicated by the fact that the water supply records and the sewerage records use different geographical subdivisions. In spite of this, the facts indicate that anticipated per capita domestic sewage flows computed on the basis of anticipated future water use are significantly

higher than those used in the 1978 Master Plan. These differences are illustrated in Table A-4. The accuracy of future water demand projections cannot be verified in this study nor can current sewage flows be accurately known without actual measurement. The water use projections indicate a combined domestic and governmental water use of 196 lcd during the peak tourist season by the year 2000. Water use of this magnitude in other developed countries is common; therefore, we see no apparent reason to doubt the possibility that water use in Alexandria may reach 196 lcd by the year 2000. If this occurs, we believe the per capita sewage flow allowances listed in Table A-2 will be too low. Consequently, we do not recommend their use for design purposes. However, since their accuracy cannot be verified without field measurement, and since it is necessary to compare sewerage alternatives developed in this Master Plan Review with those recommended in the 1978 Master Plan on a consistent basis, we determined it was appropriate to use the 1978 figures for comparison purposes only. We strongly recommend that a program of flow measurement and water use investigation be added as a task to be completed during the preparation of all bases of design reports (BODR's) preceding actual design of sewerage facilities.

For the purposes of developing sewerage alternatives for comparison with the 1978 Master Plan, the per capita sewage flows listed in Table A-2 were used. These flows along with new population projections (see Appendix C), industrial sewage flow projections (see Appendix D), infiltration allowances based on Figure A-1, and new area measurements and peaking factors based on Figure A-2 formed the basis for the specific flows by sewer area, which are shown in Appendix C.

Industrial Wastewater

Industrial flows were taken from the updated industrial information as presented in Appendix D of this report.

Infiltration

Extensive studies undertaken in the United States by the consultants indicate that infiltration flow as a percentage of total flow in a sewer system will decrease with an increase in tributary areas. These studies also indicate that sewers constructed with gasketed compression joints have an appreciably lower infiltration rate than older sewers, which were constructed with hemp-mortar or hemp-tar joints. Based on these observations, design peak infiltration curves as indicated in Figure A-1 were developed. It is recommended that these curves be used for the design of sewers in Alexandria, with the older sewer curve applicable to any new construction that employs hemp and mortar joints.

In the event that sewers with gasketed compression joints become available in Egypt, then the new sewer curve should be used in determining infiltration rates. Infiltration analyses in other parts of Egypt indicate that in areas where the condition of sewers is poor and the ground water level high, the infiltration rate may be higher than the infiltration rate given by the old sewer curve. However, where such conditions are known to exist, it would be more cost effective to provide a rehabilitation program rather than provide capacity for the excessive infiltration rates.

Peaking Factors and Minimum Flows

Our analysis of the peaking factor curve, Figure A-2, which was used in the preparation of the 1978 Master Plan, indicates that the peaking factors are adequate even if a substantial quantity of the infiltration into the system is eliminated. The consultants therefore adopted Figure A-2 for use in updating the Master Plan.

The minimum flow through a sewer system for cities of Alexandria's size is commonly found to be one-half of the average flow. Because of the potential for excessive infiltration, the ratio of minimum flow to average flow may be larger than 0.5.

WASTEWATER COLLECTION SYSTEM

Sewers

Sizes. The minimum size of sanitary sewers should be greater than the size of the building connection in general use. This is desirable because the sewer should be capable of passing all solid materials that flow through the building connections. Current practice in Alexandria follows this recommendation; building connections are generally 15 cm, and the minimum allowable street sewer size is 20 cm.

Hydraulics. Sewer carrying capacity will be determined using the Manning formula with "n" equal to 0.013 and the pipe flowing full at the design peak flow rate. Flow velocities in sewers should ideally be sufficient to prevent the deposition of solid matter. In flat areas, providing slopes that are sufficiently steep is difficult and costly. Failure to provide such slopes in Alexandria, however, can cause difficult and expensive maintenance problems because such sewers are frequently blocked by sand, stones, and other foreign matter. Such blockages are known to occur in developing areas, where flows are minimal and loose building materials or suspended solids from dewatering operations are liable to enter. Blockages and sluggish flow have a further detrimental effect in that the wastewater can turn septic and

allow the formation of corrosive and foul-smelling hydrogen sulfide. In order to prevent deposition from normal waste flows and to reduce the incidence of blockage, sewers should be laid at slopes that ensure velocities capable of transporting sand particles up to a size of approximately 0.2 mm. Table A-5 presents the recommended minimum slopes and resultant maximum and minimum velocities for commercially available Egyptian standard pipe sizes. Also presented in Table A-5 are the absolute minimum slopes that should be used only when the recommended slopes are unattainable. It should be realized that these flatter slopes imply greater operating and maintenance costs.

Sewer Construction Materials. Present sewer construction practice in Alexandria utilizes locally produced bell and spigot vitrified clay pipe in sizes up to 600 mm. For larger sewers, locally manufactured glass reinforced pipe is used. Construction experience has indicated that locally manufactured vitrified clay pipe is unsatisfactory for several reasons:

- Pipe strength and dimensional characteristics are not uniform.
- Pipe may be porous to such an extent that ground water may infiltrate the pipe itself.
- Pipe is often out-of-round, making jointing difficult.
- Internal pipe surfaces may be rough, increasing friction losses.
- Pipe may not be true in linear dimensions because of warping.

For these reasons, A/GOSD, with the assistance of Boyle Engineering and WWCG, is exploring the following alternatives:

- Spigot-spigot vitrified clay pipe (from local or U.S. sources) with a collar type joint (from the U.S.).
- Polyvinyl chloride (PVC) pipe with compression type joint, or "O" ring (from U.S.).
- Reinforced concrete sewer pipes from local or foreign sources.

Recommendations for sewer construction materials will be made based on the findings of this investigation.

Manholes. Manholes should be constructed at changes in slope, size of pipe, or direction, and at all sewer junctions. The recommended maximum spacing of manholes is shown in Table A-6.

The manholes should be 122 cm in diameter with a minimum clear opening of 60 cm to provide adequate access for maintenance purposes. Construction should be from precast sections, equipped with tongue-and-groove joints with a rubber gasket or a waterstop joint, and with cast-in-place manhole bases. Manholes should be equipped with drop connections where elevation differences between connecting sewers are sufficient to permit the use of standard fittings.

Pump Stations and Force Mains

The following criteria are recommended for pump stations:

- The firm capacity of the pump station, i.e., the capacity assuming the two largest pumps are out of service, must match the peak sewage flow rate.
- The number of pumping units installed should be equal to the number required for peak flow plus two standby pumps.
- Pumps should be sized so that excessive flow fluctuations do not occur at sensitive downstream facilities, such as treatment plants. Alternatively,variable speed pumps would be required.
- Pump stations should be provided with bar screens to protect the pump and force mains from blockage and should also have a flow measuring device.
- Force mains should be designed to maintain self-cleaning velocities with a normal maximum velocity of approximately 2.0 metres per second (m/s) and a minimum velocity of 0.6 m/s. In cases where other factors such as economics, pump selection, or maintenance of minimum velocity dictate the use of higher maximum velocities, an upper limit of 3.0 m/s may be utilized. Force mains should be sized to accommodate the peak and minimum sewage flow rates deliverable by their associated pump stations within the velocity range indicated above. Force mains should be constructed of cast-iron or ductile iron pipe with suitable thrust blocks provided at changes of alignment.

Pump stations should be of wet-dry well type. Wet wells should be divided into two compartments to facilitate cleaning and maintenance.

Maximum design water level in a wet well should be below the invert level of incoming sewers. Low water level should not be below the pump casing level. Wet wells should be sized so that the interval between pump starts is not so short as to damage associated electrical equipment, nor so long as to permit stagnation of wastewater. Dual power sources or standby power should be provided at pump stations where loss of power would otherwise cause flooding.

Pumps should be of a nonclog type capable of handling solids up to 75-mm spheres. Generally, they should be of the vertical shaft type, driven by electric motors set above possible flood level. Water seals are recommended. Dry wells should be equipped with automatic sump pumps and standby hand pumps.

WASTEWATER TREATMENT

Process Selection

Wastewater treatment facilities should be designed to produce an effluent that will not have an adverse impact on receiving water or lands. In the absence of definitive and complete effluent quality standards, we are proposing various degrees of treatment depending on the method of disposal. Table A-7 indicates the treatment facilities recommended for alternative methods of disposal. Since the quality of effluent needed for most industrial purposes would require sophisticated and costly treatment processes, we have not included industrial reuse.

Views on appropriate treatment technology are changing rapidly. Sophisticated activated sludge and physical-chemical processes, devised to conserve land and produce a consistently high quality effluent even in cold climates, are being viewed less favorably than more traditional processes that are less energy and operation intensive. This is particularly true in developing countries with warm, arid climates where simpler treatment systems such as stabilization ponds are not plagued by cold weather inefficiency, which often precludes their use in colder climates, and where sufficient land is usually obtainable.

Likewise, the reuse of treated wastewater for agricultural production is increasingly encouraged in recognition of the plant nutrients it contains; thus it is viewed as a resource to be used rather than a nuisance to be disposed of.

All the treatment processes proposed in this study are capable of producing an acceptable effluent with a minimum of mechanization and do not require sophisticated operation and maintenance procedures. The specific detailed design criteria for the unit processes involved in these treatment systems are presented in Appendixes J and K.

Reliability and Flexibility

In order to ensure that wastewater treatment plants can provide a consistently acceptable effluent, the following design standards are recommended:

- For treatment plants receiving total flow from pump stations the design peak flow should be equal to the sum of all the firm capacities of the pump stations separately discharging to the plant.

- For each unit process, with the largest flow capacity unit out of service, the remaining units should be capable of handling the design peak flow. Accordingly, bypassing any unit process will not be required.

- Each unit process should consist of a minimum of two units, including bar screens, aerated grit chambers, and chlorine contact tanks.

- Two backup pumps and blowers should be provided for each set of pumps and blowers that serve the same function. The capacity of the pumps and blowers will be such that with any two units out of service, the remaining units will have capacity to provide for peak flows.

- Two sources of electric power should be provided at each plant, with each source of power transformed to usable voltage through a separate transformer.

- An emergency generator should be provided with sufficient capacity to provide for emergency lighting, ventilation, chlorination and alarm systems.

Sludge Management

Sludge management includes the treatment, transportation, and disposition of sludge solids resulting from wastewater treatment. Treatment processes proposed herein are those that are consistent with the wastewater treatment systems earlier discussed. They are capable of reliably producing sludge characteristics suitable for agricultural use, with a minimum of complex mechanization or sophisticated operation and maintenance procedures. The recommended processes do not require large amounts of energy or chemical additives.

Transportation processes proposed include both truck and pipeline transport and are dependent on the location of sludge use and sludge treatment, as well as the sludge moisture content.

The specific detailed design criteria for the unit processes involved in sludge management associated with wastewater treatment systems are presented in Appendix J. Table A-8 indicates the sludge treatment processes recommended for use with various wastewater treatment alternatives.

STORMWATER DRAINAGE FACILITIES

Quantity of Stormwater

Stormwater quantities were computed using the rational method in which the quantity of runoff is related to rainfall intensity by the formula:

$$Q = 0.24CIA$$

where:
- Q = peak runoff rate in megalitres per day (Ml/d)
- C = runoff coefficient
- I = design rainfall intensity in millimetres per hour (mm/hr)
- A = drainage area in hectares (ha)

Runoff coefficients used were based on the zone principle of estimating runoff for a rectangular area. These coefficients are affected by the percent of impervious surface area and the duration of the rainfall. Table A-9 shows the runoff coefficients used.

Rainfall Intensity and Duration

Curves indicating the relationships between rainfall intensity and duration for storm frequencies of 2, 3, and 5 years are presented in Figure A-3. These curves were based on data taken from the report, "Alexandria, Egypt--Rainfall Characteristics (January 1979)."

Design Rainfall Frequency

The appropriate design storm frequency for any storm drainage system is a function of numerous factors. Economic justification is usually the most significant factor and, for the majority of systems, the cost should not exceed the expected benefits over the life of the system. The March 1979 report, "Alexandria Sewerage Top Priority

Projects--Proposed Storm Drain Design Criteria," proposed that a 5-year storm frequency be employed in high value residential and commercial property areas, and a 2-year storm frequency be employed in average value residential areas and industrial areas. We generally concur with these selections and recommend a 5-year storm frequency for areas with a high potential for property damage or disruption to transportation and communication systems associated with stormwater runoff. For less sensitive areas and areas where future drainage facilities are planned, we recommend a 2-year frequency.

Inlet Time and Time of Concentration

Inlet time is the time required for stormwater falling on a surface to reach established surface drainage channels (such as a street gutter), and to travel through surface channels to the nearest inlet point of the drainage facilities. Inlet times vary with ground slope, surface characteristics, and distance of travel to a surface channel. Inlet time may be computed by the formula:

$$Ti = \frac{L^{0.30}}{C^{0.70} S^{0.15}}$$

where
- Ti = inlet time in minutes
- L = horizontal distance to inlet in metres
- S = average slope of surface in metres/metre
- C = average "C" factor of contributing area

"C" varies from about 0.17 for a dense grass surface to about 0.85 for smooth paved surface.

The time of concentration is the sum of the inlet time plus the time of flow in the storm sewer to reach the point under consideration. Time of flow is computed by calculating the distance travelled in the storm sewer to a selected point and dividing it by the average full pipe velocity using the Manning equation to determine full pipe volume and, for a given pipe cross-sectional area, flow velocity.

The inlet time in Alexandria is influenced by the permeability of the soils, the slope of the topography, and the storage available in the catchment area. Our analysis indicates that the storage in many catchment areas is high because of the inability of stormwater runoff to reach paved areas and the storage available in paved areas. For this reason, we recommend that the inlet times shown in Table A-9 be used for design purposes.

Storm Drain Design

The hydraulic analysis of storm drains will be based on the appropriate storm frequency, and pipe sizes and slopes will be selected based on Manning's formula. Manning "n" values of 0.013 for new circular conduits, 0.015 for old sewers; 0.012 for new box culverts; 0.015 for new, concrete-lined, float-finished open channels; and 0.025 for new unlined channels will be used. For new closed conduits, pipe sizes and slopes will be selected to achieve design flow velocities between 0.7 and 3.0 m/s. Open channels, which are easier to clean, will be designed to achieve velocities in the range of 0.9 to 2.5 m/s.

Street inlets will preferably be located as necessary to prevent surface channel flows in excess of 100 m and will not be trapped. The circuitous nature of many of the existing street layouts may, for economic reasons, preclude provision of inlets in all streets. Manhole spacing for storm drains up to 1200 mm in diameter will not exceed 100 m and for larger pipe sizes will not exceed 150 m.

A minimum pipe size of 300 mm will be used between street inlets and manholes and a minimum pipe size of 375 mm will be used between manholes.

Pipe materials will be vitrified clay for sizes up to 600 mm and reinforced concrete pipe for larger sizes.

COMBINED SEWER SYSTEMS

The 1978 Master Plan recommended that new sewer systems for the Alexandria area be designed for sanitary sewage only and that separate storm sewers and drainage systems be employed to control stormwater flows. WWCG concurs with this recommendation and proposes that all new lateral sewer systems be designed as separate sewers for collection of wastewater only.

Table A-1
RECOMMENDED DESIGN PERIODS FOR
WASTEWATER FACILITIES

Element	Design Period (yr.)
Gravity Sanitary Sewers	
Branch	50
Interceptors	20
Interceptors[a]	50
Storm Sewers	50
Force Mains	20-30
Submarine Outfalls	30-50
Pump Stations	
Structures	40-50
Equipment	10
Treatment Plant	
Process Structures	20-30
Other Structures	10-20
Process Equipment	10-15
Other Equipment	6-10
Instrumentation & Control Equipment	10-15

[a]To be used where physical limitations will not permit the construction of a parallel facility at a later date.

Table A-2
1973 MASTER PLAN ESTIMATED DOMESTIC WASTEWATER FLOWS
(lcd)

Sewerage Region	Year 2000	Year 2030
Eastern Zone	130	161
Central Zone	136	164
Western Zone	125	150
Other Areas		
Abu Qir	108	133
Mex-Dekheila	103	127
Nouzha	100	133
Ras El-Soda	107	133
Sadat City	108	133
Slouf Kebila	108	133
Amria	136	---

Source: 1978 Master Plan--Sanitary Sewer Design Criteria Report, February 1979

Note: Figures include allowance for commercial, government, public, and dry industries wastewater flows.

Table A-3

YEAR 2000 GEOGRAPHICAL DISTRIBUTION OF WATER DEMAND FOR DOMESTIC AND GOVERNMENTAL USE

Water Regions	Permanent Population (1000's)	Seasonal Population (1000's)	Total Population (1000's)	Total Dom. and Gov't Water Use (Ml/d)	Per Capita Dom.&Gov't Water Use (lcd)	Per Capita Sewage Based on 80% Return (lcd)
Eastern Zone[a]						
Ramleh	705	54	759	175	230	184
Montazah	841	214	1,055	256	243	194
Sadat City	500	---	500	91	182	146
Subtotal	2,046	268	2,314	522		
Average					226	181
Central Zone[b]						
Manshiya	56	--	56	9	161	129
Attarine	93	--	93	17	183	146
Moharrem Bey	369	--	369	58	157	126
Bab Sharkay	530	173	703	153	218	174
Subtotal	1,048	173	1,221	237		
Average					194	155
Western Zone						
Gomrok	170	--	170	26	153	122
Labban	112	--	112	18	161	129
Karmouz	242	--	242	30	124	99
Minyet El-Basal	370	--	370	42	114	91
Subtotal	894		894	116		
Average		--			130	104
Dekhella	137	155	292	56	192	154
Amria	445	--	445	81	182	146

Source: 1978 Master Plan, Vol. III, Appendix B

[a]The eastern zone water distribution area includes Abu Qir, Ras El-Soda, and Siouf Kebila whereas the eastern zone sewered area (Table A-2 and A-4) does not.

[b]The central zone water distribution area includes Nouzha whereas the central zone sewered area (Tables A-2 and A-4) does not.

Table A-4
COMPARISON OF DOMESTIC SEWAGE FLOWS
(1978 MASTER PLAN) AND FLOWS BASED ON AN
80% RETURN OF PROJECTED WATER USE FOR THE YEAR 2000
(lcd)

Sewerage Region	1978 Master Plan Allowance	80% Return of Projected Water Use
Eastern Zone	130	201
Central Zone	136	158
Western Zone	125	104
Abu Qir	108	218
Mex-Dekhella	103	154
Nouzha	100	97
Ras El-Soda	107	178
Sadat City	108	146
Siouf Kebila	108	156
Amria	136	146
Average, all regions	125	157

Table A-5
MINIMUM SEWER SLOPES

Sewer Diam. (cm)	Recommended Min. Slope (m/m)	Velocity of Flow[a] Full Flow (m/s)	Velocity of Flow[a] 25% Flow (m/s)	Absolute Min. Slope (m/m)	Velocity of Flow[b] Full Flow (m/s)	Velocity of Flow[b] 35% Flow (m/s)
20	0.0053	0.75	0.52	0.0040	0.65	0.51
25	0.0037	0.74	0.50	0.0030	0.66	0.51
30	0.0032	0.77	0.53	0.0022	0.64	0.49
37.5	0.0025	0.79	0.54	0.0015	0.61	0.47
45	0.0021	0.82	0.56	0.0012	0.62	0.48
50	0.0018	0.81	0.56	0.0010	0.61	0.47
60	0.0017	0.89	0.61	0.0009	0.66	0.50
75	0.0014	0.94	0.65	0.0008	0.71	0.55
90	0.0012	0.99	0.67	0.0008	0.81	0.61
105	0.0010	1.00	0.68	0.0008	0.89	0.69

[a]Based on Manning's formula, n = 0.013. Minimum velocity computed based on 10% of peak instantaneous flow, which is equivalent to a flow depth of 25% of pipe diameter for pipes laid at the recommended minimum slope.

[b]Based on Manning's formula, n = 0.013. Minimum velocity computed based on 20% of peak instantaneous flow, which is equivalent to a flow depth of 35% of pipe diameter for pipes laid at the absolute minimum slope.

Table A-6
RECOMMENDED MANHOLE SPACING

Sewer Diameter (cm)	Maximum Spacing (m)
20 and 25	40
30 and 37.5	50
45 and 60	60
60, 75, 90 and 105	60
122 cm and over	60

Table A-7
PROPOSED WASTEWATER TREATMENT FOR
INTENDED RECEIVING WATERS OR EFFLUENT USE

Method of Disposal	Proposed Treatment Plan
Restricted irrigation[a]	Anaerobic ponds--surface irrigation
Irrigation[b]	Anaerobic ponds--rapid infiltration
Drain	Activated sludge, trickling filters, or stabilization ponds
Sea	Primary treatment

[a]Use of effluent from this process would be restricted for use on crops such as forage, seed, and fiber crops or crops that would require processing to destroy all pathogenic organisms.

[b]Use of effluent from these processes would not have the above restrictions due to the high degree of treatment.

Table A-8
PROPOSED SLUDGE TREATMENT PROCESSES
TO PRODUCE SLUDGE SUITABLE FOR AGRICULTURAL USE

Wastewater Treatment Method	Type of Sludge	Proposed Sludge Treatment Process
Anaerobic Lagoons	Stabilized Primary	Air dewatering in out-of-service lagoon plus storage after removal; truck transport of dewatered sludge to agricultural use
Primary Treatment	Raw Primary	Vacuum filter dewatering plus windrow method of composting; truck transport of finished compost to agricultural use
		or
Secondary Treatment	Raw Blended Primary and Secondary	Air drying on sand drying beds plus 1- to 2-year storage; truck transport of dried sludge to agricultural use; truck or pipeline transport of liquid sludge to drying beds

Table A-9
RUNOFF COEFFICIENTS AND INLET TIMES

Runoff Coefficients

Time Minutes	Pervious Surface (10% Impervious)	Semipervious Surface (50% Impervious)	Impervious Surface (90% Impervious)
10	.211	.370	.528
20	.266	.429	.592
30	.305	.472	.636
45	.351	.519	.688
60	.388	.559	.729
75	.42	.592	.764
90	.448	.621	.795
105	.477	.642	.807
120	.492	.653	.814
135	.505	.663	.821
150	.517	.672	.828
180	.539	.690	.840
240	.576	.720	.863
300	.607	.745	.883
360	.634	.768	.902

Inlet Times

Population Density	Permeability	Slope	Inlet Time (min)
High	Low	Moderate to High	10-15
Medium	Medium	Moderate	15-20
Low	High	Moderate	>20

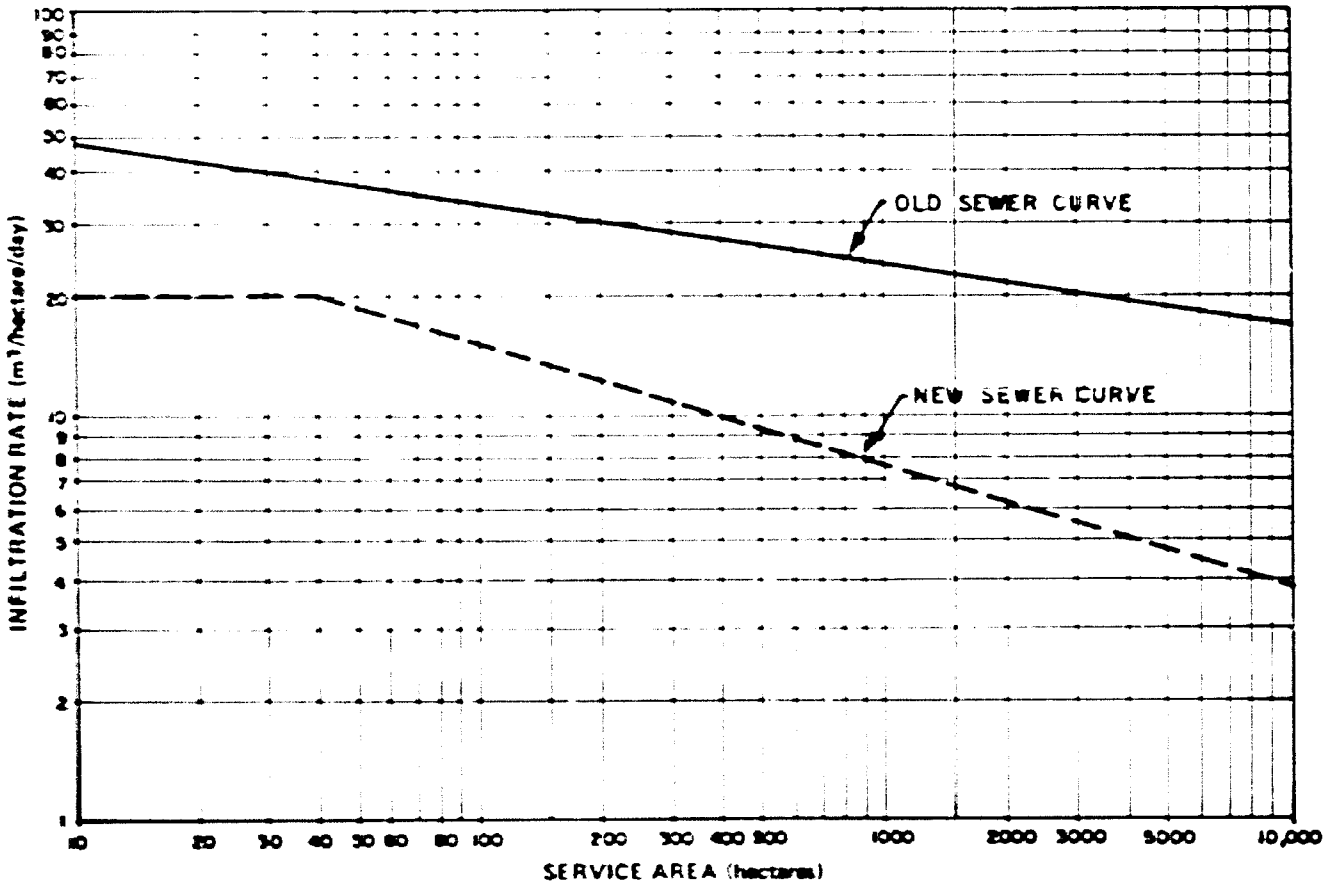

FIGURE A-1

PEAK INFILTRATION RATES

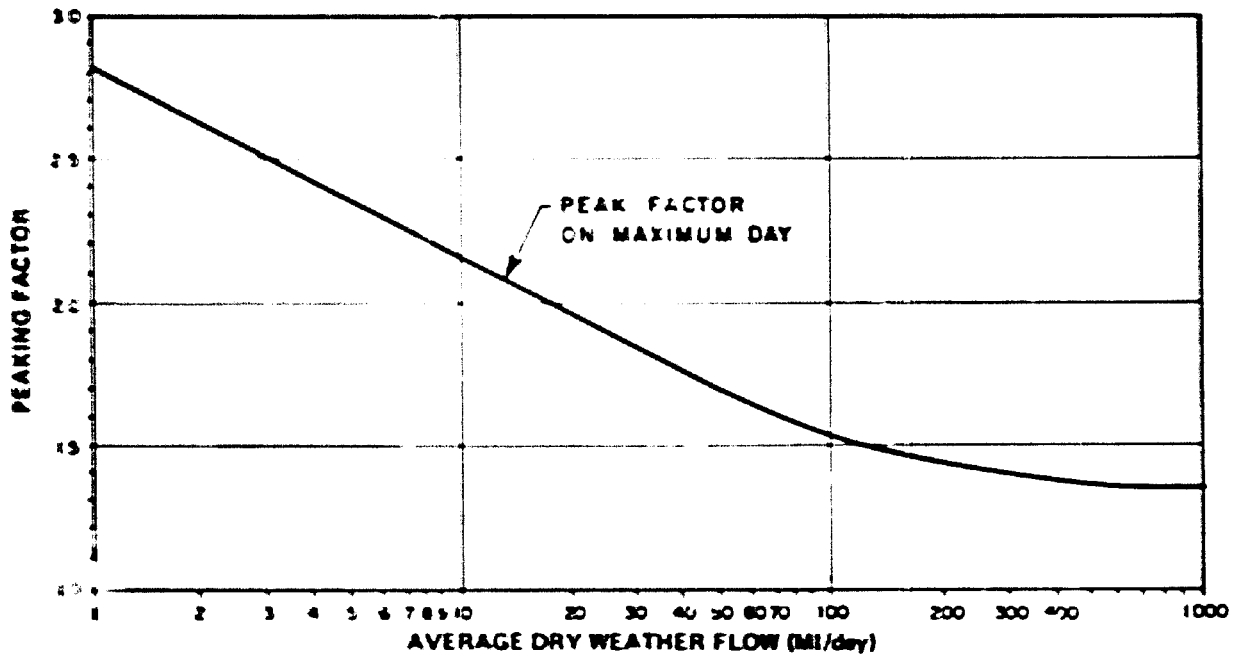

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FIGURE A-2

PEAK FACTOR CURVE

A/GOSD WWCG W/ECG

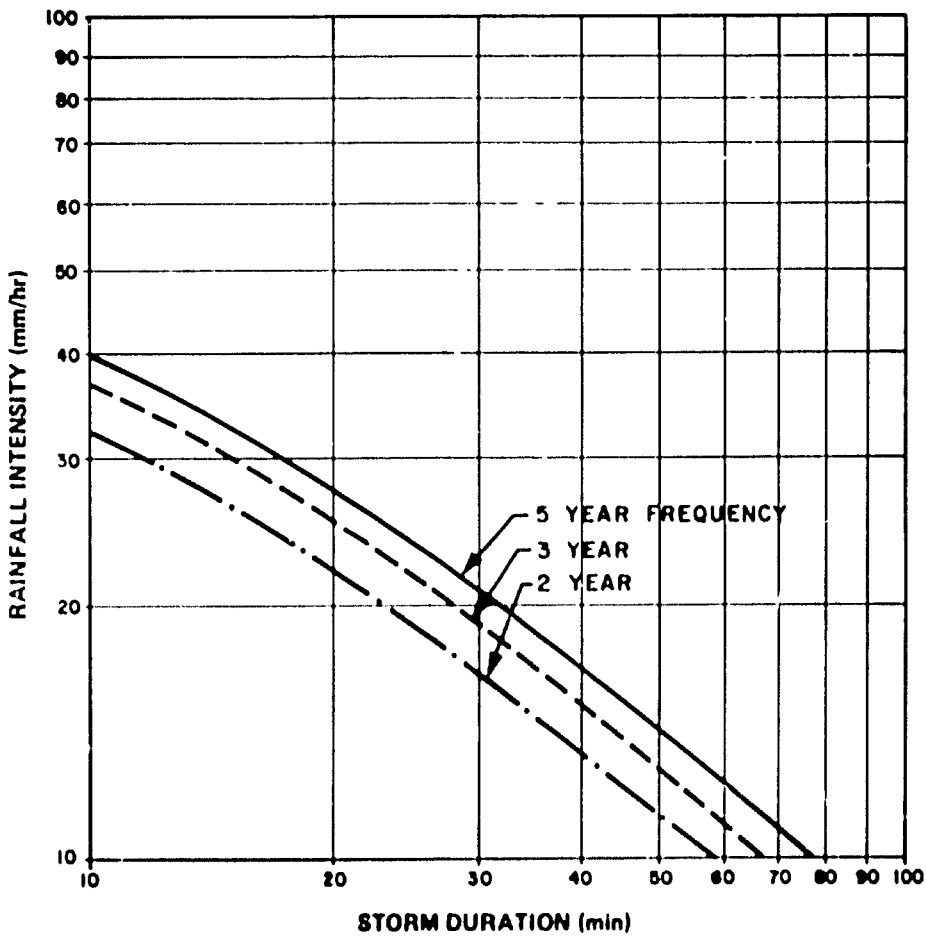

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FIGURE A-3

RAINFALL INTENSITY-DURATION

A/GOBD WWCG W/ECG

APPENDIX B

COST DATA BASE UPDATE

Due to the magnitude of the project and variety of components, project costs for this Master Plan Review were developed utilizing a number of conceptual, budget, and definitive level estimating techniques. In all instances final costs are specific to Egyptian construction methods and materials. Foreign costs were utilized only for materials that are not currently available in Egypt.

Multiple resources were used in developing project costs; these included visits to appropriate ministries, discussions with local contractors and material suppliers, visits to local construction sites, and reviews of current bid tenders and previous reports submitted on this and other appropriate projects.

Estimates are conceptual or planning level and are based on the design information outlined in other portions of this report. Costs presented herein were only developed for purposes of comparison and should not be construed to have definitive level accuracy.

DATA BASE DEVELOPMENT

In order to effectively develop cost estimates for the various project components, a data base had to be established. This base includes labor and material costs as well as in-place unit costs for common construction items. The three tables following the text of this appendix list the basic costs used as the foundation of the cost data base.

A significant effort was made to recognize the rapidly advancing construction techniques used by local contractors and to avoid escalating costs based on outdated construction practices.

The labor, equipment, and material costs shown in the tables were researched and basic unit costs were developed from this data. Labor and material costs (Tables B-1 and B-2), do not include any markups; however, the in-place unit costs (Table B-3) reflect appropriate taxes, fringe benefits, and contractors' overheads and profits.

In those instances where cost curves based on U.S. construction experience were used, they were specifically adjusted by type of work (e.g., earthwork, concrete, process equipment) to reflect local costs. Additionally, curves were checked for variations in the cost intensive items, and more discrete techniques were used to verify their accuracy under local conditions.

To facilitate this cost estimating, curves were developed for major project components. Where appropriate, these figures are included following the tables in this appendix. The basic assumptions for each curve are listed on the figure. All other costs were developed on a case-specific basis for the sizes and capacities required.

All costs were developed on an end-of-1980 cost base. An exchange rate of 0.70 was used when necessary to convert U.S. values to Egyptian values.

COST ANALYSIS TECHNIQUES

In order to accurately assess each alternative, consideration must be given not only to the capital cost of construction but also to the cost of operating and maintaining the facility. Additionally, the effects of time on the value of money must be considered in the economic analysis of the project.

Operation and Maintenance Costs

Operation and maintenance costs were prepared using two sources. Where possible, local power, fuel, material, and labor requirements were calculated. This data was supplemented by published requirements for appropriate unit processes and facilities developed from experience with a number of operating plants in the United States. In each case the estimated quantities for power, fuel, material, and labor were based on local conditions. Where published U.S. experience was used as a quantity base, the quantities were adjusted to local conditions using factors appropriate for the specific item and process.

The estimated quantities were extended by local 1980 unit prices and summed for a total annual cost for each project component (e.g., pump station, force main, treatment process). These techniques will permit future adjustments if required by changes in the project definition.

Estimates of the operation and maintenance costs were made for current, 1990, and design year conditions, with consideration given to the impact of phased construction. Annual costs for the intervening years, 1982-1989 and 1991-1999, were estimated by straight-line interpolation. End-of-1980 values were used for all unit prices. Shipping is included as part of the cost of chemicals that must be imported.

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Present Worth Analysis

An accurate review of the real cost of the various project alternatives requires an analysis accounting for the time value of money as well as construction, operating, and maintenance costs. In a present worth analysis, all future costs are reduced to their present worth and summed for each alternative. The base discount rate used for this project was 6 percent.

For purposes of this analysis, inflation was assumed to affect all prices equally and is therefore not addressed.

The following service lives were used in the analysis:

Type of Component	Service Life (years)
Gravity Sanitary Sewers	
Branch	50
Interceptors	20
Interceptors[1]	50
Storm Sewers	50
Force Mains	20-30
Submarine Outfalls	50
Pump Stations	
Structures	40-50
Equipment	10
Treatment Plant	
Process Structures	20-30
Other Structures	10-20
Process Equipment	10-15
Other Equipment	6-10
Instrumentation & Control Equipment	10-15

[1]To be used where physical limitations will not permit the construction of a parallel facility at a later date.

Salvage values for equipment and structures were estimated on the basis of straight-line depreciation during the service life of the component. Land was taken at 100 percent of its value.

The salvage value associated with a project or portion thereof is the remaining value that project or portion thereof provides by virtue of the fact that it can continue to serve in its same role for some time period beyond the analysis timeframe. This is not to be confused with

and is different from the value of a useful commodity remaining which could be sold at the end of the analysis.

The design year used in the analysis of alternatives was 2000. In those instances where phased construction is contemplated, appropriate adjustments were made.

Annual costs for the land application alternatives reflect the return on investment realized from harvesting crops. Treatment alternatives and sea disposal alternatives do not result in any monetary return.

BASIC ASSUMPTIONS

Specific assumptions for the project components are included with each conceptual estimate. However, in order to equitably evaluate the various alternatives, a set of basic assumptions was established. The following assumptions were used throughout as appropriate.

Assumptions for Capital Costs

- Costs do not include escalation. All costs are end-of-1980 values.
- Customs, duties, import taxes are excluded from the capital costs.
- Costs are segregated into Egyptian and U.S. costs as follows:

Egyptian	U.S.
Site and earthwork	Process equipment
Concrete	Process mechanical systems
Masonry	Instrumentation & control
Metals	Hoisting and conveying equipment
Wood and plastic (as available)	Conveying equipment
Thermal and moisture protection	Process electrical systems
Doors and windows	
Painting and finishes	
Special interior fixtures	
Furnishing	

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Plumbing
Light HVAC
Lighting

- Allowable contingencies are as follows:

  - Sewer work 25%
  - Pump stations 25%
  - Force mains 20%
  - Treatment plants 25%
  - Transmission mains 20%
  - Sea portion of outfalls 40%

- Costs are at an order-of-magnitude level with an expected accuracy of +30 to -50 percent (i.e., the actual cost may be expected to be in the range of 50 percent greater to 30 percent less than the estimate).

- An exchange rate of LE 0.70 = $1.00 is used.

- Shipping costs are included for all U.S. imported equipment and material.

- Labor and materials required will be available at the time of construction.

- An adequate power supply will be brought to the site by others.

Assumptions for Present Worth Analysis

- Interest rate is 6 percent.

- Capital recovery period is through the year 2000.

- Land application alternatives include return on investment for crop yield.

- Straight-line depreciation is used for equipment and structure salvage values.

- Land salvage value is taken as 100%.

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Assumptions for Operation and Maintenance Costs

- Local labor, power, chemical costs are used.
- Shipping is considered as part of the replacement cost of imported items.
- Annual maintenance costs for equipment and structures are taken as a percent of their capital value.
- Pipeline maintenance includes routine cleaning as well as repair and replacement.

CONCEPTUAL ESTIMATES

Conceptual level estimates as defined by the American Association of Cost Engineers are based on cost-to-capacity curves, factored equipment costs, and costs of similar projects adjusted for escalation and geographical location. The expected accuracy of such an estimate is +30 to -50 percent.

It is not possible to predict the impact of future conditions including, but not limited to, bid climate conditions, labor and material availability, unknown subsurface conditions, the effect of escalation or variations in project concept due to new data or constraints. Furthermore, no definitive design on which to base estimates of quantities and other estimates has been developed. Costs should therefore be considered conceptual and used only as planning level tools. Priority in developing costs was given to maintaining unbiased cost estimates for the various alternatives and sensitivity analysis of cost-intensive items.

Table B-1
ASSUMED AVERAGE LABOR RATES

Professional	700 LE/mo
Clerical	350 LE/mo
Operation staff average	6.5 LE/day
Skilled[a]	7.5 LE/day
Semiskilled[b]	5.5 LE/day
Unskilled[c]	3.0 LE/day

[a]Examples: electrician, finish carpenter, plasterer, mason, plumber

[b]Examples: mechanic, rough carpenter, rodman

[c]Examples: earthwork laborer, watchman, material handler, concrete and mortar mixer

Table B-2
ASSUMED AVERAGE MATERIALS COSTS

Item	LE
Granular bedding	6.50/m^3
Concrete gravel	10.00/m^3
Sand	3.50/m^3
Facing brick	100/1000
Structural brick	50/1000
Plywood	40/m^2
Lumber	200/m^3
Rebar--deformed	880/tonne
Cement	70/tonne
Fabricated steel	1.00/kg
Power supply	
Up to 500 kW	0.01/kWh
More than 500 kW	0.005/kWh
Power demand	25/kW
Chlorine	0.30/kg
Diesel fuel	0.035/l
Polymer	4.50/kg

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Table B-3
ASSUMED IN-PLACE UNIT COSTS

Item	LE
Trench excavation	
2 m	2.10/m^3
3 m	4.30/m^3
4 m	6.40/m^3
5 m	8.60/m^3
6 m	10.80/m^3
7 m	12.90/m^3
8 m	15.10/m^3
Structural excavation	3.00/m^3
Imported backfill	15/m^3
Native backfill	1.50/m^3
Haul and waste	0.60/m^3/km
Land leveling	1.05/m^3
Grubbing	75/ha
Unimproved roads	9,400/km
Secondary roads	34,700/km
Primary roads	59,500/km
Paving	12/m^2
Piling, 400 mm	30/m
Sheeting	
Wood removed	6/m^2
Wood left in place	18/m^2
Steel	33/m^2
Manholes	450/vertical metre
Dewatering--sump	7.50/m
Dewatering--well point	24/m
Canal lining	75/m^3
Concrete without reinforcing	
Slabs on grade	60/m^3
Walls, straight	145/m^3
Walls, curved	190/m^3
Slabs, elevated	140/m^3
Reinforcing, deformed	0.65/kg
Masonry	40/m^3
Plaster	4.00/m^2
Fabricated metal work	2.10/kg

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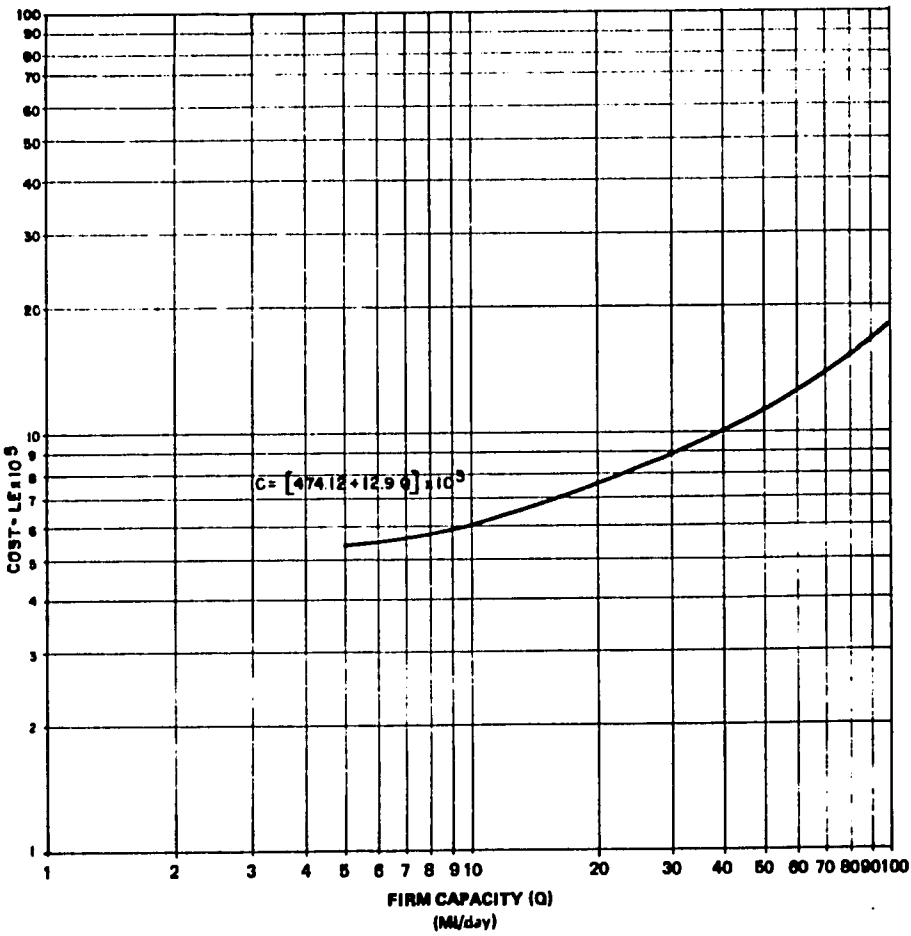

- INCLUDES STANDBY POWER
- WET PIT/DRY PIT CONSTRUCTION
- INCLUDES TWO REDUNDANT PUMPS
- INCLUDES DEWATERING COST
- INCLUDES FENCING AND APPURTENANT STRUCTURES
- EXCLUDES ENGINEERING, LEGAL, AND ADMINISTRATIVE COSTS
- EXCLUDES CONTINGENCIES

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FIGURE B-1

PUMP STATION CAPITAL COSTS
(END OF 1980 VALUE)

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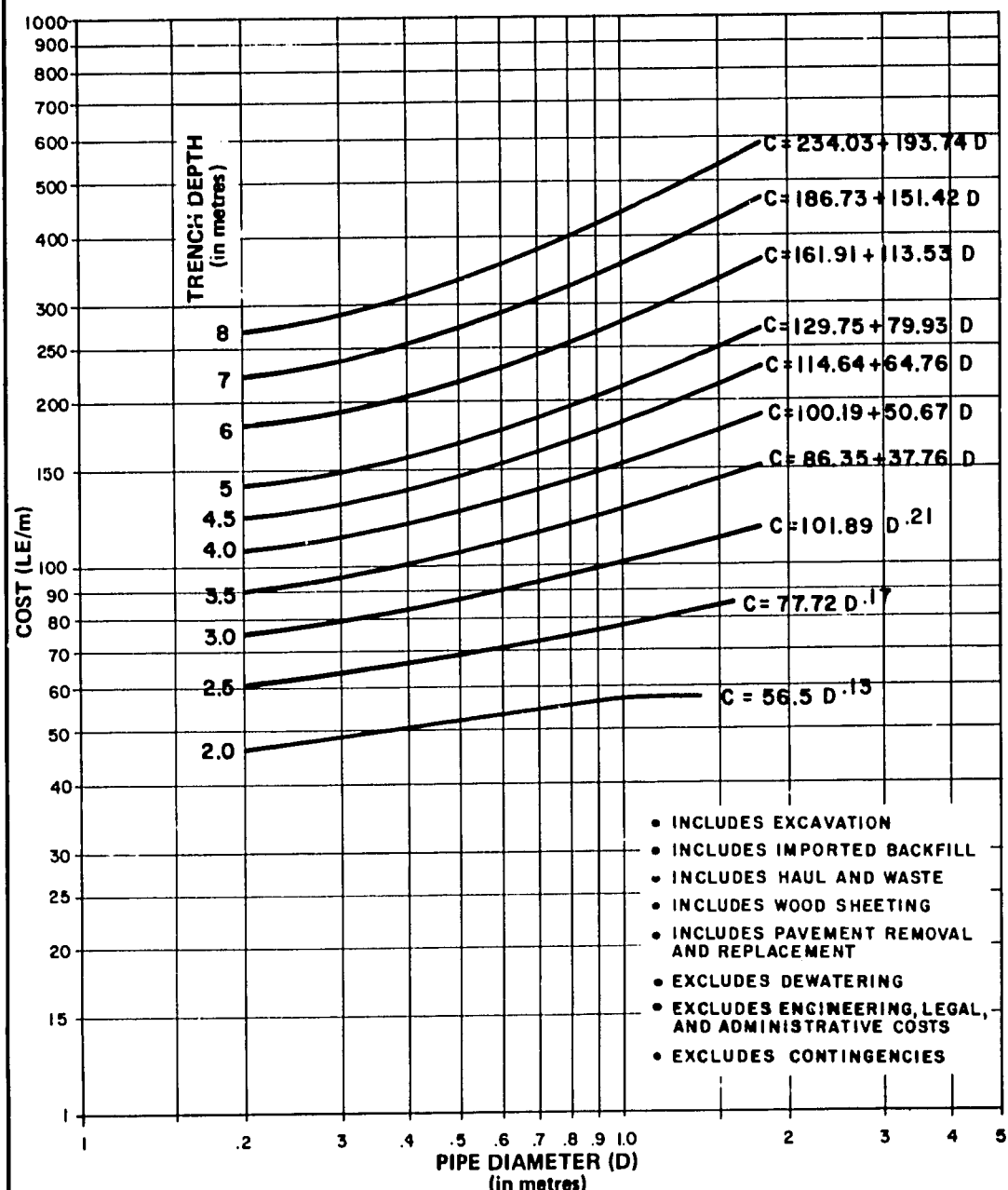

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FIGURE B-2

**TRENCH CAPITAL COSTS--
IMPORTED BACKFILL
(END OF 1980 VALUE)**

A/GOSD WWCG W/ECG

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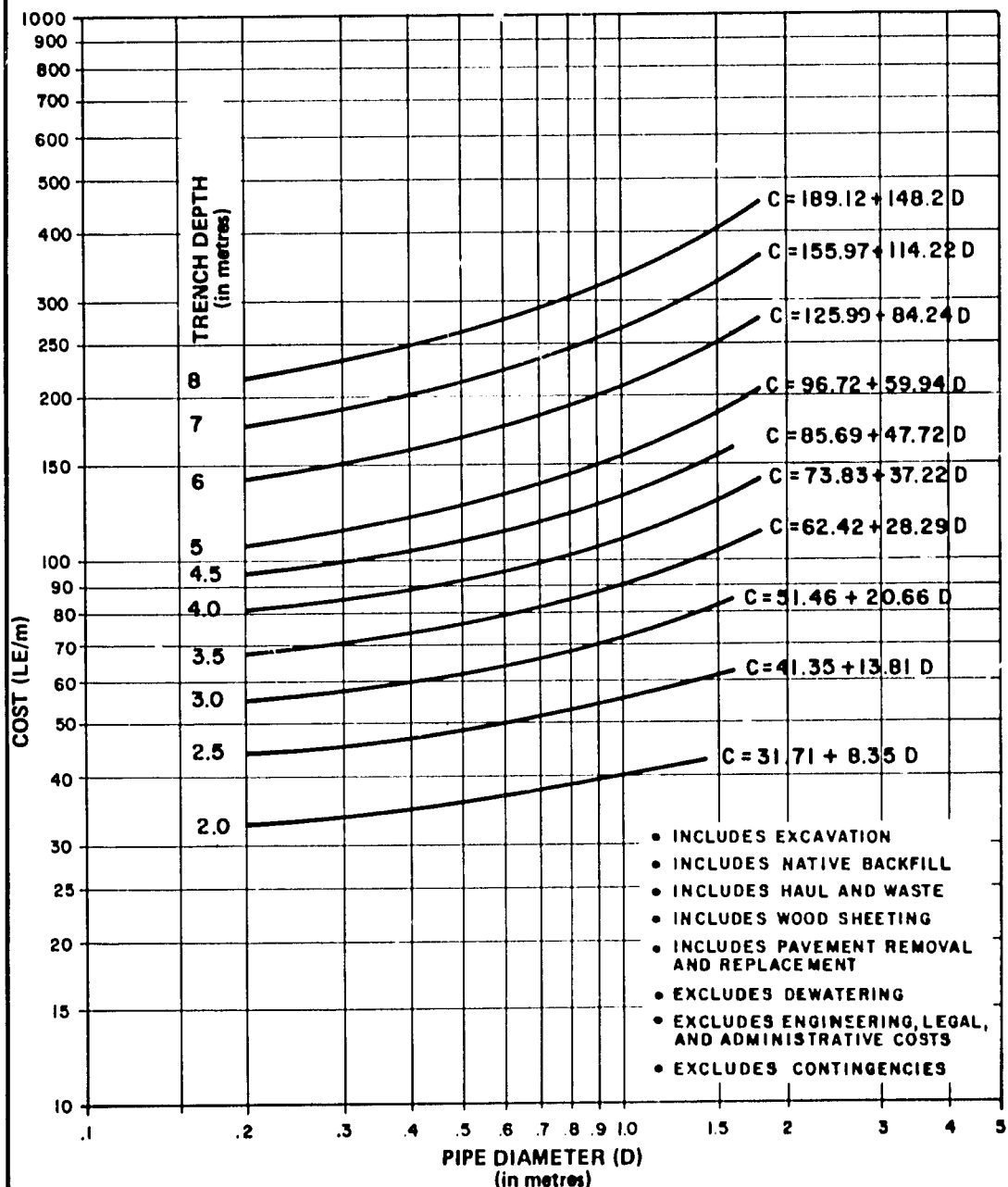


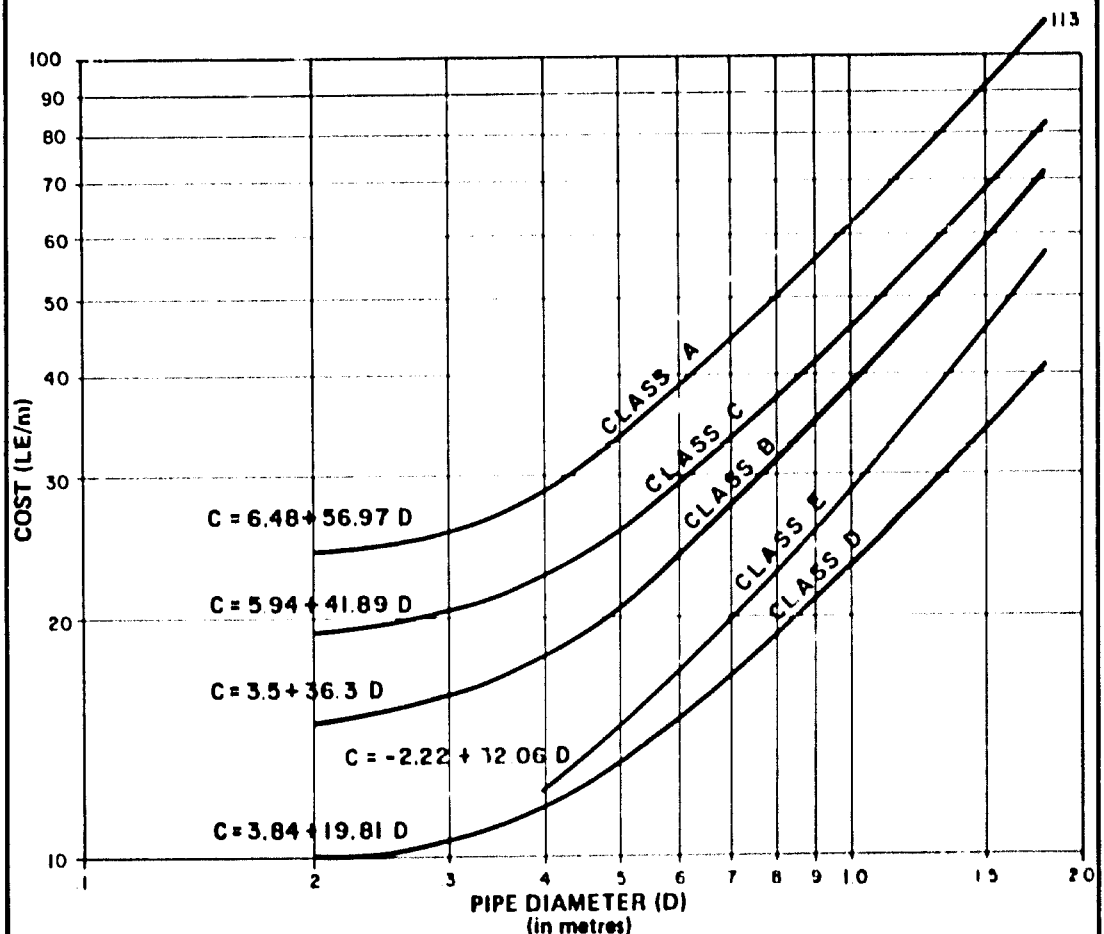
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE B-3

**TRENCH CAPITAL COSTS--
NATIVE BACKFILL
(END OF 1980 VALUE)**

A/GOSD WWCG W/ECG

222



- EXCLUDES ENGINEERING, LEGAL, AND ADMINISTRATIVE COSTS
- EXCLUDES CONTINGENCIES
- INCLUDES SAND BASE

CLASS	ABOVE PIPE	BELOW PIPE
A	CONCRETE	CONCRETE
B	CONCRETE	GRAVEL
C	GRAVEL	CONCRETE
D	GRAVEL	GRAVEL
E	SOIL CEMENT	SOIL CEMENT

1978 ALEXANDRIA WASTEWATER MASTER PLAN
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FIGURE B-4

PIPE ZONE BACKFILL CAPITAL COSTS (END OF 1980 VALUE)

A/G05D WWCG WECG

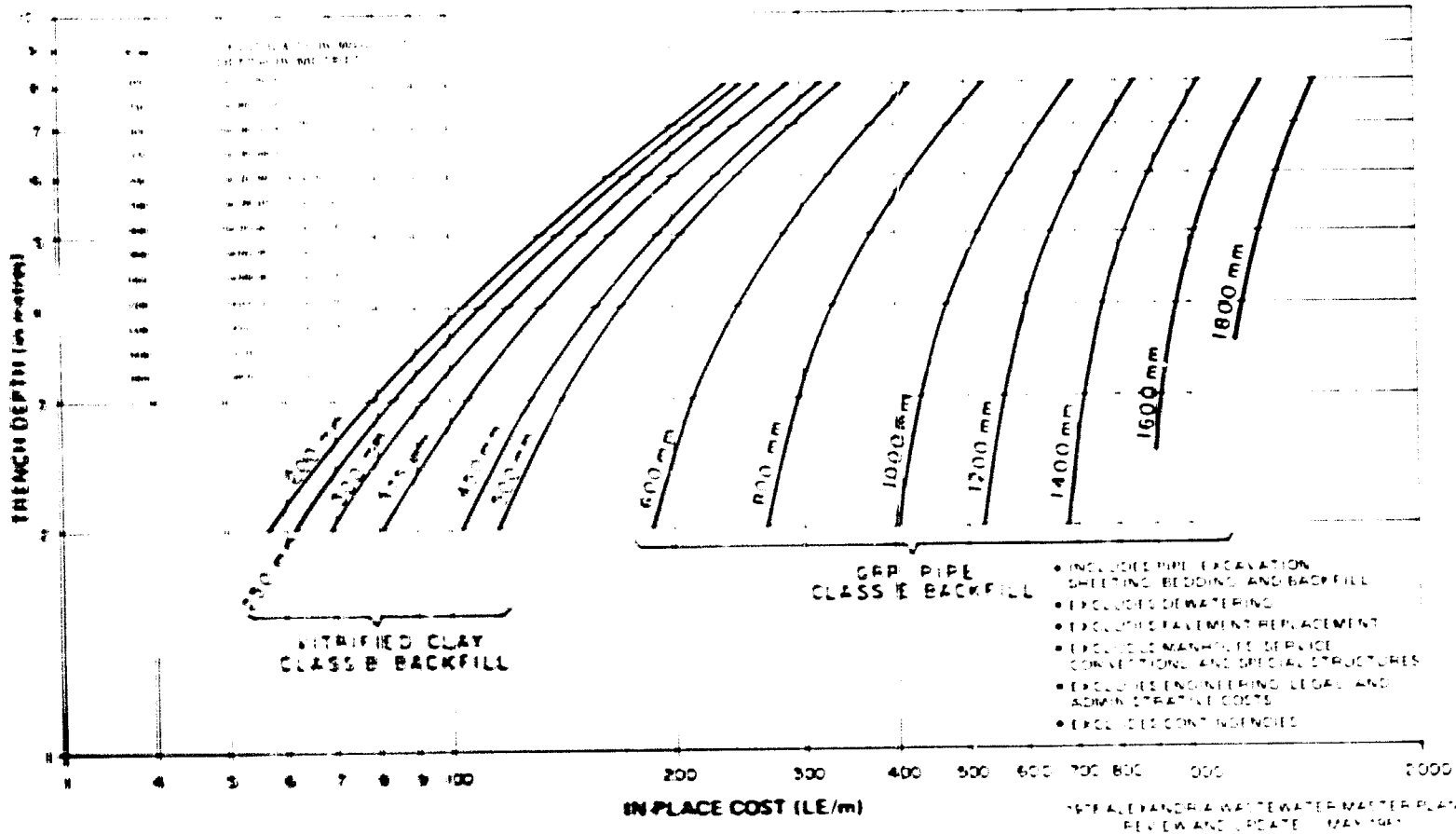
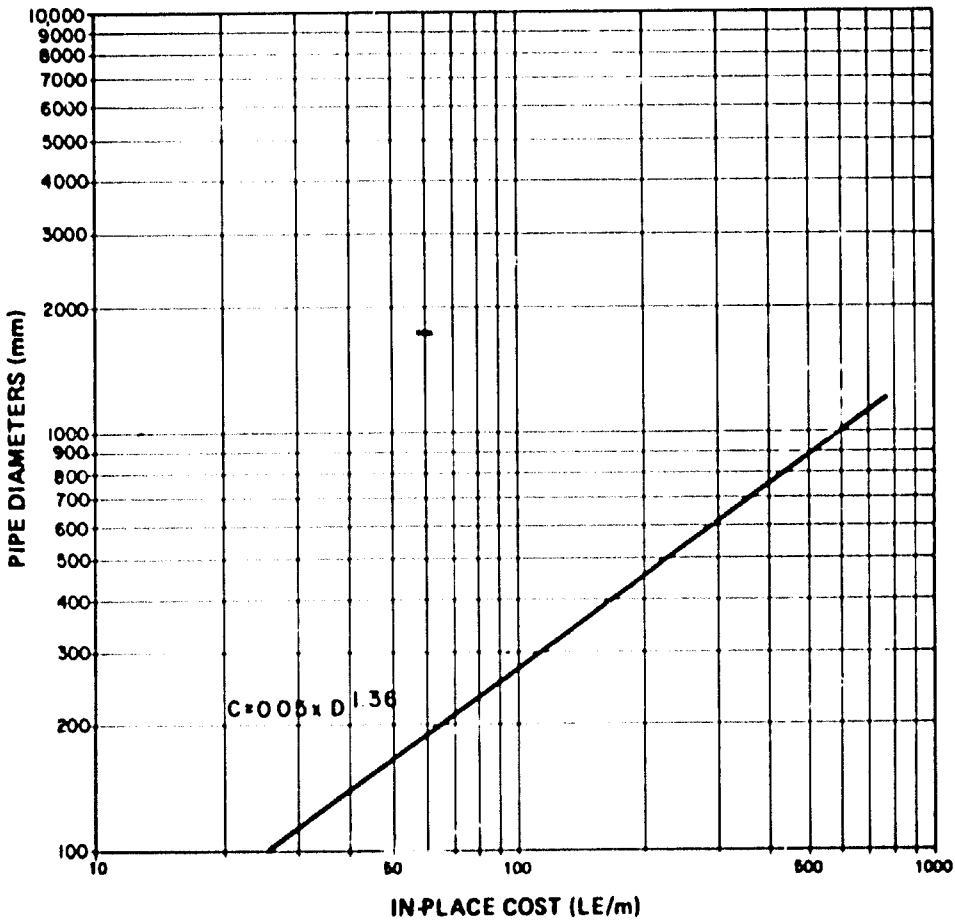


FIGURE B-5

GRAVITY SEWERS - -
IN-PLACE CAPITAL COSTS
(END OF 1980 VALUE)

A GOSD WASS WECG



- INCLUDES PIPE, EXCAVATION, AND BACKFILL
- INCLUDES 1 metre COVER
- INCLUDES NATIVE BACKFILL
- EXCLUDES DEWATERING
- EXCLUDES ENGINEERING, LEGAL, AND ADMINISTRATIVE COSTS
- EXCLUDES CONTINGENCIES

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE B-6

FORCE MAINS - -
IN-PLACE CAPITAL COSTS
(END OF 1980 VALUE)

A/0080 WWCO W/ECO

APPENDIX C

PLANNING CONSIDERATIONS UPDATE

The main purpose of this appendix is to update the wastewater flow projections presented in the 1978 Master Plan. To achieve this objective, population, water use, and land use projections as presented in the 1978 Master Plan were reviewed. The projections were then updated to reflect information that has become available since the preparation of the 1978 Master Plan.

The limits of the sewerage system study area are shown on Figure C-1. To facilitate the evaluation of various collection and treatment alternatives, the study area has been subdivided into two principal divisions, the inner and outer areas, as indicated in Table C-1 and on Figure C-1.

The scope of work covers the period between the years 1980 and 2000. This is consistent with the time interval considered in the 1978 Master Plan.

POPULATION AND LAND USE

Data received from the Alexandria Planning Department (APD); the Ministry of Finance, Department of Taxation (MFDT); and the Central Agency for Public Mobilization and Statistics (CAPMAS) were used to evaluate the population forecasts presented in the 1978 Master Plan.

The findings of our investigation are presented and discussed in the following paragraphs.

Existing Population

Permanent Population. Population statistics provided by CAPMAS and presented in Table C-2 indicate that for the years 1976 and 1980 the population of Alexandria was 2,319,000 and 2,537,000, respectively. This represents an annual increase of 2.27 percent.

As indicated on Figure C-2, the Alexandria Governorate encompasses 2722 square kilometres and is subdivided into 14 census tracts. Table C-2 presents (based on information supplied by CAPMAS and MFTD) the population for the years 1976 and 1980, and the numbers of structures and dwelling units for the year 1980 by census tract. This information was

used to determine the density of dwelling units and populations by census tract for the year 1980. The resulting densities are presented in Table C-2.

Existing population and dwelling unit densities indicate that the pattern of development in Alexandria has been linear, with the major development occurring along the coast to the east of the downtown area. Within this pattern of development, population densities range from a high of 879 people per hectare (p/ha) in Gomrok, to 487 in Ramleh, to 31 in the far eastern zone of Montazah when the entire area of that census tract is considered. Those areas in Montazah that are north of the Montazah Canal have much higher densities than those parts of Montazah located south of the canal. Population densities in the Montazah tract range from 500 p/ha in the west end to 69 p/ha in Abu Qir.

Similar development has not occurred west of the downtown area because of the influence of port activities in Minyet El-Basal and the lack of infrastructure in the western areas. Except for the planned city of Amria and the industrial free zone, development to the south has not occurred because much of the area is devoted to agricultural uses. The Mahmoudiya and Montazah canals and Lake Maryout also provide natural barriers to the development of the southern area.

Seasonal Population. Alexandria, due to its location along the Mediterranean shore, its fine beaches, and excellent tourist facilities, is a major summer resort area. It attracts tourists not only from Egypt, but from other Arabic countries and Europe. Because of the large influx of tourists, planning for wastewater facilities must account for the impact of this seasonal population. The 1978 Master Plan estimated that the year 1980 peak month seasonal population would reach 400,000. It also projected that tourists would be attracted to the Alexandria area starting in mid-March through September, a 230 day period. The peak seasonal monthly population was anticipated for August. These assumptions imply that the total seasonal population that was expected to visit Alexandria in the year 1980 was approximately 1,500,000 people. The Ministry of Tourism estimates that in the year 1980 about 1,200,000 people visited Alexandria during the tourist season. Since the estimated seasonal population presented in the 1978 Master Plan appears conservative (high), we have adopted that estimate for projection purposes.

The seasonal population occurs mainly in Montazah, Bab Sharky, Ramleh, Sidi Gaber, and the summer resort area of Agami. Montazah has three major resort areas--Mamoura, Montaza Palace, Sidi Bishr and Abu Qir. The estimated 1980 monthly peak seasonal population and its distribution are presented in Table C-3.

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Projected Population

Permanent Population. Population projections in the 1978 Master Plan indicate that Alexandria can reasonably expect a population of 4,660,000 by the year 2000. This represents an average annual increase of 3.08 percent when compared with a 1980 population of 2,537,000. This increase would be greater than the average annual increase of 2.27 percent experienced between 1976 and 1980. A population of 4,660,000 represents an increase in the percentage of Egypt's population projected to reside in the Alexandria area from 6.3 to 7.0 between 1980 and 2000. Accordingly, we believe that the population projections presented in the 1978 Master Plan are conservative. For wastewater planning purposes, population projections should be conservative to ensure that the planned facilities are adequate to meet the future needs of a community. In the event that population growth is deferred, then the components of the system can be either phased or delayed to meet actual growth experience.

Seasonal Population. We believe that the areas of Montazah, Bab Sharky, Ramleh, and Agami will continue to grow as seasonal resort areas. There are also indications that, due to the development of resort facilities along the Mediterranean shore, seasonal population growth will also occur in the Amria area. By the year 2000, the western part of Dekheila, with particular emphasis on the Agami area, will become a prominent resort area. This development will be encouraged through providing a more adequate infrastructure, water, and wastewater service in the Dekheila area. We estimate a peak seasonal monthly population of 670,000 will visit the Alexandria area by the year 2000, some 70,000 more tourists than estimated in the 1978 Master Plan. This estimate is consistent with development of the Agami coastline as a resort area.

Distribution. For the purpose of planning wastewater facilities, not only the total population, but also its geographical distribution must be known. Table C-4 presents the estimated permanent and seasonal populations and densities by census tract. The distribution recognizes that the planned developments of Sadat City in Montazah and Amria will each have 500,000 inhabitants by the year 2000. The table indicates that major population growth will occur in Ramleh, Montazah, Sidi Bishr, and Amria. This growth pattern is in conformance with past growth trends and present planning concepts.

Table C-4 indicates that population densities by the year 2000 will range from a high of 899 p/ha in Gomrok to 129 p/ha in Montazah. Areas in Montazah north of the Mamoura Canal will have population densities ranging from 620 in Sidi Bishr to 320 p/ha in Abu Qir.

Summary. Table C-5 compares population distributions presented in this study and the 1978 Master Plan. For comparison purposes, geographical districts as defined in the 1978 Master Plan have been used.

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Table C-5 indicates that this Master Plan Update anticipates a smaller proportion of the total estimated population will reside in the central and western zones. Correspondingly, more people will reside in the eastern zone areas of Ramleh, Montazah, and Bab Sharky.

These Master Plan Update population distributions are based on an analysis of future growth trends and land use in Alexandria. The findings of this analysis indicate that the inner city areas of Gomrok, Labban, Attarine, and Minyet El-Basal are presently overcrowded and are not likely to experience any large population increase over the next 20 years. Similarly, due to the potential for commercial and industrial development, large permanent population increases will not occur in Minyet El-Basal and Dekheila. The distributions also reflect our estimate that Amria and Agami will have seasonal populations of 70,000 and 160,000, respectively, by the year 2000. The findings of the future growth trend and land use analysis have been reviewed with the Alexandria Planning Department and they concur with the findings of this analysis.

As previously noted, the study area has been classified into two major divisions, inner and outer areas, to facilitate the evaluation of various collections and treatment alternatives. These major divisions have been further subdivided into seven and three districts, respectively. Estimated year 1990 and year 2000 populations within these districts are presented in Table C-6.

Land Use

In order to facilitate development of an effective wastewater master plan for Alexandria, a land use map (shown on Figure C-3) indicating existing and future land use was prepared. The map is based on data secured from the APD and other appropriate agencies. It indicates areas of existing and future residential and industrial development, and the location of existing and future facilities that will have major impacts on the development of Alexandria.

Residental. The map indicates that major residental developments are planned to the south in Amria, and to the east in Montazah (Sadat City). Each of these planned developments is expected to have a population of 500,000 by the year 2000. Areas that are now under development or that are proposed for immediate development are located in Ras El-Soda, Sidi Gaber, and Ramleh.

If reasonable development densities (400 to 450 p/ha) are maintained, the land use map indicates that sufficient land to provide for needs beyond the year 2000 is available, provided that western and southern areas are actively developed.

In order to house a permanent population of 4,600,000 by the year 2000, and to maintain a reasonable level of densities in developed areas, additional residental areas will be required. To provide this need, we anticipate that the areas of Siouf, Manshiya, El Bahameyeh, and Mahmoudiya Keblia will be developed by the year 2000 to have a population of 162,000. These developments, along with Amria and Sadat City, will provide for over 50 percent of the population growth expected between the years 1980 and 2000. This pattern of development is consistent with past growth trends, and it recognizes the necessity of extending water and wastewater facilities to these areas.

For needs beyond the year 2000, the areas most likely to be developed, as indicated in Figure C-3, lie east of the planned city of Amria, west of Dekheila, and south of the Matrouh Road.

Seasonal. A peak monthly seasonal population of 670,000 is expected to visit the Alexandria area by the year 2000. Existing recreational areas are expected to accommodate 440,000 of these visitors. The remaining 230,000, as indicated on Figure C-3, will be accommodated at Agami and in Amria, west of Agami along the Mediterranean Sea. If this substantial seasonal population is to be accommodated in these western areas, water and wastewater service will be required.

Industrial. Existing industrial areas, although widely scattered throughout the Governorate, are mainly located in Abu Qir, Siouf Keblia, Smouha, Nouzha, and in Dekheila.

Areas that are expected to undergo substantial industrial development are located in the planned city of Amria, along the northern edge of Lake Maryout in Dekheila and Karmouz, and in Abu Qir along Abu Qir Bay.

In Amria, an industrial free zone is under development, and it is anticipated that such a development will increase the potential for growth in that area. A major petroleum transfer station is located in the western part of the Governorate and a nuclear energy project to the west of the petroleum transfer station is planned.

Warehouses (mainly associated with activities of the western port) are located in Minyet El-Basal and Dekheila. With the development of the proposed Dekheila port facilities, warehousing facilities should increase in these areas. This will limit the potential for residential development.

Our preliminary studies to date indicate that the forecasted rate of industrial development between 1976 and 1980 has not been realized. Although many other factors may contribute to a low economic growth rate, one reason may be the lack of reliable water and wastewater ser-

vice. Accordingly, in planning for wastewater service for industrial areas, we have assumed that the estimated growth rate between the years 1980 and 1990 will be accelerated so that year 1990 planning goals will be achieved. We have also assumed that industries will be required to remove all constituents from their wastewaters that would prove harmful to the wastewater system or proposed treatment processes.

WATER USE

In the absence of wastewater flow records, water consumption records can be used to estimate existing and future wastewater flows. This procedure was used in the 1978 Master Plan, and for this purpose, water consumption by such categories as domestic, governmental, and industrial were established. This procedure recognizes that many components of water demand, such as system losses, are not returned to the sewer system.

Since much of data presented in the 1978 Master Plan was derived from the 1978 Waterworks Master Plan, the latter study was also taken into consideration in the review process.

Data Base

The year 1976 was used as a data base to establish water consumption within the study area. The Alexandria Water General Authority (AWGA) serves the study area, Marsa Matrouh and western areas, the western desert, and southern Beheira. The study area is subdivided into 38 water supply districts, which are shown as Figure C-4. To serve this area, the AWGA operates six water treatment plants with a combined (1976) nominal design capacity of 558,000 m^3 per day. The quantity of water produced by the six water treatment plants is estimated by noting the hours of pumping and relating the pumping capacity to rated pump operating conditions. Flow information from actual metering is used to supplement estimates when available. Actual flow measurments indicated that the estimated production quantities were 8 to 17 percent too high. Based on this analysis, and assuming an average 12-percent overestimation, the 1976 estimated demand of 684,790 m^3 per day was reduced to 600,000 m^3 per day.

For each of the water supply districts shown on Figure C-4, an economic classification was established, and unit domestic consumption rates in litres per capita per day (lcd) were determined, based on an analysis of water sales. To further confirm the findings of this analysis, eight selected areas in the city were subjected to an intensive analysis. The results of these investigations are presented in Table C-7. Table C-7

indicates that the unit domestic consumption varies widely with economic classification, and that the 1976 area-wide average unit domestic consumption was 115 lcd. In 1976, AWGA served 2,413,700 people within the study area. Our analysis indicates that 780,400 (32 percent) were living within a low income area. Estimated unit domestic water consumption for these persons was 65 lcd. This very low water consumption affecting nearly one-third of the current study area population significantly lowers area-wide average water consumption.

Table C-8 indicates the estimated 1976 water consumption by category. Domestic water consumption was determined by estimating the population served, including seasonal, and applying a consumption rate of approximately 115 lcd. Industrial consumption was based on a review of AWGA records, adjusted for underestimation and included an allowance of approximately 6000 cubic metres per day to cover small industries, and those industries for which data from AWGA was not available.

Governmental water use was estimated on the basis of the population served and a unit consumption of 30 lcd. Other uses include an allowance for water used by ships in the harbor, some commercial establishments, charitable societies, and places of worship. System losses include leakage, waste, and other uses that could not be identified. System losses constitute about 15 percent of the total estimated water consumption within the study area.

Projected Water Use

Future water demands are estimated by first developing unit consumption rates by category of use such as domestic, industrial, and governmental. To determine future water demands, the unit consumption rates are then related to population forecasts and an allowance is made for system losses. This procedure was used in the 1976 Master Plan. Table C-9 indicates unit consumption projections that were used to develop water demands in the study area. Unit domestic water consumption rates were expected to increase from approximately 116 to 159 lcd between 1976 and 2000. This represents an annual increase of 1.32 percent. This rate of increase is substantially greater than the rate of increase now being experienced in the United States but is reasonable for developing countries.

Industrial consumption is expected to increase from 44.3 to 70.2 lcd over the 24-year period. This represents a 1.9 percent annual increase. Other categories of unit demands are estimated to remain relatively constant, with system losses ranging from 15 to 12 percent of the total demand.

Table C-10 indicates water use projections within the study area by category. Because of the lack of meaningful unit consumption data by category, determination of the actual increase in water demand within the study area between the years 1976 and 1980 is difficult. To do so would require a detailed analysis of water billing data and an extensive field measurement program, both of which are beyond the scope of this study. For approximate evaluation purposes, production records as provided by the AWGA have been reviewed. Table C-11 indicates the quantity of water produced by each of the six water treatment plants serving the entire service area between the years 1976 and 1980. The total indicates that, between the years 1976 and 1980, the total production increased from 600,000 to 816,000 m^3 per day, a 36 percent increase. In the 1978 Master Plan, a total production of 816,000 m^3 per day for the entire service area was expected to be required by the year 1982. Table C-11 indicates that water consumption increased faster than that forecasted.

However, production figures for the year 1980 may have been overestimated. The AWGA, in determining corrected values of production, used correction factors that were developed for 1976 in the 1978 Waterworks Master Plan Study. Because of the method of measurement, when plant production increases, the relationship between actual production and reported production will decrease. Therefore the use of 1976 correction factors may be in error.

Interestingly, the three water treatment plants which experienced the greatest increase in production were Manshiya, Maryout, and Siouf. Manshiya serves the central zone, Mex Dekheila including Agami, and Matrouh. Since the central zone is a relatively stable area, the increased production can be attributed to increased demand in Mex Dekheila, Matrouh, and along the pipeline that serves Matrouh. Similarly Maryout serves the desert area and a small portion of Amria. Plant production records indicate that water demand may have substantially increased in the desert area. Siouf serves Sidi Gaber, an area east of Sidi Gaber as far as Miami, and villages in Manshiya El-Bahareyah. This service area is expected to have an appreciable growth in population within the next 24 years, and water production figures at the Siouf water treatment plant confirm that an appreciable growth in this area has occurred between 1976 and 1980.

Analysis of 9 months of water consumption records for the year 1980, by category, indicates that total water demand within the study area, including losses, may be reasonably assumed to be 673,000 m^3 per day. Comparison with estimated demand figures presented in Table C-10 for the year 1980 indicates that the estimated and calculated demands agree within four percent.

Since the unit water consumption projections appear conservative and are in close agreement with studies the consultant has undertaken elsewhere in Egypt, the unit water consumption data set forth in the 1978 Master Plan was used in this investigation.

WASTEWATER FLOWS AND CHARACTERISTICS

Identifying existing wastewater flows and characteristics is most important to the master planning process. Through quantification of existing flows and loads, future flows and loads may be more accurately projected. In addition, knowledge of existing wastewater characteristics often can aid in the selection of a treatment process or equipment which will function best for a given condition.

The following analysis of wastewater flows and characteristics is based on the data presented in the 1978 Waterworks and Wastewater Master Plans, and the consultants' experience elsewhere in Egypt. In the review process, particular attention has been given to the relationship between water unit consumption rates and wastewater flows.

Because of the extensive industrialization of Alexandria and the use, by industries, of appreciable quantities of water from sources other than the AWGA, the 1978 Master Plan undertook a detailed industrial analysis to determine industrial wastewater flows and characteristics. This portion of the study has been reviewed and the findings of this investigation are reported in Appendix D of this report. These findings have been incorporated, as applicable, into the wastewater flows and characteristics presented in this appendix.

Wastewater Flows

Data Basis. Flow data is helpful to the design of system components as it provides a basis for the determination of appropriate unit flow rates, peaking factors, and minimum flows. Normally, historical flow records at treatment plants and pumping stations provide the best source of flow information. However, historical flow information is not available in Alexandria, as flow measuring devices are not installed at pump stations. An acceptable alternative approach under such circumstances is to use the water use data to estimate wastewater flows. This approach was used in the 1978 Master Plan.

The 1978 Master Plan estimated wastewater flows for the year 1977. These estimated wastewater flows are presented in Table C-12, and represent those flows that can be expected to occur within the study area during the peak monthly seasonal period. We reviewed these wastewater

flows in some detail to determine if they can be used realistically as a basis for projecting future wastewater flows. Although this update slightly modified the 1976 population statistics presented in the 1978 Master Plan, we determined that the 3 percent change was not significant enough to warrant revising the population statistics in Table C-12. Existing population estimates are not critical to this analysis, since future wastewater flows will be determined on the basis of population estimated for the period under consideration.

Based on information furnished in the 1978 Master Plan, the 1976 water consumption for selected categories is presented in Table C-13. The percentage of water consumption expected to be returned to the sewer system (in the absence of specific wastewater flow data) must be determined by either an analysis of water use and the customs of the local people or by the consultants' experience in similar environments elsewhere. In the 1978 Master Plan, a wastewater return analysis considered all losses as well as consumptive uses. This analysis apparently indicated that a wastewater return ratio of 83 percent of the domestic and commercial water consumption would adequately account for all wastewater flows derived from all community activities, excluding industrial flows. Accordingly, a unit wastewater flow of 100 lcd was used to calculate the wastewater generated from all sources other than industrial in Table C-12. Based on 1976 water consumption as set forth in Table C-13, an area-wide average 100-lcd unit wastewater flow represents a 68 percent return when domestic-commercial and governmental water demand are considered, and a return of 65 percent when water demands from the classification of other uses are considered. We believe these return rates to be low in the light of conditions in Alexandria and our experience elsewhere in Egypt.

Since field data is not available to confirm the return rates established in the 1978 Master Plan, we were unable to validate the wastewater return rate indicated in that study. Since the purpose of this report is to develop alternatives to be compared to the sea disposal plan, we have used the unit domestic wastewater flow rates as proposed in the 1978 Master Plan as being representative of the quantity of wastewater that may be expected from all activities excluding industry. This has the advantage that all alternative disposal plans may be evaluated on a common basis. We strongly recommend that during preparation of the basis of design reports, actual wastewater measurements be undertaken to ensure that the proposed facilities will meet future needs.

It should be noted further that the area-wide average unit domestic wastewater flow rate of 100 lcd has been used as a basis to determine the quantity of wastewater in both sewered and unsewered areas. This implies that wastewater production will be similar in both areas irrespective of the method of collection or disposal. Experience indi-

cates that wastewater production, as with water use, is normally higher in sewered areas. Accordingly, the domestic return rates set forth in Table C-12 should be considered to have only area-wide significance.

The quantity of contaminated industrial flows indicated in Table C-12 was determined in the 1978 Master Plan after undertaking an extensive industrial survey and analysis. Since this update, as indicated in Appendix D, has not materially changed these flows, particularly within inner areas, they properly reflect 1977 and 1980 conditions within those areas. In our proposed study covering outside areas, the industrial situation in Amria, Mex Dekheila, and Abu Qir will be more accurately defined.

Infiltration allowances cannot be accurately determined without undertaking flow measurements. In the absence of actual flow measurements, and considering the method of construction and age of the sewers in Alexandria, we have used the infiltration curves presented in Appendix A of this report to determine infiltration. These curves have proven adequate in determining infiltration quantities in other parts of Egypt. Based on the use of the old sewer curve, as shown in Appendix A, an infiltration allowance of 82 Ml/day would be an adequate allowance for an area of 4300 hectares.

Wastewater Flow Projections. Based on the population statistics and analysis of existing wastewater flows presented in the previous sections, the following procedures were used in estimating future wastewater flows:

- Total permanent and peak monthly seasonal population projections have been used in projecting future wastewater flows. The estimated flows, accordingly, represent peak monthly wastewater flows for the year under consideration.

- Population distribution as developed for this update review have been used in determining wastewater flows from specific areas.

- The area withi each district has been determined by actual measurement using recent aerial mapping.

- Unit domestic wastewater flows are assumed to include domestic, commercial, and govermental activities. Unit domestic wastewater flows as presented in the 1978 Wastewater Master Plan and subsequent reports have been used throughout.

- In accordance with the industrial update as presented in Appendix D, present and future wastewater flows as set forth

in the 1978 Master Plan have been used in estimating future industrial wastewater flows.

- Where applicable, a special allowance to include military installations, hospitals, and recreational areas has been used in developing future wastewater flows.

- Infiltration rates have been determined using the infiltration curves presented in Appendix A of this report.

Future wastewater flows are presented for the years 1990 and 2000 in Tables C-14 and C-15, respectively. As indicated, the total average daily flow during the maximum month can be expected to reach 1115 and 1476 Ml/day by 1990 and 2000, respectively. Industrial activities are expected to contribute over 50 percent of the total flows. This is in accordance with local planning concepts to maintain Alexandria's role as an industrial center of Egypt.

Comparison of wastewater flows developed for the 1978 Master Plan and this study are presented in Table C-16. Analysis of the flow presented in Table C-16 indicates that wastewater flows developed for the Master Plan Update are within 2 percent of those projected in the 1978 Master Plan, being somewhat higher. This result was anticipated, since domestic and industrial wastewater projections as developed in the 1978 Master Plan were used throughout.

The wastewater flows indicated in Table C-16 are to be used solely for comparison of the alternatives developed in this study. During preparation of the basis of design reports, we recommend that wastewater flow measurements be made to ensure that any facility will be adequate to meet the future needs of its service area.

Wastewater Characteristics

Wastewater characteristics are normally defined by noting the chemical (COD) and biochemical (BOD) oxygen demands, solids and nitrogen concentrations, as well as concentrations of various other constituents. These include heavy metals, which may interfere with the treatment process or preclude final disposal to land or sea.

Because of the large industrial wastewater flows, industrial wastewaters will have a major impact on the characteristics of the wastewater generated in Alexandria. Certain industrial wastewater constituents will have to be removed before discharge to the sewer system, because of their potential adverse impact on the wastewater treatment process, the ecology of the disposal environment, and the collection system

itself. Also, in certain cases, flow equalization may be required to prevent the wastewater system from being subjected to excessive hydraulic loads.

Data Base. In order to determine the characteristics of the wastewater generated in Alexandria, a limited sampling program was undertaken during the 1978 Master Plan. The sampling program included six sites and consisted of taking four samples at each site, composited over a 6-hour period. The findings of this analysis are presented in Table C-17.

Since, for any given disposal option, the characteristics of the raw wastewater determine the degree and type of treatment, we believe that a more extensive sampling and analysis program is required. The program is necessary to:

- o Aid in the final design of treatment systems
- o Determine the concentrations of toxic materials
- o Determine the effect of treated wastewater disposal on land or sea, and the effect of sludge disposal on land

To confirm the indicated results shown in Table C-17, we have reviewed the BOD analytical results developed at the East Treatment Plant laboratory from January 1979 to April 1980. For most of this period, the East plant was in limited operation, and 439 samples of the raw wastewater were taken and analyzed for BOD. Analysis of the data indicates that BOD concentrations ranged from a maximum value of 550 mg/l to a minimum value of 60 mg/l. The average BOD concentration over the period was 268 mg/l. Because of the nature and extent of the sampling, and in the absence of flow weighting, this analytical evidence is not accurate enough to warrant its full acceptance over the entire study area. However, it does indicate the need for a sampling and analysis program to confirm the findings presented in Table C-17.

Based on the findings presented in Table C-17, and industrial wastewater flows and concentrations given in the 1978 Master Plan, Table C-18 was developed to estimate 1977 BOD and COD loadings. Table C-18 indicates that 1977 domestic BOD loadings are equivalent to a per capita contribution of 56 grams of BOD per capita per day (gcd). This is about two-thirds of the domestic BOD concentration experienced in the United States today. For the purposes of projections, the 1978 Master Plan used a 1977 value of 50 gcd of BOD which is, although slightly lower, consistent with the findings reported in Table C-18. COD values normally range from 2 to 2.5 times BOD values, so that a domestic COD contribution of 124 gcd is consistent with the BOD contribution indicated in Table C-18.

Industrial wastewater loads were determined in the 1978 Master Plan through an industrial wastewater survey covering some 145 industries. Discharge data for 129 industries, which were estimated to contribute over 90 percent of the industrial wastewater flow, were obtained. The findings of this analysis are set forth in Table C-19. As indicated in the table, 1977 estimated industrial BOD and COD loadings are based on an average daily discharge of 226 Ml/day, and BOD and COD concentrations of 640 and 1240 mg/l, respectively.

The industrial survey also identified miscellaneous constituents that might be harmful to the wastewater systems or have an adverse impact on existing and proposed wastewater treatment processes. The findings of this analysis are presented in Table C-20. These loadings are of sufficient magnitude to warrant investigating their impact on the proposed treatment processes. These impacts are discussed in Appendix J. Preliminary investigations indicate that the quantity of such heavy metals as zinc, copper, and nickel will have to be reduced to permit land application of sludge. Similarly, lead must be reduced to permit the proper operation of biological treatment processes, and chromium must be reduced to permit land application of effluent on crops.

Projected Wastewater Characteristics. Domestic BOD contributions were estimated to increase from 50 to 74.6 gcd between the years 1977 and 2000 in the 1978 Master Plan. COD unit loadings were projected to have the same rate of increase, increasing from 126 to 140 gcd between the years 1970 and 2000. Domestic BOD contribution by the year 2000 of 74.6 gcd is about equivalent to the unit loading experienced in the United States today. Accordingly, this implies an appreciable increase in the standard of living in Alexandria over the next 23 years. If the use of dishwashers, garbage grinders, and similar appliances becomes widespread, the projected value of 74.6 gcd may be low. COD unit values have characteristically been taken as approximately 190 percent of BOD unit values.

The industrial base in Alexandria covers a wide range of activities and, in projecting future BOD and COD unit industrial loads, the present mix and diversity of industry has been assumed over the planning period. In the 1978 Master Plan, industrial BOD concentrations were anticipated to range from 640 mg/l in the year 1977 to 700 mg/l by the year 2000. This represents a 0.3 percent annual increase. Correspondingly, COD concentrations are projected to increase from 1240 mg/l in the year 1977 to 1500 mg/l by the year 2000. These nominal rates of increase imply that future large industries, which may contribute large BOD and COD loadings, should be considered on a case-by-case basis.

Projected BOD loads are indicated by district for the years 1990 and 2000 on Tables C-21 and C-22. The tables indicate that total area-wide

BOD loadings of 672,000 and 902,000 kg/day can be anticipated in the years 1990 and 2000, respectively. For the year 2000, 507,000 kg/day, or 56 percent of the total BOD load, is expected to be derived from industrial sources. This further indicates the impact that the planned industrial growth will have on wastewater master planning.

The BOD industrial loadings presented in Table C-21 and C-22 are derived from area-wide loading parameters. The industrial loading indicated in any area must be adjusted to reflect actual conditions when design is undertaken.

Unsewered Areas. For planning purposes, some allowance should be made for BOD loadings from unsewered areas because the septage waste generated should be disposed of at treatment plants. For this purpose, we have used an allowance of 36 grams of BOD per capita per day in estimating future BOD loads.

Table C-1
CLASSIFICATION OF STUDY AREA

Inner Areas	Outer Areas
Western zone	Abu Qir
Central zone	Amria
Eastern zone[a]	Mex-Dekhella

[a]The eastern zone includes the future Sadat City and currently unsewered Nouzha, Ras El-Soda, and Slouf Bahareyah areas.

Table C-2
EXISTING POPULATION AND HOUSING DENSITIES IN ALEXANDRIA

Census Tract	Gross Area (ha)	No. of Structures[a] 1980	No. of Dwelling Units[a] 1980	Permanent Population[b] 1976	Permanent Population[b] 1980	Densities: Dwelling Units/ha 1980	Densities: People/ha[c] 1980
Gomrok	178	5,490	37,381	142,800	156,464	210	879
Labban	105	3,333	22,902	78,496	86,003	218	819
Karmouz	330	8,402	56,259	213,737	234,179	170	710
Mina of Alex.	112	-----	----	501	547	---	4.9
Ramleh	1,003	29,096	43,287	446,155	488,828	143	487
Montazah	10,860	49,631	200,330	310,054	336,706	18	31
Bab Sharkey	480	6,747	72,970	215,639	236,265	152	492
Attarine	202	2,810	33,620	75,062	82,238	166	407
Manshiya	75	1,980	78,652	44,485	49,129	235	655
Moharrem Bey	530	15,847	89,200	336,228	368,386	168	695
Sidi Gaber	1,080	9,216	76,872	134,516	147,382	71	136
Dekhella	2,900	15,436	26,730	45,868	50,255	9	17
Amria	241,504	14,704	26,642	47,025	51,522	1	1
Minyet El-Basal	1,667	16,362	54,956	227,725	249,505	33	150
Alexandria Total	261,026[d]	179,054	819,801	2,318,291	2,537,409	3.14	9.7

[a]Source: Ministry of Finance, Tax Department

[b]Source: CAPMAS estimate

[c]Persons per hectare

[d]Alexandria total area does not include 11,215 ha of cultivated land.

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Table C-3
ESTIMATED 1980 PEAK MONTHLY
SEASONAL POPULATION

Area	Peak Monthly Seasonal Population
Montazah	200,000
Bab Sharkey	85,000
Ramleh	40,000
Sidi Gaber	50,000
Dekhella (Includes Agami)	25,000
Total	400,000

Source: 1978 Master Plan

Table C-4
PRESENT AND FUTURE PROJECTED POPULATION AND
DENSITY IN ALEXANDRIA

Census Tract	Gross Area (ha)	Permanent Population			Density (p/ha)		Seasonal Population		Total Population	
		1980	1990	2000	1990	2000	1990	2000	1990	2000
Gomrok	178	156,464	160,000	160,000	899	899	--	--	160,000	160,000
Labban	105	86,003	90,000	90,000	857	857	--	--	90,000	90,0000
Karmouz	330	234,179	250,000	250,000	758	758	--	--	250,000	250,000
Mina of Alex.	112	547	1,000	1,000	9	9	--	--	1,000	1,000
Ramleh	1,003	488,828	600,000	700,000	598	698	50,000	54,000	650,000	754,000
Montazah[a]	10,860	336,706	818,800	1,400,000	75	129	205,000	214,000	1,023,800	1,614,000
Sidi Gaber	1,080	147,382	304,000	360,000	282	333	65,000	66,000	369,000	426,000
Attarine	202	82,238	85,000	85,000	421	421	--	--	85,000	85,000
Manshiya	75	49,129	52,000	52,000	693	693	--	--	52,000	52,000
Moharrem Bey	530	368,386	406,900	449,300	768	848	--	--	406,900	449,300
Bab Sharkey	480	236,265	261,000	288,100	544	600	100,000	106,000	361,000	394,100
Dekhella	2,900	50,255	58,300	64,600	20	23	80,000	160,000	138,300	224,600
Amria	241,504	51,522	193,000	500,000	1	2	40,000	70,000	233,000	570,000
Minyet El-Basal	1,667	249,505	260,000	260,000	155	155	--	--	260,000	260,000
Alexandria Total	261,026	2,537,409	3,540,000	4,660,000	13.6	17.8	540,000	670,000	4,080,000	5,330,000

[a]Includes Sadat City

Table C-5

COMPARISON BETWEEN PROJECTED PERMANENT AND SEASONAL POPULATION DISTRIBUTIONS FOR 2000

District	Permanent Population		Seasonal Population		Total Population	
	Master Plan Update	1978 Master Plan	Master Plan Update	1978 Master Plan	Master Plan Update	1978 Master Plan
Gomrok	160,000	170,000	----	----	160,000	170,000
Labban	90,000	112,000	----	----	90,000	112,000
Karmouz	250,000	252,000	----	----	250,000	252,000
Ramleh[a]	770,600	706,000	54,000	54,000	824,600	760,000
Montazah[b]	1,400,000	1,345,000	214,000	214,000	1,614,000	1,559,000
Attarine	85,000	93,000	----	----	85,000	93,000
Manshiya	52,000	56,000	----	----	52,000	56,000
Moharrem Bey	449,300	385,000	----	----	449,300	385,000
Bab Sharkey[a]	578,500	532,000	172,000	172,000	750,500	704,000
Dekheila	64,600	145,000	160,000	160,000	224,600	305,000
Amria	500,000	500,000	70,000	----	570,000	500,000
Minyet El-Basal[c]	260,000	370,000	----	----	260,000	370,000
Total	4,660,000	4,666,000	670,000	600,000	5,330,000	5,266,000

[a]Includes part of Sidi Gaber

[b]Includes Sadat City

[c]Includes Mina of Alexandria

Table C-6
PROJECTED POPULATION IN DEFINED DISTRICTS

Area	1990 Population (1000's)	2000 Population (1000's)
Inner Areas		
Western Zone	580	602
Central Zone	935	1,016
Eastern Zone[a]	2,115	2,811
Subtotal	3,630	4,429
Outer Areas		
Abu Qir	79	106
Amria	223	570
Mex-Dekhella	138	225
Subtotal	440	901
Total	4,070	5,330

[a]The eastern zone includes the future Sadat City and currently unsewered Nouzha, Ras El-Soda, and Siouf Bahareyah areas.

Table C-7
1976 UNIT DOMESTIC WATER CONSUMPTION
(lcd)

Area Classification	Range	Average
Lower Income Areas	69-93	80
Middle Income Areas	128-156	154
Higher Income Areas	229-237	230
Area-wide Average		115

Source: Alexandria Waterworks Master Plan

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Table C-8
1976 ESTIMATED WATER CONSUMPTION BY CATEGORY

Category	Average Water Consumption[a] (1000 m^3/day)	Percent
Domestic	281.3	50
Industrial	106.9	19
Government	72.3	13
Other	19.1	3
Losses	81.4	15
Total	561.0	100

Source: Alexandria Waterworks Master Plan

[a]Does not include the 39,000 m^3/day that is consumed in the Marsa Matrouh and western areas, the western desert, and southern Beheiria.

Table C-9
UNIT WATER CONSUMPTION PROJECTIONS
TO 2000 BY CATEGORY
(lcd)

Category	1976	1982	1990	2000
Domestic	116.5	129.2	148.3	159.4
Industrial	44.3	64.1	70.2	70.2
Government	30.0	31.0	32.0	32.0
Other	7.9	8.0	9.0	9.1
Losses	33.7	40.1	39.1	37.3
Total	232.4	272.4	298.6	308.0

Source: 1978 Master Plan

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Table C-10
WATER DEMAND PROJECTIONS TO 2000 BY CATEGORY
($1000\ m^3$/day)

Category	1976	1980	1982	1990	2000
Population served[a] (1,000's)	2,413.7	2,507.0	2,840.0	3,604.0	4,770.0
Water use[b]					
Domestic	281.3	313.0	366.9	534.4	760.4
Industrial	106.9	144.0	182.0	253.0	335.0
Government	72.3	75.2	88.0	115.2	152.7
Other	19.1	20.0	22.7	32.6	43.2
Losses	81.4	96.8	113.8	140.9	178.1
Total	561.0	649.0	773.4	1,076.1	1,469.4

Source: 1978 Master Plan

[a]Includes seasonal population

[b]Does not include water use in Marsa Matrouh and western areas, the western desert, and southern Beheiria.

Table C-11
WATER PRODUCTION BY TREATMENT PLANT, 1976-1980
($1000\ m^3$/day)

Year	Rond Point	Siouf	Manshiya	Furn El-Gueraya	Mamoura	Maryout	Total
1976	281.6	155.3	54.2	54.2	25.2	38.1	608.6
1980	294.8	230.7	160.3	39.5	31.0	59.7	816
% increase	5	49	196	-27	23	57	34

Source: Alexandria Water General Authority, Agency Paper, January 1981.

Note: Corrected values

Table C-12
ESTIMATED 1977 WASTEWATER FLOW

Population	
Total Alexandria (permanent plus seasonal peak)	2,800,000[a]
Within presently sewered areas (percent sewered = 85%)	2,380,000
Domestic sewage generation (lcd)	100
Wastewater flows (Ml/day)	
Domestic:	
Sewered area	238
Unsewered areas	42
Total	280
Contaminated industrial:	
To sewers	89
To drains, canal, and the sea	137
Total	226
Infiltration (4300 ha x 0.15 l/sec/ha)	56
Average dry weather flow:	
In sewered areas	383
In unsewered areas	179
Total	562

Source: 1978 Master Plan

[a]1976 permanent population (2.382 million) times annual growth rate (2.7%) plus a seasonal population peak of 358,000

Table C-13
1976 WATER CONSUMPTION BY CATEGORY

Category	Daily Consumption (1000 m^3/day)	Unit Demand (lcd)
Domestic-commercial	281	116
Governmental	72	30
Subtotal	353	146
Other uses[a]	19	8
Total	372	154

Note: Assumes a population of 2,414,000

[a]Includes nongovernment institutions, water sales stations, firefighting systems, irrigation, and small special rate users.

Table C-14
ESTIMATED 1990 WASTEWATER FLOWS

District	Area (ha)	Population[a] (1000's)	Unit Domestic Rate (lcd)	Domestic Average Flows[b] (Ml/day)	Industrial Average Flow (Ml/day)	Special Allowance[c] (Ml/day)	Infiltration (Ml/day)	Total Average Flow (Ml/day)
Inner Areas								
Western zone	1,068	576	105	60	92	-	13	165
Central zone	1,322	933	125	117	28	4	21	170
Eastern zone	4,306	2,061	118	243	130	4	33	410
Nouzha[d]	266	25	83	2	25	-	2	29
Outer Areas								
Abu Qir	1,100	75	112	8	132	2	18	160
Amria	1,535	146	132	19	43	-	10	72
Mex-Dekhella	1,027	104	87	9	102	-	8	119
Unsewered		160	--	--	--	--	--	--
Totals	10,624	4,080		458	552	10	105	1,125

[a]Includes seasonal population

[b]Includes domestic, governmental, and commercial uses

[c]Includes allowances for military establishments, hospitals, and recreational centers

[d]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis

Table C-15
ESTIMATED 2000 WASTEWATER FLOWS

District	Area (ha)	Population[a] (1000's)	Unit Domestic Rate (lcd)	Domestic Average Flows[b] (Ml/day)	Industrial Average Flow (Ml/day)	Special Allowance[c] (Ml/day)	Infiltration (Ml/day)	Total Average Flow (Ml/day)
Inner Areas								
Western zone	1,265	602	125	75	110	-	15	200
Central zone	1,322	1,016	136	137	34	4	20	195
Eastern zone	4,660	2,741	122	334	189	4	35	562
Nouzha[d]	521	30	100	3	36	-	3	42
Outer Areas								
Abu Qir	1,480	106	108	12	150	2	10	174
Amria	2,900	430	136	58	62	-	16	136
Mex-Dekheila	1,350	135	103	14	144	-	9	167
Unsewered		270	-	-	-	-	-	-
Totals		5,330		633	725	10	108	1,476

[a] Includes seasonal population

[b] Includes domestic, governmental, and commercial uses

[c] Includes allowances for military establishments, hospitals, and recreational centers

[d] Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.

Table C-16

COMPARISON OF WASTEWATER FLOWS FOR YEARS

1990 and 2000

(Ml/day)

Component	1978 Master Plan Average Flow	Updated Master Plan Average Flow
Year 1990		
Domestic	456[a]	458
Industrial	552	552
Special	--	10
Infiltration	73	95
Total	1,081	1,115
Year 2000		
Domestic	633	633
Industrial	725	725
Special	--	10
Infiltration	87	108
Total	1,445	1,476

[a]Adjusted to reflect unit domestic wastewater flow rate of 98 lcd in Amria.

Table C-17

SUMMARY OF WASTEWATER GAUGING AND SAMPLING PROGRAM

	East Plant[a]	Smouha Drain	Ind. P.S.	Gheit el-Enab	Gabbary Drain	Kalt Bey	Flow Weighted Average
Flow, (m^3/s)[b]							
Average	0.55	0.57	0.45	0.54	0.15	1.17	-
Range (high)	0.64	0.69	0.45	0.68	0.22	1.71	-
(low)	0.16	0.44	0.45	0.34	0.08	0.65	-
Quality[c] (mg/l, with exception of pH)							
pH	7.0	7.5	7.0	7.0	7.0	7.0	7
BOD_5	498	543	1,100	536	445	570	565
COD	1,315	738	3,281	1,257	1,075	705	1,245
Oil & Grease	167	163	1,436	186	153	304	396
Alkalinity ($CaCO_3$)	344	362	441	472	450	358	390
Total Solids	1,850	1,300	776	1,843	1,610	1,295	1,216
Volatile	596	332	1,435	760	465	436	530
Suspended	658	282	1,107	645	412	416	552
Vol-Susp.	423	183	757	486	290	313	390
Ammonia (as N)	36	27	9	30	35	34	29
Albuminoid Nitrogen	5	5	3	7	12	11	8
Chloride	407	317	2,848	388	274	425	750
Phosphate	9	7	2	14	13	12	10

Source: 1978 Master Plan

[a]Flow measurements are known to be less than what actually occurs at intake to plant.

[b]Over a 24-hour period

[c]Average quality based on four 6-hour composited and flow weighted samples at each site.

Table C-18
ESTIMATED 1977 WASTEWATER CHARACTERISTICS

Component	Total			Industrial			Domestic			Unit Loading (gcd)
	Flow (Ml/day)	Conc. (mg/l)	Load (1000 kg/day)	Flow (Ml/day)	Conc. (mg/l)	Load (1000 kg/day)	Flow (Ml/day)	Conc. (mg/l)	Load (1000 kg/day)	
Biochemical oxygen demand	327	585	191	89	640	57	238	563	134	56
Chemical oxygen demand	327	1,245	407	89	1,240	110	238	1,247	297	124

Source: 1978 Master Plan

Note: Assumes a population of 2.380 million

Table C-19
ESTIMATED 1977 INDUSTRIAL
WASTEWATER FLOWS AND CHARACTERISTICS

Flow (Ml/day)	
From:	
Sanitary	8
Process	207
Cooling	11
Total	226
Wastewater Characteristics	
BOD (mg/l)	640
BOD (kg/day)	145,000
COD (mg/l)	1,240
COD (kg/day)	280,000

Source: 1978 Master Plan

Table C-20
MISCELLANEOUS CONSTITUENT LOADINGS

Constituent	Loadings (kg/day)
Iron	289
Chromium	85
Copper	19
Manganese	26
Zinc	49
Fluoride	0.5
Sulfide	4
Lead	9
Nickel	14
Cobalt	0.8

Source: 1978 Master Plan

Table C-21

ESTIMATED 1990 BIOCHEMICAL OXYGEN DEMAND LOADS

(kg/day)

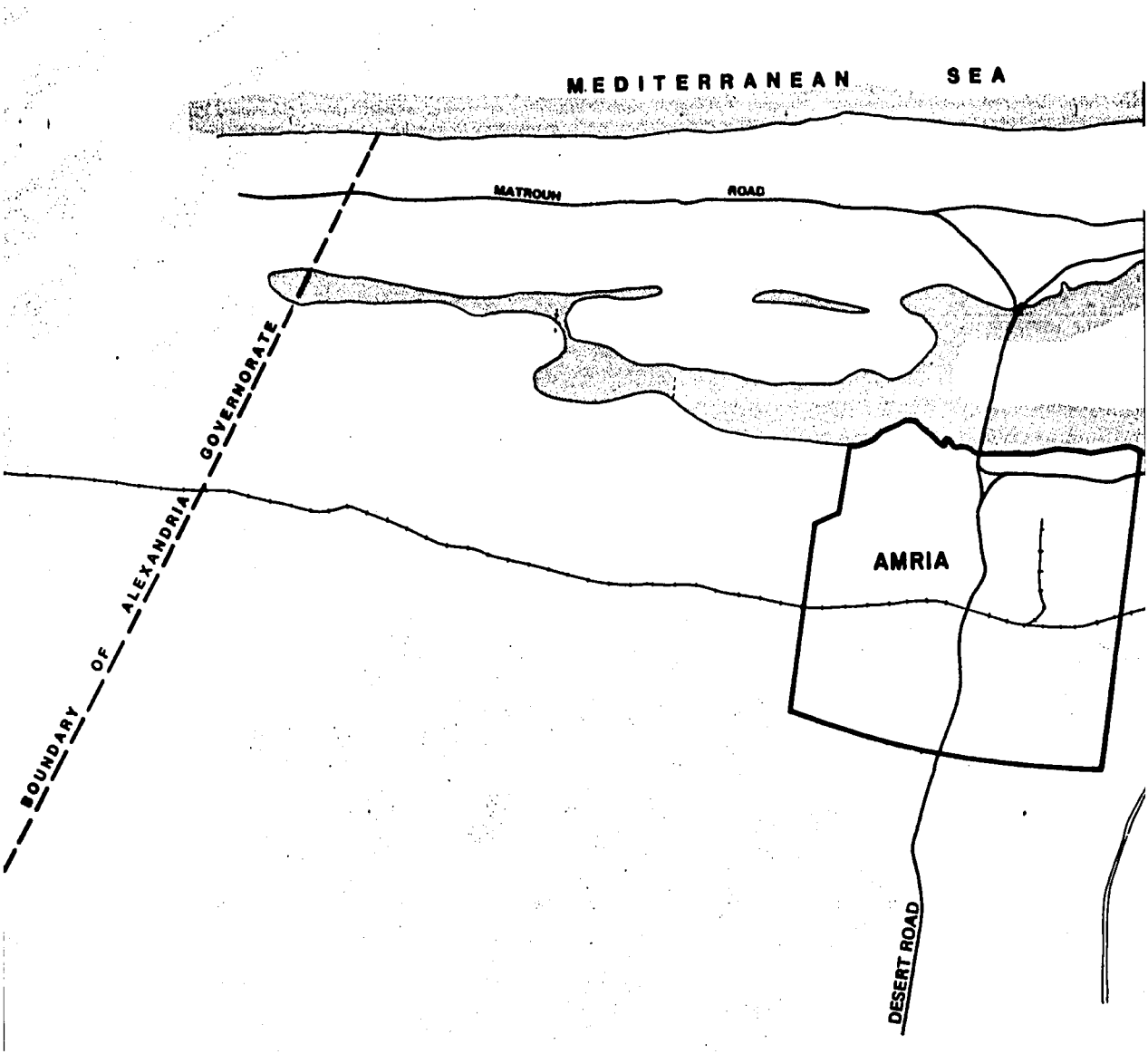
District	Domestic	Industrial	Total
Inner Areas			
Western zone	42,000	62,000	104,000
Central zone	70,000	19,000	89,000
Eastern zone	152,000	88,000	240,000
Nouzha[a]	2,000	17,000	19,000
Subtotal	266,000	186,000	452,000
Outer Areas			
Abu Qir	8,000	89,000	97,000
Amria	11,000	29,000	40,000
Max-Dekhelia	8,000	69,000	77,000
Subtotal	27,000	187,000	214,000
Unsewered	6,000	----	6,0000
Total	299,000	373,000	672,000

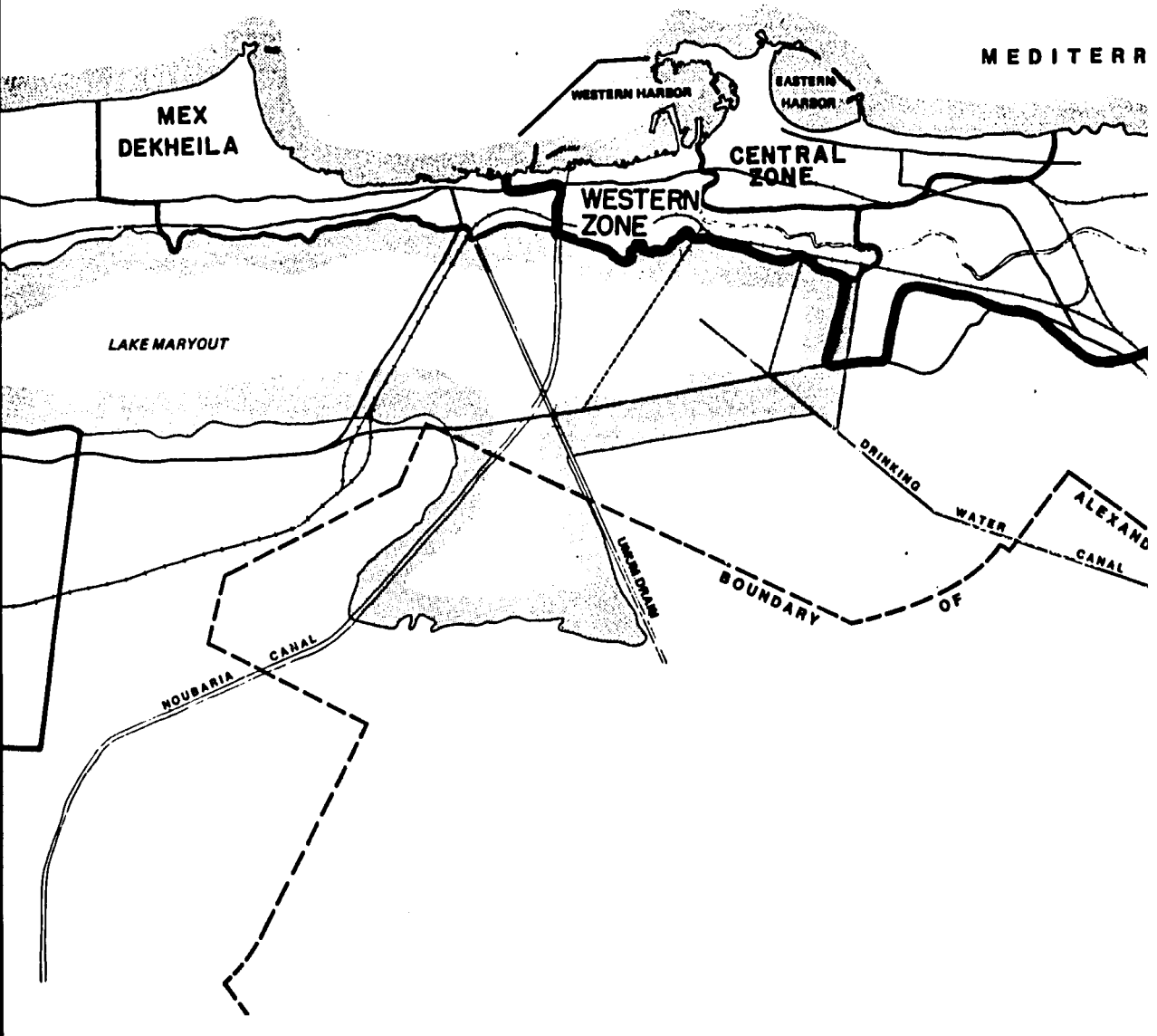
[a]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.

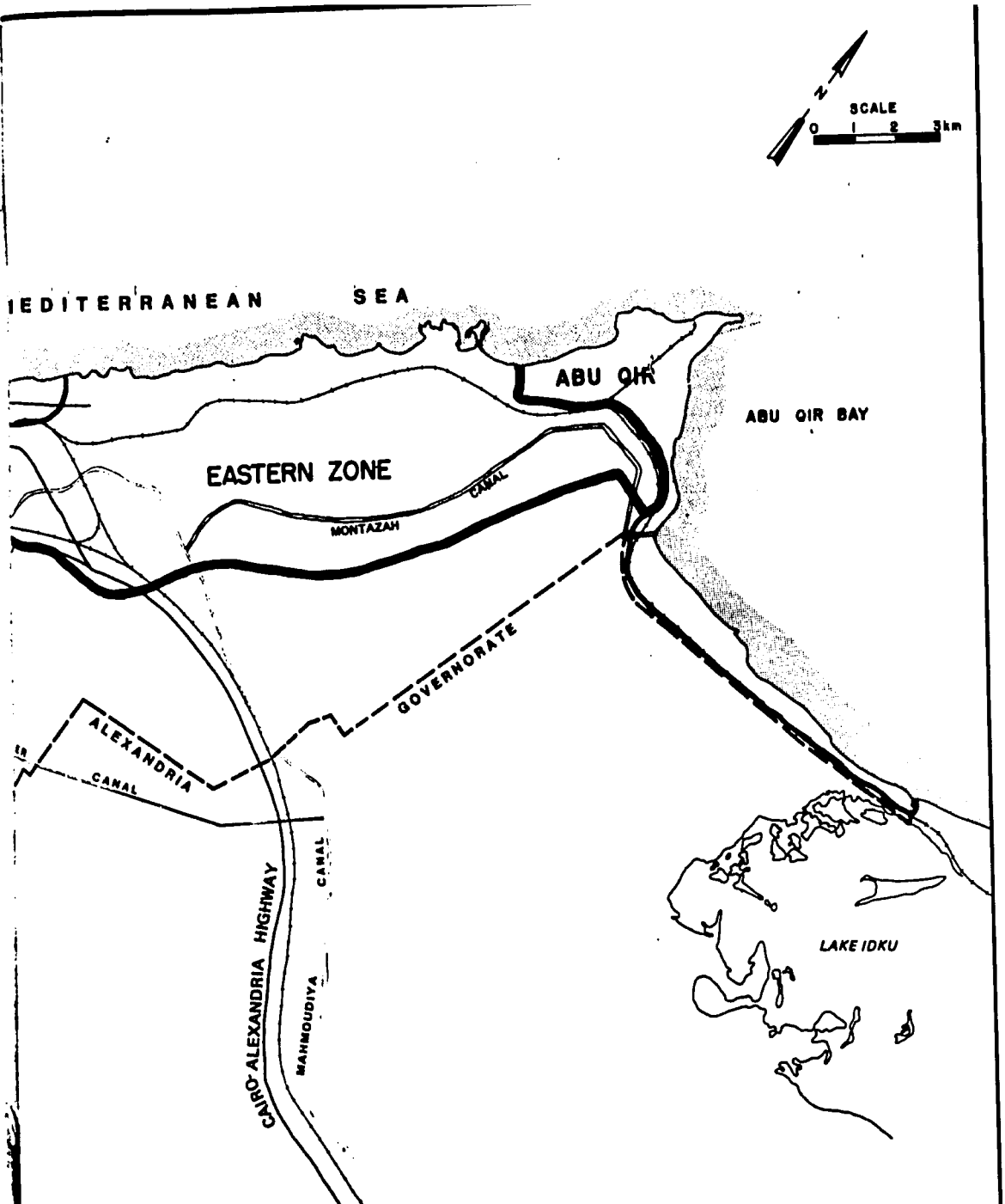
Table C-22
ESTIMATED 2000 BIOCHEMICAL OXYGEN DEMAND LOADS
(kg/day)

District	Domestic	Industrial	Total
Inner Areas			
Western zone	45,000	77,000	122,000
Central zone	78,000	24,000	102,000
Eastern zone	207,000	132,000	339,000
Nouzha[a]	2,000	25,000	27,000
Subtotal	332,000	258,000	590,000
Outer Areas			
Abu Qir	11,000	105,000	115,000
Amria	32,000	43,000	75,000
Mex-Dekhella	10,000	101,000	111,000
Subtotal	53,000	249,000	301,000
Unsewered	10,000	-----	10,000
Total	395,000	507,000	901,000

[a]Nouzha flow is carried separately from the balance of the eastern zone to facilitate East Treatment Plant analysis.


MEDITERRANEAN SEA
MATROUH ROAD
BOUNDARY OF ALEXANDRIA GOVERNORATE
AMRIA
DESERT ROAD


MEDITERR
MEX
DEKHEILA
WESTERN HARBOR
EASTERN HARBOR
CENTRAL ZONE
WESTERN ZONE
LAKE MARYOUT
DRINKING WATER CANAL
ALEXAND
BOUNDARY OF
NOUBARIA CANAL



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE C-1

SEWERAGE SYSTEM STUDY AREA

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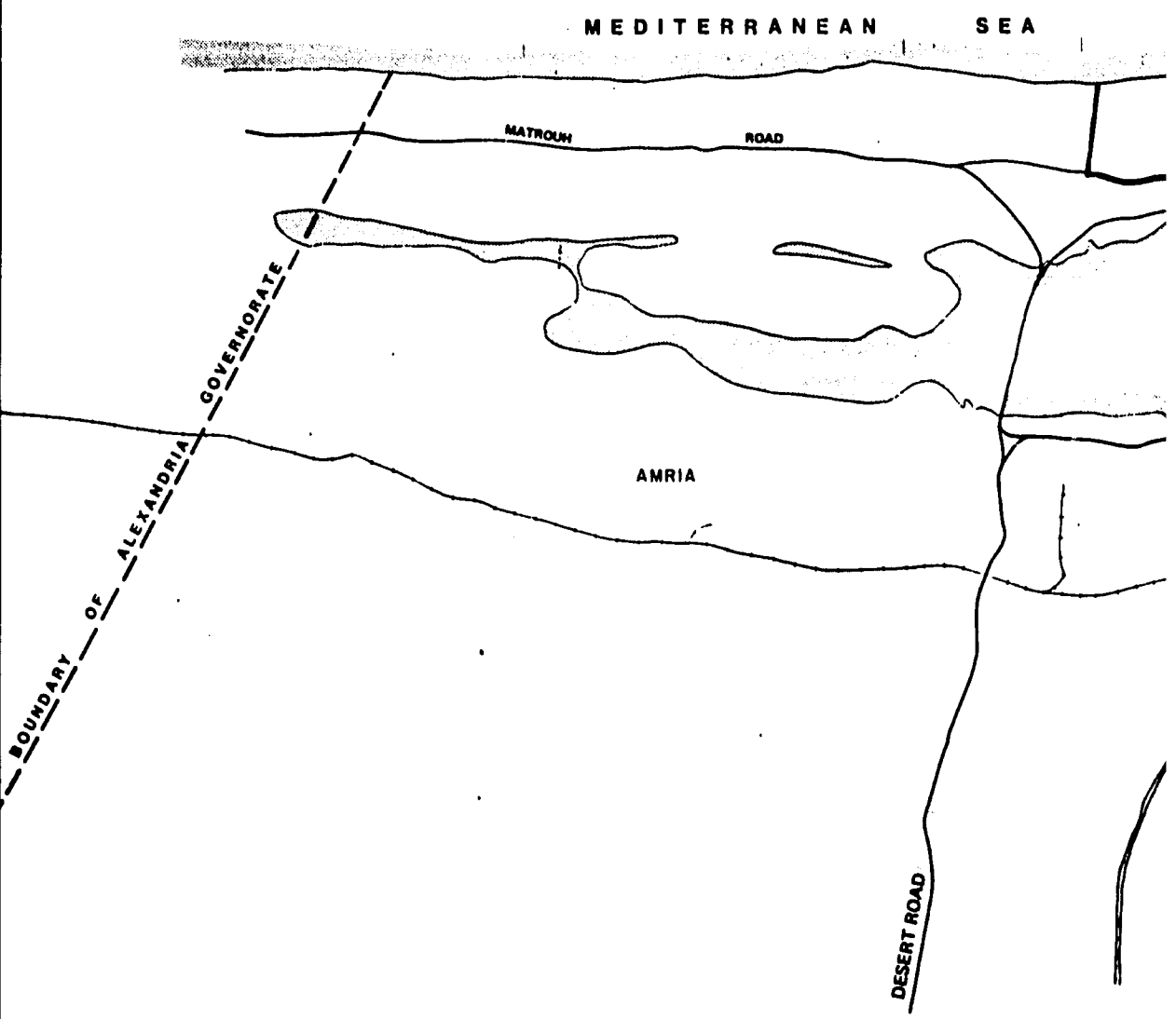
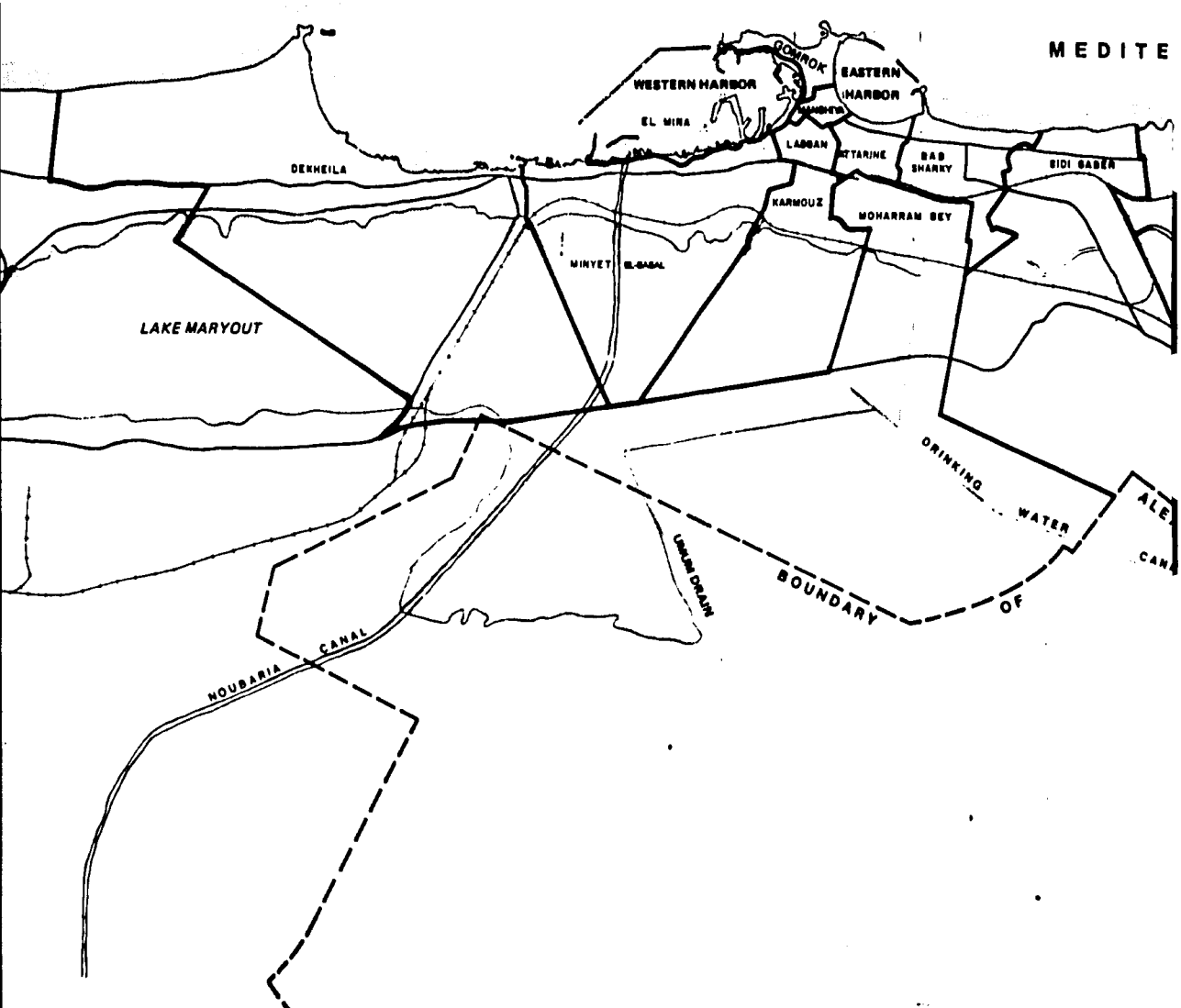

MEDITERRANEAN SEA
MATROUH ROAD
BOUNDARY OF ALEXANDRIA GOVERNORATE
AMRIA
DESERT ROAD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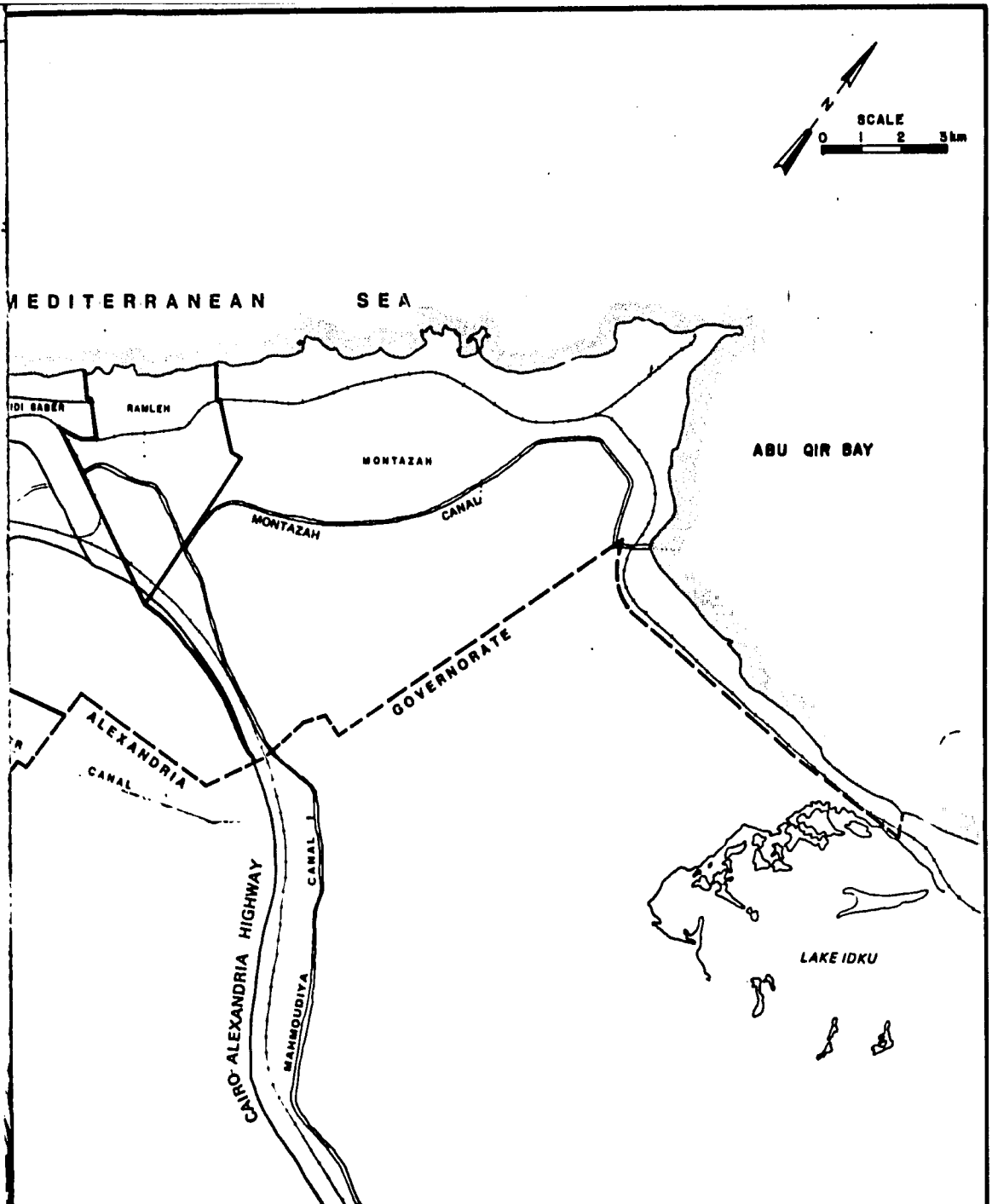
EASTERN HARBOR
EL MIRA
DEKHEILA
LABBAN
BAB SHARKY
SIDI GABER
KARMOUZ
MOHARRAM BEY
MINYET EL-BASAL
LAKE MARYOUT
DRINKING WATER
BOUNDARY OF
NOUBARIA CANAL



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE C-2

CENSUS TRACTS

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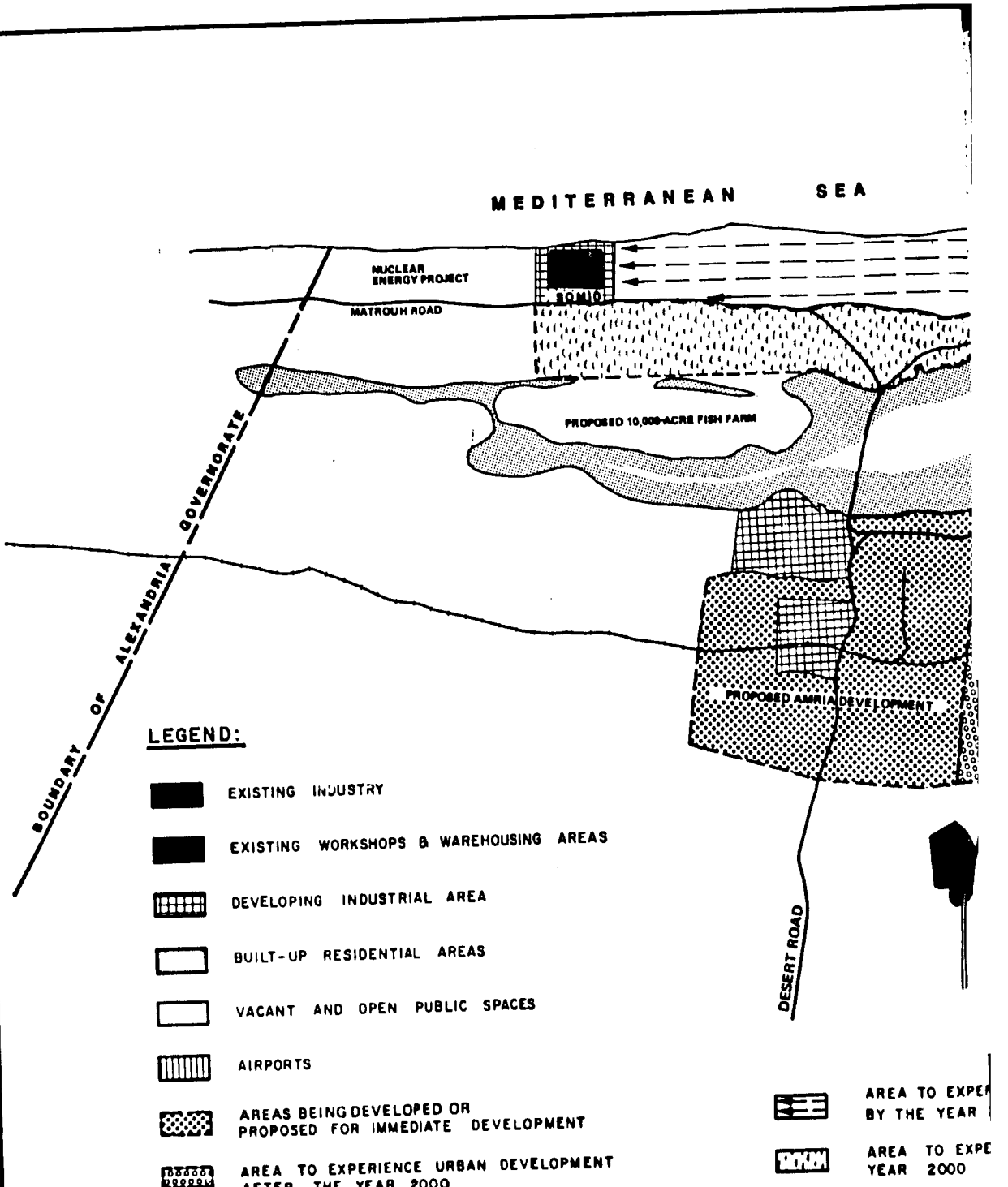
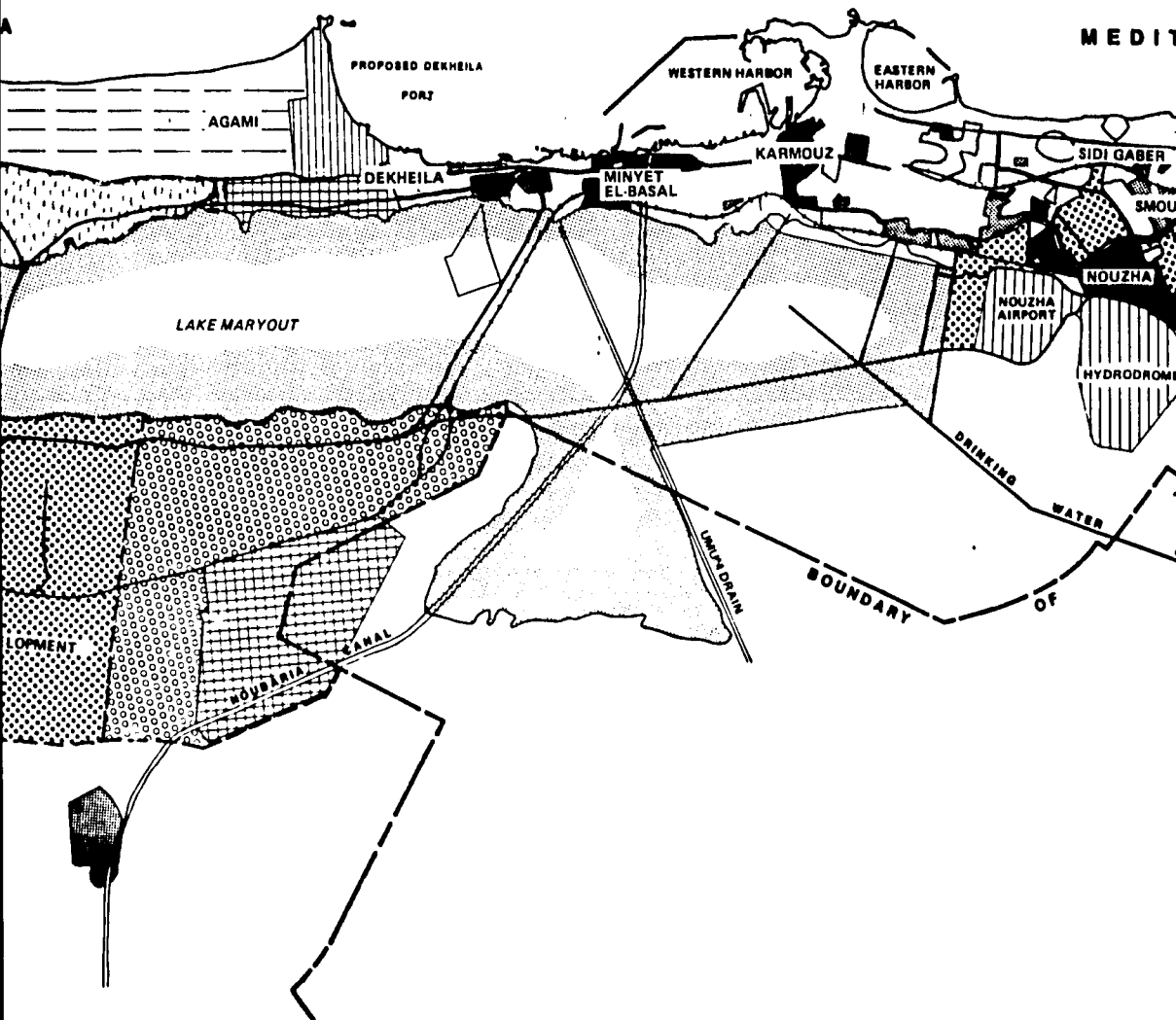
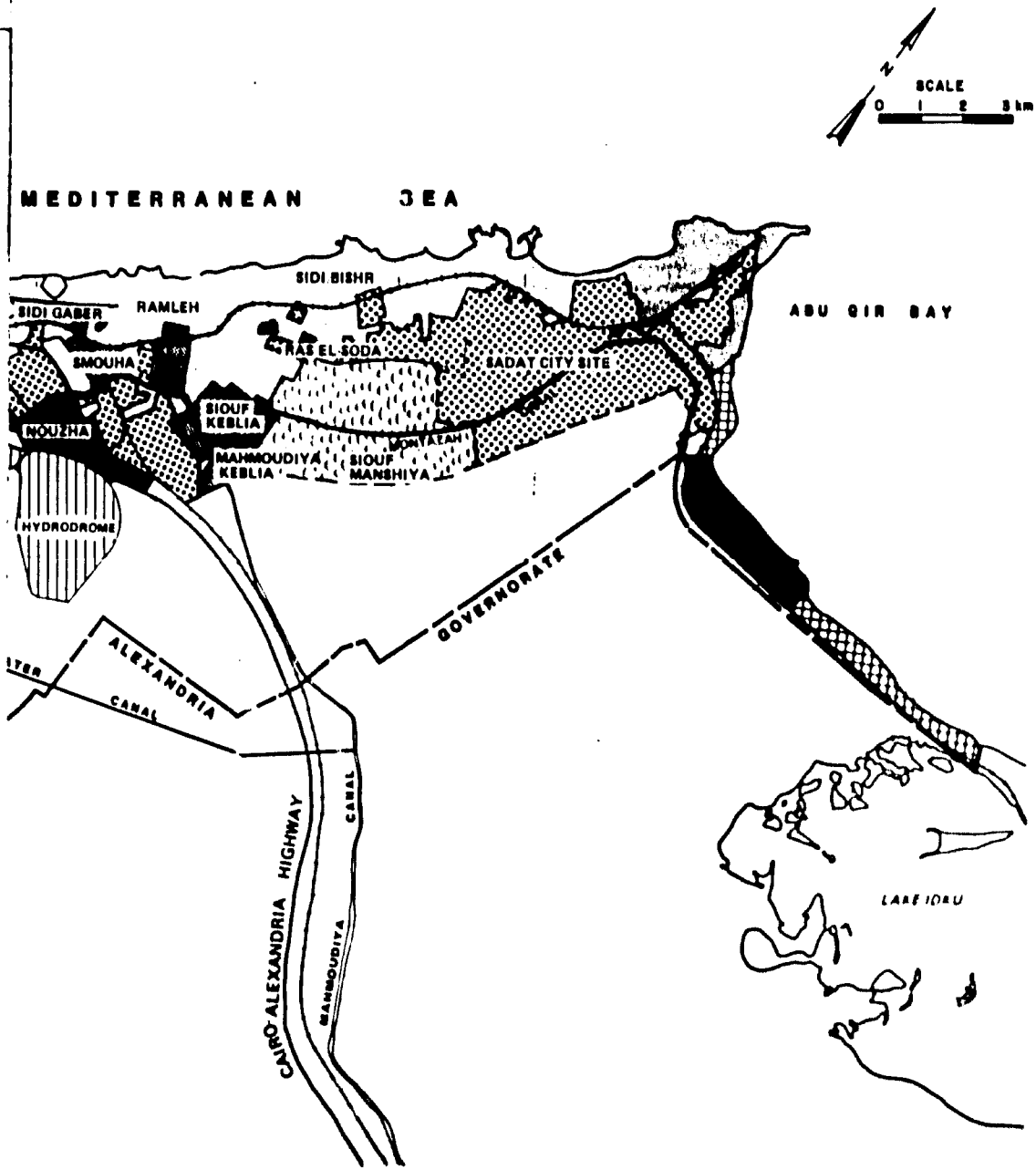

MEDITERRANEAN SEA
NUCLEAR ENERGY PROJECT
MATROUH ROAD
PROPOSED 10,000-ACRE FISH FARM
BOUNDARY OF ALEXANDRIA GOVERNORATE
PROPOSED AMRIA DEVELOPMENT
DESERT ROAD
LEGEND:
EXISTING INDUSTRY
EXISTING WORKSHOPS & WAREHOUSING AREAS
DEVELOPING INDUSTRIAL AREA
BUILT-UP RESIDENTIAL AREAS
VACANT AND OPEN PUBLIC SPACES
AIRPORTS
AREAS BEING DEVELOPED OR PROPOSED FOR IMMEDIATE DEVELOPMENT
AREA TO EXPERIENCE URBAN DEVELOPMENT AFTER THE YEAR 2000
AREA TO EXPE
BY THE YEAR
AREA TO EXPE
YEAR 2000



EA TO EXPERIENCE SEASONAL URBAN DEVELOPMENT
THE YEAR 2000

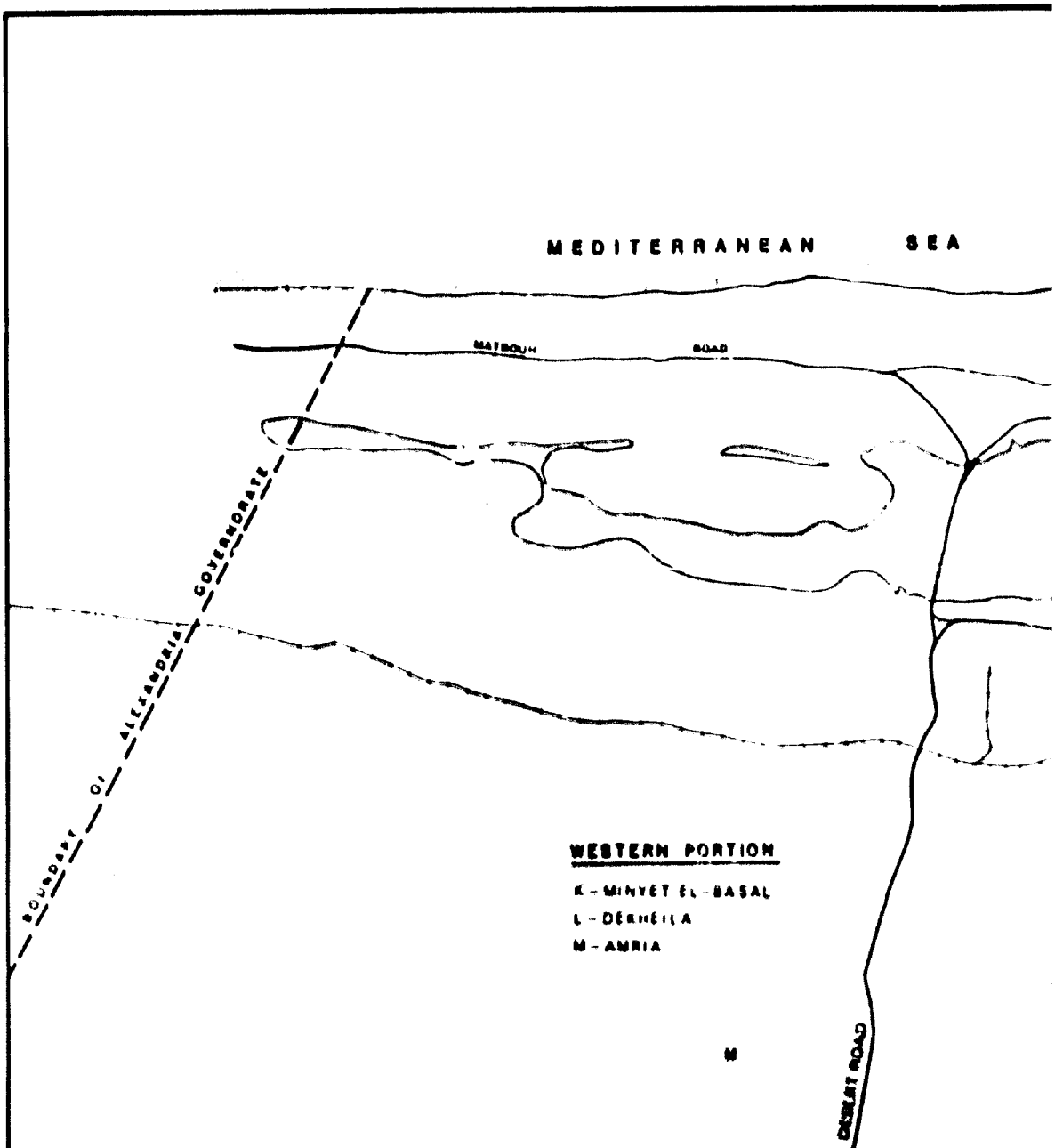
EA TO EXPERIENCE URBAN DEVELOPMENT BY THE
AR 2000

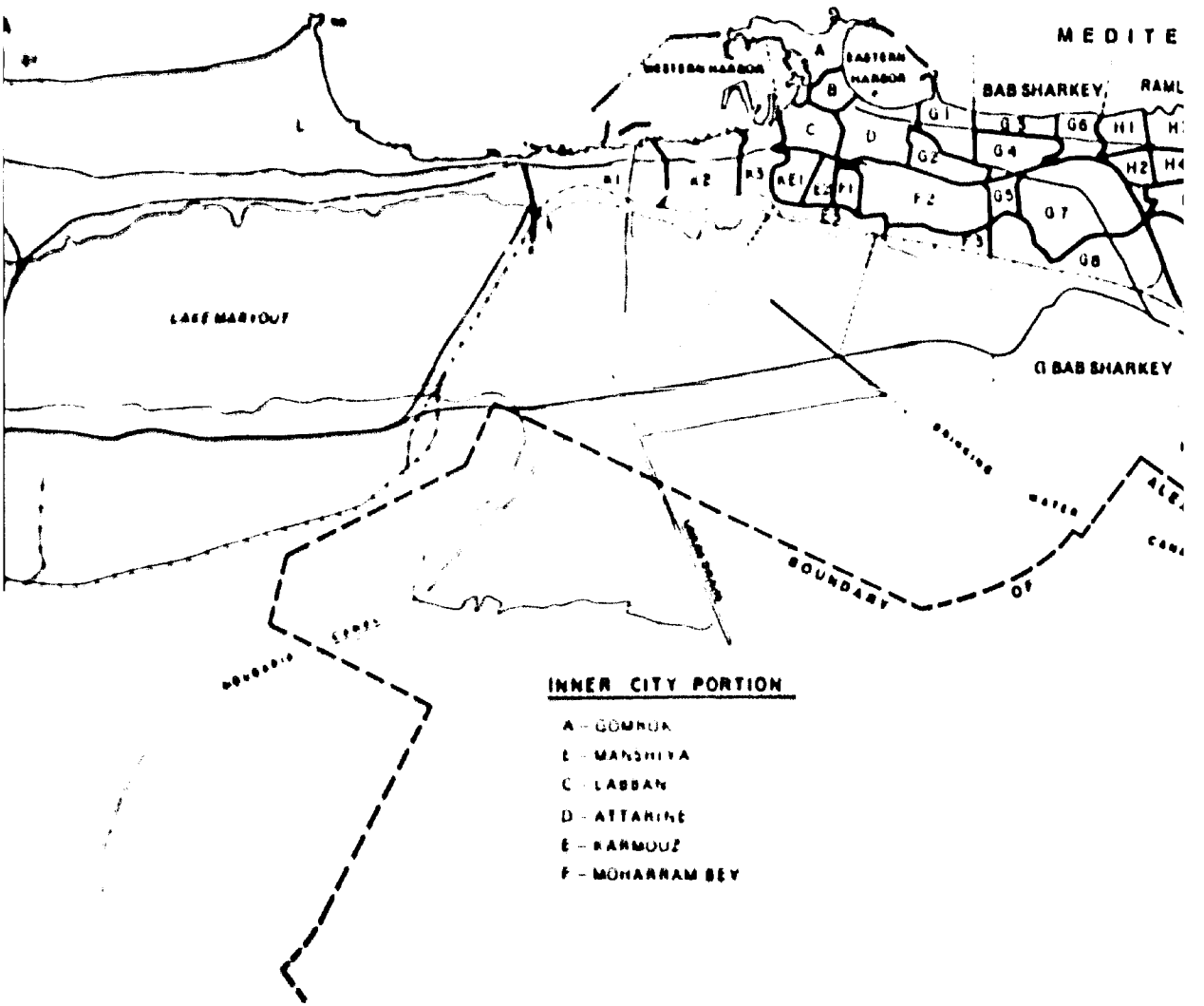


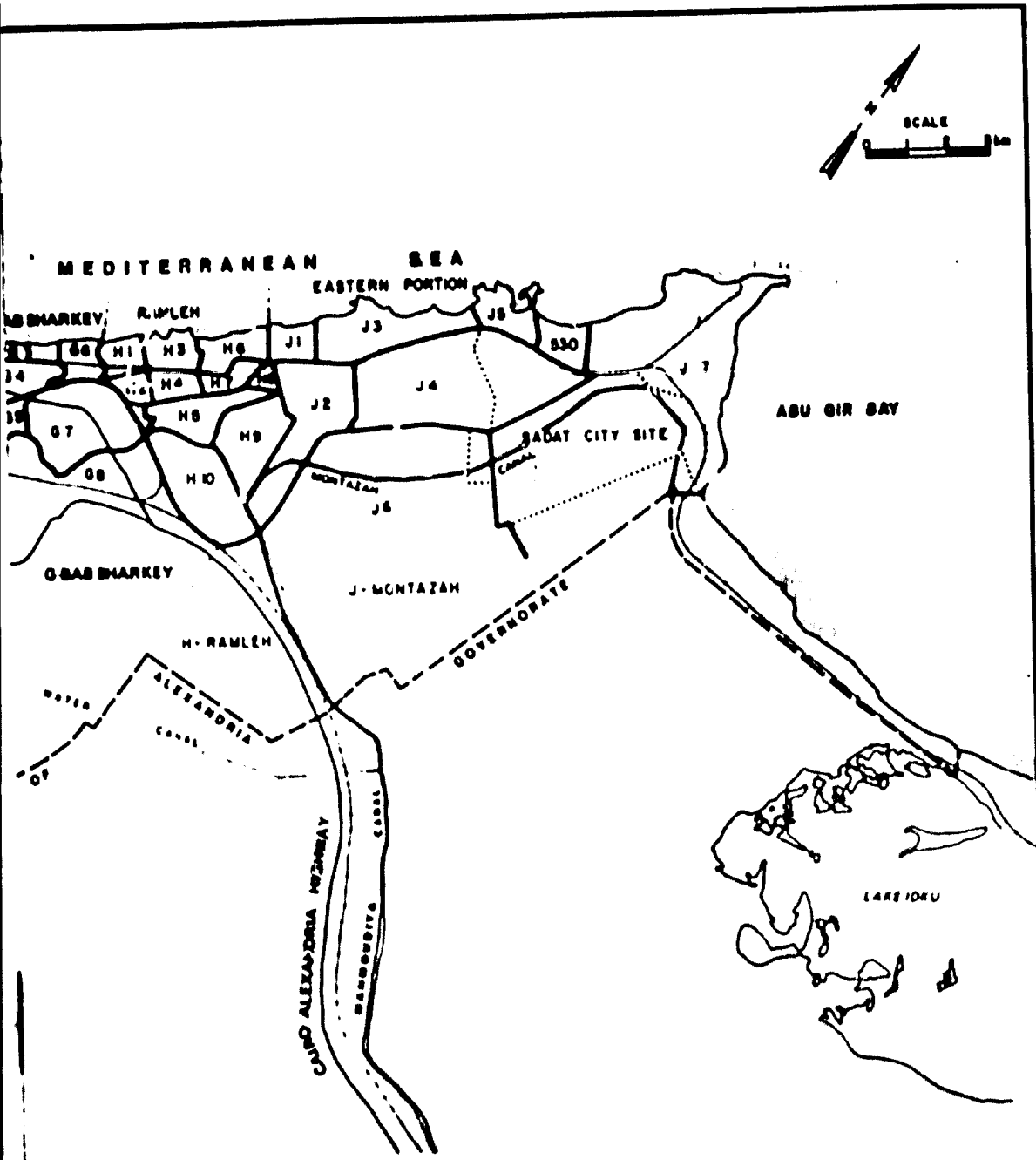
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE C-3

LAND USE


MEDITERRANEAN SEA
MATROUH ROAD
BOUNDARY OF ALEXANDRIA GOVERNORATE
WESTERN PORTION
K - MINYET EL-BASAL
L - DERNEILA
M - AMRIA
M
DEBLET ROAD


MEDITE
BAB SHARKEY
LAKE MARIOUT
BAB SHARKEY
BOUNDARY OF
INNER CITY PORTION
A - GOMRUK
B - MANSHIYA
C - LABBAN
D - ATTARINE
E - KARMOUZ
F - MOHARRAM BEY



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE C-4

WATER SUPPLY DISTRICTS

A/CORP MONCO W/ECO

APPENDIX D

INDUSTRIAL UPDATE

INTRODUCTION

The 1978 Master Plan presented the results of an extensive industrial waste study conducted in 1977. That study included results of field visits to the industries that contribute about 90 percent of the industrial wastewaters generated in Alexandria. The projections of industrial flows and loadings both current (then) and future were based on the data gathered and recommended industrial discharge practices.

Industrial flows on the whole apparently have not changed much in character since the earlier study. The lack of sewage treatment facilities has meant that the pretreatment recommendation of the 1978 Master Plan has not been carried out.

The General Organization for Industry (GOFI) is currently starting a 4-year program to add pretreatment facilities to nine industries considered to be of high priority. These industries are listed in Table D-1. Later, but likely within the next 2 years, other industries will be added to the list to receive pretreatment facilities. This program will also include recommendations for recovery and reuse of constituents previously discharged into the wastewater and recommendations for water reuse.

REVIEW OF EXISTING REPORTS AND DATA

GOFI has conducted in-plant reviews of many of the large industries in Alexandria. These were performed as a part of their program to identify those industries for which pretreatment and other wastewater related programs would be of greatest benefit in improving the wastewater disposal system of Alexandria. The GOFI staff, together with staff from the University of Alexandria and the High Institute of Public Health, also searched available data on the industries and performed related studies. The GOFI reports were reviewed as a part of this Master Plan review.

The 1978 Master Plan itself was reviewed. Attempts were made to verify or update the data presented therein. However, water use data for the industries (exempting that contained in the GOFI reports) was unobtainable during the review period. Therefore, for this review, reliance on the data in the 1978 Master Plan (with some revision resulting from the GOFI reports) was necessary.

The A/GOSD was contacted to determine if there were any private industries which might significantly affect the projections of

industrial wastewater flows and loadings. In all, some 371 small industries, employing less than 200 persons each, were found. Essentially, these industries can be classed either as nondischarging, except for minimal "domestic" flow, or as insignificant when compared with major industries, which account for about 90 percent of the industrial wastewaters generated in Alexandria. The industries identified are typically small and are predominantly forming operations (as with plastic goods) producing handicrafts, or are small food products industries.

A directory of the industries in Alexandria entitled "Egyptian Industrial Union" was reviewed, as well as the update of that directory for 1980. An additional 724 industries were found which included 19 of those listed in the Master Plan. Of the 724, 146 were determined to be of no consequence to the industrial wastewater flows. One hundred thirty-five industries or branches were listed in the Master Plan. Thus, 1211 individual industries or branches were found in this review. The Master Plan concluded that 135 of these accounted for 90 percent of the wastewater flow and 930 for the remaining 10 percent. No information that might alter the conclusions of the 1978 Master Plan as they relate to planning considerations was discovered during our review. More detailed investigations will be needed, however, for individual basis of design reports.

Location of Major Industries

Major industries are located in several areas of the city. The industries are grouped as one would expect on proximity to transportation, water supply, and labor force. Figures D-1, D-2, and D-3 show the location of major industries by drainage basins. A list of major industries as taken from the 1978 Master Plan is shown in Table D-2. No significant changes from the 1978 Master Plan were found in the locations of the major industries.

GOFI Industrial Waste Survey

Surveys conducted by GOFI indicated some changes in wastewater flows from the 1978 Master Plan for several industries. The revised flows are shown in Table D-3. The net result in these changes was to increase the flow by about three percent. GOFI did not conduct an indepth review of all major industries nor were all GOFI survey reports reviewed. In fact, the reports reviewed accounted for only one percent of the total flow. The surveys did include data on some constituents that could be of concern to wastewater treatment and disposal. Unfortunately, most of the data are not adequate to permit an indepth analysis. However, the

data are helpful in identifying potential problems to municipal treatment plants.

Staff of the High Institute of Public Health conducted a pilot plant study of treatability of wastewater from industries at Moharrem Bey, which presently discharge wastewater to Lake Maryout. The characterization of wastes flowing to the pilot treatment plant was not available at the time of this report. The results of the pilot plant study would be most beneficial if complete industrial waste treatment is a viable option. Initially, however, pretreatment of industrial flows with subsequent discharge to the Alexandria sewer system appears to be the most likely method of industrial waste treatment.

The current WWCG scope of work does not include a new industrial survey. Therefore, our review of the 1978 Master Plan in conjunction with more recent available information, forms the basis for the industrial waste characterization contained herein. Much of the data are unchanged from the 1978 Master Plan. Table D-4 presents data on constituents of the industrial wastewater found at the time of the 1978 Master Plan. Additional ongoing studies are being conducted by the High Institute of Public Health. As these studies are completed, they should be made available to A/GOSD for continuous updating of the information base.

Flow and Quality Projections

A projection of industrial flow and quality was made in the 1978 Master Plan. Assumptions were made as to the growth of industry; the effect of their wastes on collection, treatment, and disposal; and the degree of pretreatment of industrial wastewater needed. The review of the results of the 1978 Master Plan, together with a review of recent available data did not disclose any information that would tend to change those projections. Therefore, the industrial wastewater flow and quality projected for the design year is as shown in the 1978 Master Plan. These projections are tabulated in Table D-5.

Significant conclusions of the 1978 Master Plan and the WWCG review were:

- The industries should provide pretreatment.
- Different treatment methods and disposal options will require somewhat different levels of pretreatment.
- Certain constituents must be removed from the wastewater before it is discharged into the sewer system.

- Properly pretreated wastewater can be discharged into the sewer system.

EVALUATION OF IMPACTS ON TREATMENT/DISPOSAL

The impacts of industrial wastewater on municipal sewage treatment and treatment plant effluent disposal/reuse depends on the amounts and types of constituents in the wastewater. In addition, the impacts depend on the type of treatment and the location and use of the discharge.

Some constituents of industrial wastewater are easily biodegradable given sufficient oxygen to keep the process aerobic or given adequate containment and off-gas handling to keep the process anaerobic and nuisance free. Wastes, particularly from food processing industries, typically contain biodegradable constituents. Wastewater from such industries should be screened for solids removal as a minimum and should have facilities for flow equalization.

At the other end of the spectrum of industrial wastewater quality are the chemical, plastics, drug and metal production and finishing industries. Constituents from these industries often inhibit or are toxic to biological treatment processes. Unless treated, they may also be toxic to both plants and animals that may be present in or on the receiving area of the wastewater. In particular some constituents such as heavy metals are toxic in even moderate concentrations. Recommended maximum allowable effluent concentrations for these metals are presented in Appendix L. The impacts of industrial wastes on the specific alternative treatment processes and alternative discharge (disposal or reuse) options considered in this study are presented in Appendix J.

EVALUATION OF TREATMENT ALTERNATIVES

The 1978 Master Plan recommended pretreatment of industrial flows with discharge to the sewers, with the exception of industries in the Abu Qir area. For the Abu Qir industries, an individual treatment system (evaporation at Lake Idku) was recommended. This review does not address that recommendation since the Abu Qir industries are located outside the area considered for incorporation into the Alexandria system.

Pretreatment facilities to serve certain similar industries closer to the center of Alexandria were recommended in the Toxic Industrial Wastes Study, September 1978. Included were the textile companies of Hadara and Siouf and the El-Mex Slaughterhouse and Tanneries. As previously mentioned, the GOFI staff together with USAID staff have begun a study of industrial treatment needs. This work and the subsequent GOFI

pretreatment program should be monitored closely to ensure that adequate pretreatment of industrial wastes is accomplished prior to discharge to the Alexandria sewer system.

RECOMMENDATIONS

Treatment for Industrial Zones

Some industries will likely find it more economical to follow the recommendation of the Toxic Industrial Wastes Study to pretreat in a regional facility. Benefits may include reduced capital costs resulting from the use of smaller flow equalization tanks and reduced annual costs due to lessened manpower requirements.

Pretreatment Requirements

Our recommendation is that industries should pretreat to the level dictated by sewer discharge constraints and constraints imposed by the treatment/disposal option selected. The latter constraints are discussed in some detail in Appendix J.

Heavy metals such as zinc, copper and nickel will have to be reduced in amount (not just concentration) regardless of the treatment/disposal option chosen, since they will limit the application of sludge on land. Lead will have to be reduced if biological processes are chosen for the treatment option, and chromium will likely have to be reduced if land application of effluent on crops is the chosen disposal/reuse option.

Recommendations on pretreatment needed to remove potentially toxic organic materials cannot be made at present since only inadequate data exist by which to estimate their concentrations, if any, in current wastewater flows.

Sampling Program

A sampling program is urgently needed in order to obtain better data on industrial discharges. Most of the 1978 Master Plan data came from a limited number of grab samples. More and longer term sampling is required in order to adequately assess the pretreatment needs and best method of limiting the potentially harmful constituents.

WWCG recommends an initial sampling program consisting of the following work:

- One 24-hour, flow composited sample should be taken daily for 10 days plus one grab sample every 3 hours at each of eight sample points.

- One sample point should be the influent to the East Treatment Plant.

- The remaining seven sample points should be industrial wastewater discharges (whether presently discharging to a sewer or a drain). The industries chosen should be representative of the industries discharging the more toxic wastes in Alexandria. For example, discharges should be sampled from the following industries or plants: 1) chemical, 2) metal products, 3) plastics, 4) a large garage, 5) textile, 6) newsprint or cardboard, and 7) tannery.

The recommended analyses to be conducted on the samples are as shown in Table D-6.

Table D-1
HIGH PRIORITY INDUSTRIES
SCHEDULED FOR PRETREATMENT FACILITIES

Industry	Location (Zone)	Product
1. Misr Chemicals	Outer west	Caustic, chlorine & soda ash
2. El Nasr Tannery	Outer west	Tanns; leather, shoes & glue
3. National Spinning and Weaving	Inner west	Cotton yarn & cotton and silk fabric
4. Egyptian Salt & Soda	Inner west	Cottonseed oil, laundry soap, oil, candles & floor polish
5. Extracted Oil	Inner west	Extracted & edible oils & laundry soap, toilet & laundry soap and cattle feed
6. El Belva Dyers	Abu Qir	
7. Egyptian Starch and Yeast	Inner east	Rice starch, detergent & Na_2SO_4
8. Egyptian Starch and Yeast	Inner east	Rice starch, glucose, yeast
9. Egyptian Starch and Yeast	Inner east	Yeast
10. Eatina Foods*	Outer east	Canned fruits & vegetables, jams and juices

Source: Government Organization for Industry, GOFI

*May be removed from list

Table D-2
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
1	El Nasr Bottling Co.	Galal El Desouki Hadara	Carbonated soft drinks
2-1	Egyptian Bottling Co., Labban unit	Sherif Street, Labban	Carbonated soft drinks
2-2	Egyptian Bottling Co., Main unit	Ibrahim Sherif St., Pasha Mustafa	Carbonated soft drinks
3-1	Egyptian Vineyards and Distilleries, Boinski unit	Sarsya No. 3 St., Bab Sharkey	Brandy and rum
3-2	Egyptian Vineyards and Distilleries, Zottos unit	Victoria	Rum
3-3	Egyptian Vineyards and Distilleries, Kassimatis unit	Nouzha	Rum
4	Pyramids Brewery	Abdel Moneim Sonad Ibrahimia	Beer
6	Egyptian Copper Works	Mahmoudiya Canal St., Zaharia	Copper, aluminum, steel products
7	Alexandria Pharmaceutical Co.	Gamila Bohraid St., Siouf	Pharmaceuticals, rubber gloves, and plasters
8-1	Nile Match Co., Manshiya unit	Suez Canal St., Manshiya	Wooden matches
8-2	Nile Match Co., Moharrem Bey unit	Mahmoudiya Canal St., Moharrem Bey	Wooden matches
10-1	Industrial Gases, Hadara unit	Mahmoudiya Canal St., Hadara	Acetylene, oxygen, & nitrogen

Source: 1978 Master Plan

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
11	El Nasr Electric & Electronic Co. (Philips)	Mahmoudiya Canal St., Nouzha	Light bulbs and fluorescent tubes
12	Alexandria Chocolate & Confectionery Co.	Mahmoudiya Canal St., Hadara	Chocolate candy
13-1	El Nasr Dehydration Co.	Sarsya No. 3 St., Hadara	Dried vegetables
13-2	El Nasr Dehydration Co.	Malik Shaw St., Gabbary	Dried vegetables
13-3	El Nasr Dehydration Co.	Gabbary Mosque St., Gabbary	Dried vegetables
13-4	El Nasr Dehydration Co.	Mahmoudiya Canal	Dried vegetables
14-1	Edfina Co., Main plant	Mustafa Kamal, Ras El-Soda	Canned fruits & vegetables, jam
14-2	Edfina Co., Agar unit	Muffatish St., Hadara	Agar
15	Egyptian Food Co. ICA Chocolate factory	Mahattat El Souq, Bacos	Chocolate, halawa chewing gum, candy
16	El Nasr Canned Food Co. (KAHA)	Tabia, Abu Qir	Juices, jam, canned fruit and vegetables
17-1	North Alexandria Rice Hulling Co., Moharram Bey unit	Mahmoudiya Canal St., Moharram Bey	Rice
17-2	North Alexandria Rice Hulling Co., Karmouz unit	Beside Raghab Bridge Gomhouria	Rice
17-3	North Alexandria Rice Hulling Co., Gabbari unit	Malek Shah St., Gabbary	Rice

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
17-4	North Alexandria Rice Hulling Co., El Masreya unit	Nouzha Bridge	Rice
17-5	North Alexandria Rice Hulling Co., El Hadissa unit	Nouzha Bridge	Rice
17-6	North Alexandria Rice Hulling Co., El Fatha unit	Hagar el-Nawatiya	Rice
18	Misr Milk & Food Co. (Sicism Dairy)	Mustafa Kamel St., El-Rasafa	Milk, cheese, yogurt
19-1	South Alexandria Milling Co., Sanatet el Tahn unit	El Rasafa St., Ext. Moharrem Bey	Flour
19-2	South Alexandria Milling Co., Aead Mohnem Mill	Tawfikia St., Karmouz	Flour
19-3	South Alexandria Milling Co., Nafal Mill	Karmouz St., Karmouz	Flour
19-5	South Alexandria Milling Co., Hargfarat A Gawad Mill	Akhshidi St., Gabbary	Flour
19-6	South Alexandria Milling Co., Wasef Mill	Gabbary Mosque St., Gabbary	Flour
19-7	South Alexandria Milling Co., A Gawad Cylinder Mill	El Amen St., Gabbary	Flour
19-8	South Alexandria Milling Co., El Sawah Mill	Tahoudi St., Gabbary	Flour

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Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
19-9	South Alexandria Milling Co., A Gaad Son's Mill	El Aaan St., Gabbary	Flour
20-1	North Alexandria Milling Co., Muharrem bey unit 1	Mahmoudiya Canal St., Muharrem Bey	Flour
20-2	North Alexandria Milling Co., Muharrem Bey unit 2	Mahmoudiya Canal St., Muharrem Bey	Flour
20-3	North Alexandria Milling Co., Mahmoudiya unit	Mahmoudiya Canal St., Muharrem Bey	Flour
20-4	North Alexandria Milling Co., Ghali Mayele's unit	Mun. Allas El-Mahal St., Muharrem Bey	Flour
20-5	North Alexandria Milling Co., Sharaf unit	Mustafa Kamal St., Bacus	Flour
20-6	North Alexandria Milling Co., El Shanaal unit	Mustafa Pasha St., Muharrem Bey	Flour
20-8	North Alexandria Milling Co., El Mallah unit	Aldak St., Karmous	Flour
20-9	North Alexandria Milling Co., Ibrahim Nawal unit	Mahmoudiya Canal St., Muharrem Bey	Flour
20-10	North Alexandria Milling Co., Shafa unit	Mahmoudiya Canal St., Karmous	Flour
20-11	North Alexandria Milling Co., El Kaan unit	6, Mustafa El-Sharif, Ghali El-Kaan	Flour

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
21	El Nasr Cast Iron Co.	Mahmoudiya Canal St., Moharrem Bey	Cast iron, cast steel
22-1	Alexandria Ice & Refrigeration Co., Mahmoudiya unit	Mahmoudiya Canal St., Hadara	Ice
22-2	Alexandria Ice & Refrigeration Co., Moharrem Bey unit	Mahmoudiya Canal St., Moharrem Bey	Ice
22-3	Alexandria Ice & Refrigeration Co., Labban unit	El Kawkab St., Labban	Ice
22-4	Alexandria Ice & Refrigeration Co., Karmouz unit	Mahmoudiya Canal St., Karmouz	Ice
22-5	Alexandria Ice & Refrigeration Co., Athanos unit	Suez Canal St., Moharrem Bey	Cold storage
22-6	Alexandria Ice & Refrigeration Co., Souk el Gomaa unit	El Kabri El-Kadima St., Souk el Gomaa	Cold storage
22-8	Alexandria Ice & Refrigeration Co., El Mouzha unit	Albert I-St., Mouzha	Cold storage
24	West Delta Nile Bus Co.	Victor Emmanuel St., Sporha	Workshop and garage
25-1	National Paper Co., Moharrem Bey unit	Mahmoudiya Canal St., Moharrem Bey	Wrapping paper and box board
29	Labban Power Station and Gas Works	Mahmoudiya Canal St., Karmouz	Electric power & petroleum gas
30	Sioufi Power Station	El Anayad	Electrical power

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
31	Moharram Industrial Press	Mahmoudiya Canal, Nouzha	Printed material packages, gummed paper
32	Conversion of Paper Co. (Verta)	Mahattet El Souq St.	Printed paper, paper products
33	Egyptian Co. for Refractories	Mahmoudiya Canal St., Nouzha	Ceramic tiles, electric insulators
34	Egyptian Plastic Electric Co.	Ibn Akil St., Victoria	Plastic articles, lead acid batteries
35	El Nasr Co. for Glass & Crystal	Mufatish St., Hadara	Glass & fiber-glass
36	Transportation and Engineering Co.	Number 38 St., Smouha	Tires and inner, tubes
37-1	Alexandria Oil & Soap Co., Karmouz unit	Mahmoudiya Canal St., Karmouz	Hydrogenated oil
37-2	Alexandria Oil & Soap Co., Kafr el Zayat unit	Mahmoudiya Canal St., Karmouz	Edible oils, soaps & detergent
37-3	Alexandria Oil & Soap Co., Raghab unit	Mahmoudiya Canal St., Moharram Bey	Extracted cotton seed oil
37-4	Alexandria oil & Soap Co., Nouzha unit	Mahmoudiya Canal St., Nouzha	Laundry soap, toilet soap and glycerine
38-1	Egyptian Salt & Soda Co., Karmouz unit	S. Mahmoudiya Canal St. Karmouz	Cottonseed oil & laundry soap
38-2	Egyptian Salt & Soda Co., Moharram Bey unit	S. Mahmoudiya Canal St., Moharram Bey	Refined oil, candles, floor polish

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Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
39-1	Extracted Oils Co., El Haddisa unit	Suez Canal St., Moharrem Bey	Edible oils, laundry soap
39-2	Extracted Oils Co., Oil refinery	Mahmoudiya Canal St., Karmouz	Edible oils
39-3	Extracted Oils Co., Hexane Extn. unit	Mahmoudiya Canal, Moharrem Bey	Extracted oils
40-1	Egyptian Starch & Yeast Co., Sloui plant	Abu Soliman St., Sloui	Rice starch, detergent, and Na_2SO_4
40-2	Egyptian Starch & Yeast Co., Starch plant	Suez Canal St., Moharrem Bey	Rice starch, glucose
40-3	Egyptian Starch & Yeast Co., Yeast plant	Mahmoudiya Canal, Moharrem Bey	Baker's yeast
44	General Co. for Poultry	Cairo-Alexandria Highway	Slaughtered & processed chicken
46	Arab Rug Co.	Mufettish St., Hadara	Dyed yarn and knitted fabrics
47	Gomm Shops	Mufettish St., Hadara	Bleached & dyed cotton fabrics
48	Pyramids Dyeing Co.	Mahmoudiya Canal St., Moharrem	Dyed and printed viscose & cotton
49-1	Alexandria Spinning & Weaving Co., Mouna unit	Mahmoudiya Canal St., Mouna	Cotton yarn
49-2	Alexandria Spinning & Weaving Co., Hadara unit	Mufettish St., Hadara	Dyed thread & ready-made clothing

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
49-3	Alexandria Spinning & Weaving Co., Bacos unit	Mahattet El Souq St., Bacos	Woven cotton fabric
50-1	El Nasr Clothing Co., Main unit	Mahmoudiya Canal St., Hadara	Underwear, knit clothing & fabrics
50-2	El Nasr Clothing Co., Hosiery unit	Mufattish St., Hadara	Nylon stockings & cotton socks
51-1	El Nasr Textile STIA, plant no. 1	Prince Ibrahim St., Nouzha	Cotton & viscose fabric
51-2	STIA Plant No. 2	Mufattish St., Hadara	Underwear
51-3	STIA Plant No. 3	Mufattish St., Hadara	Tricot knits, and thread
51-4	STIA Plant No. 4	Mahmoudiya Canal St., Hadara	Woolen fabrics
51-5	STIA Plant No. 5 & 6	Mahmoudiya Canal St. & Muffatish St.	Woolen blankets
51-6	STIA Plant No. 7	Abu Sellaan St., Siouf	Woolen blankets
51-7	STIA Plant No. 8	Gawad Hosni St., Siouf	Blankets
51-8	STIA Plant No. 9	Zaki Ragab St., Nouzha	Wool & wool synthetic fabrics
52	El Siouf Spinning & Weaving Co.	Mahmoudiya Canal St., Siouf	Woven cotton fabric & cotton yarn
53-1	Modern Textile Co. Bavara unit	Suez Canal St., Moharram Bey	Cotton, synthetic & silk fabrics

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Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
53-2	Modern Textile Co., Durra unit	Sarayn No. 3 St.	Cotton, synthetic & silk fabrics
53-4	Modern Textile Co., Culvertex unit	Moh. Bey El-Shafee St., Bab Sidra	Bleach, dyed & printed fabrics
54-1	National Spinning & Weaving Co., Karmouz unit	Mahmoudiya Canal St., Karmouz	Cotton yarn
54-2	National Spinning & Weaving Co., Moharrem Bey unit	S. Mahmoudiya Canal St., Moharrem Bey	Woven cotton & silk fabric, terry cloth
55	Oriental Linen & Cotton Co.	Mustafa Kamal, Ras El-Soda	Linen & cotton fabric
56-1	Arab & United Spinning & Weaving Co., unit No. 1	Gamila Bokhaid St., Sioui	Cotton yarn
56-2	Arab & United Spinning & Weaving Co., unit No. 2	Gamila Bokhaid St., Sioui	Cotton yarn and fabric
56-3	Arab & United Spinning & Weaving Co., unit No. 3	Gamila Bokhaid St., Sioui	Cotton yarn
56-4	Arab & United Spinning & Weaving Co., unit No. 4	Gamila Bokhaid St., Sioui	Finished cotton fabric
56-5	Arab & United Spinning & Weaving Co., unit No. 5	Abu Sultan St., Sioui	Cotton yarn
57-1	AMSA Bab Sharqy plant	Miami Bahari, El Meyra	Water
57-2	AMSA Sioui plant	Mahmoudiya Canal St., Sioui	Water

Table D-2 (continued)
LIST OF MAJOR ALEXANDRIA INDUSTRIES

No.	Name	Location	Product
57-3	AMGA Mansiya plant	Gabbary Rapid	Water
57-4	AMGA Furn El-Guareya plant	Mahmoudiya Canal St., Karmouz	Water
57-5	AMGA Mamoura plant	Abu Qir St., Mamoura	Water

Table D-3

REVISION OF 1978 MASTER PLAN INDUSTRIAL FLOWS

(m^3/day)

Number	1978 Master Plan Values		ODFI Values	
	Water Use	Wastewater Flow[a]	Water Use	Wastewater Flow[a,b]
6	12,160	9,000	9,600	7,100[b]
8-1	600	550	400	350[b]
18	642	500	600	400
25-1	3,000	3,000	5,000[c]	5,000[b]
31	340	320	350	280[b]
33	505	575	100-200	85[b]
34	2,310	2,210	1,155	1,105[b]
36	2,340	2,250	2,300-2,600	2,450[b]
37-3	345	345	400-500	450[b]
40-2	832	832	3,150	3,150
52	4,400	4,100	3,500-5,200	4,100[b]
Total	27,474	23,682	26,555-28,755	24,470

Note: Based on ODFI Reports issued since the 1978 Master Plan for industries within the review study area.

[a]Not all flows would be discharged to sewers.

[b]Estimated from ODFI & Master Plan relationships

[c]Could be reduced to 500 by recycling

Table D-4
SUMMARY OF CONSTITUENTS SHOWN IN
THE 1978 MASTER PLAN

	Eastern Zones (kg/d)	Western Zones (kg/d)	Total (kg/d)	Minimum Concentration[a] (ug/l)
Iron	70.4	50.4	120.8	249
Chromium	0.2	80.1	80.3	155
Copper	14.22	0.2	14.4	27.9
Manganese	--	9.9	9.9	19.2
Zinc	8.3	29	37.3	72.2
Fluoride	0.3	--	0.3	0.58
Sulfide	3.3	--	3.3	6.38
Lead	2.4	6.6	9	17.4
Nickel	2.9	11	14	27.1
Cobalt	--	0.75	1	1.93

Note: Not all industries were sampled, therefore, these are minimum values given the specific conditions at the time of sampling.

[a]At a 1980 east and west area average annual flow of 517 Ml/d if all constituents were discharged to sewers.

Table D-5
ESTIMATED INDUSTRIAL WASTEWATER FLOWS
FOR THE MERIDIAN STUDY AREA
(Ml/d)

	1990		2000	
Sewered Area	Average	Peak	Average	Peak
Western Zone	93	138	140	169
Central Zone	70	42	34	51
Subtotal	170	180	164	246
Eastern Zone[a]	160	240	150	365
Total	180	420	314	561

[a]Includes Manulife plus 9.0 Ml/d from New Gir in 1990 and 18.0 Ml/d from New Gir [illegible]

Table D-6
INDUSTRIAL WASTE SAMPLING PROGRAM
RECOMMENDED ANALYSES

24-hr Composite Samples, One Sample Per Day for 10 Days

BOD
COD
Total solids
Total dissolved solids
Total volatile solids
Suspended solids
Volatile suspended solids
Total organic carbon
Trace organics
Oil and grease
Electrical conductivity
Organic nitrogen
NH_3
NO_3
NO_2
Total alkalinity
HCO_3^-
CO_2

S^{--}
Na
Mg
Zn
Cu
Ni
Cd
Pb
Hg
Cr
pH

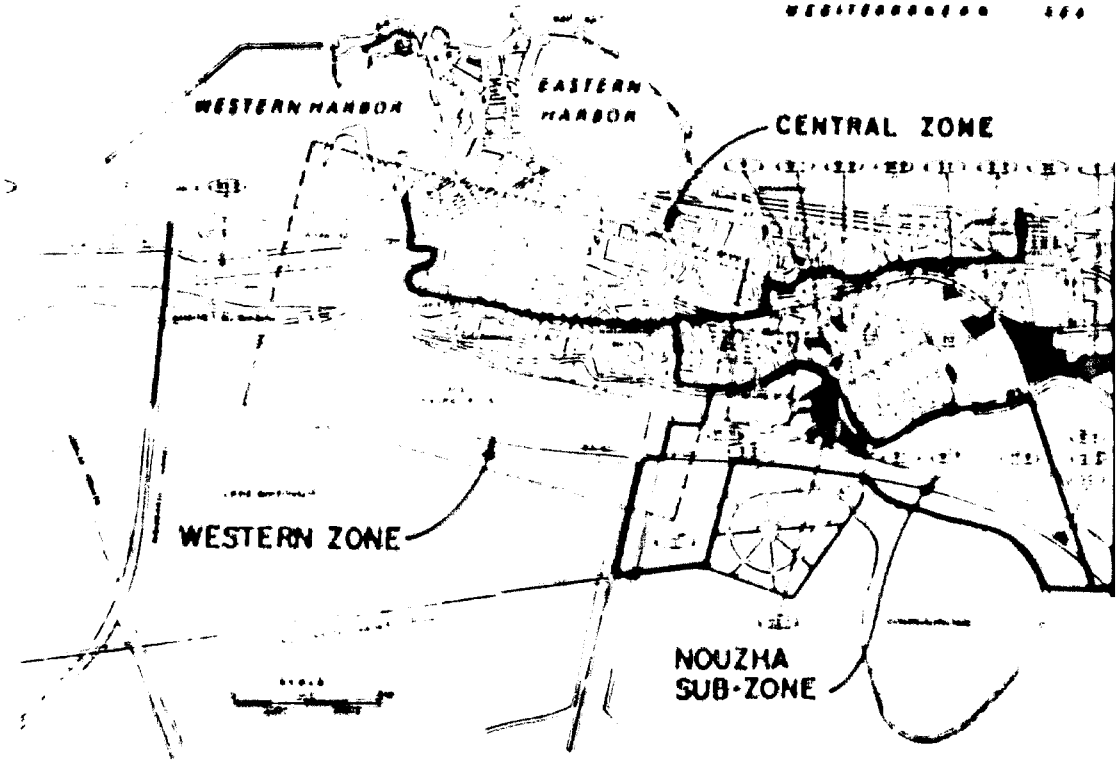
24-hr Composite Samples, Two Samples Collected at Random for Each of 2 Weeks

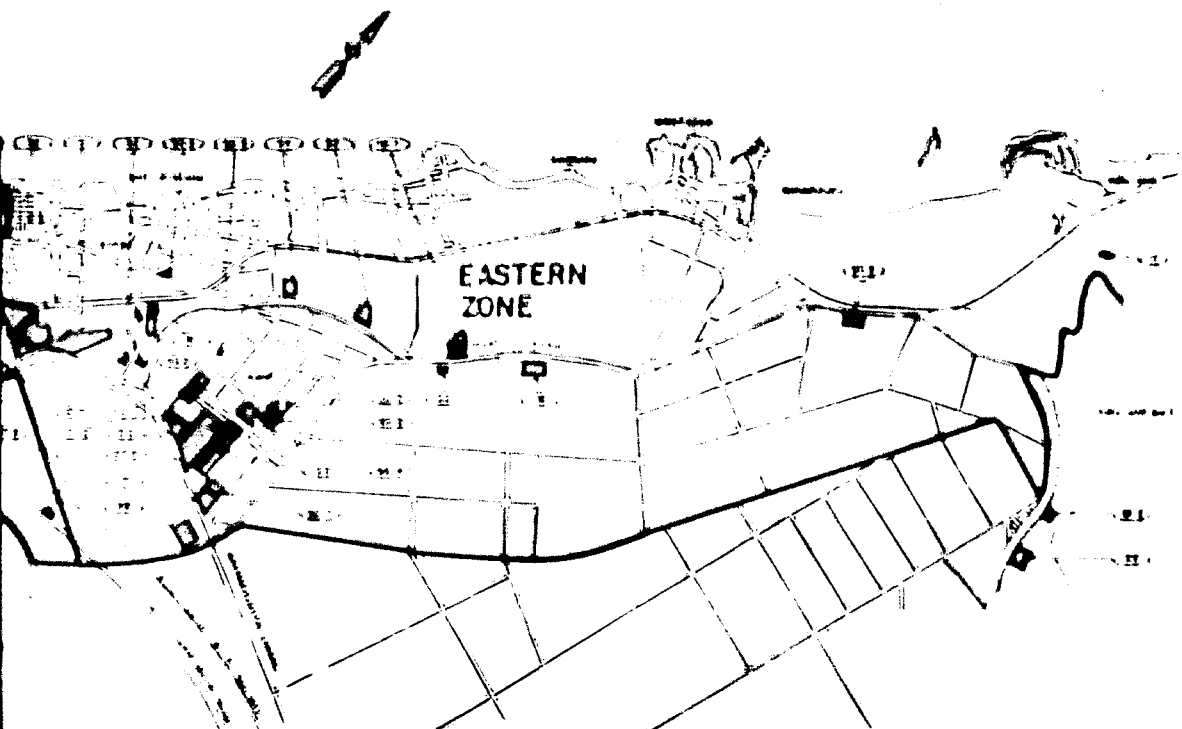
Cl^-
SO_4^{-2}
Ca
K
B

3-hr Grab Samples, Eight Samples per Day for 10 Days

pH
Mineral acidity
Temperature
Electrical conductivity
Suspended solids
Volatile suspended solids
COD

287


WESTERN HARBOR
EASTERN HARBOR
CENTRAL ZONE
WESTERN ZONE
NOUZHA SUB-ZONE

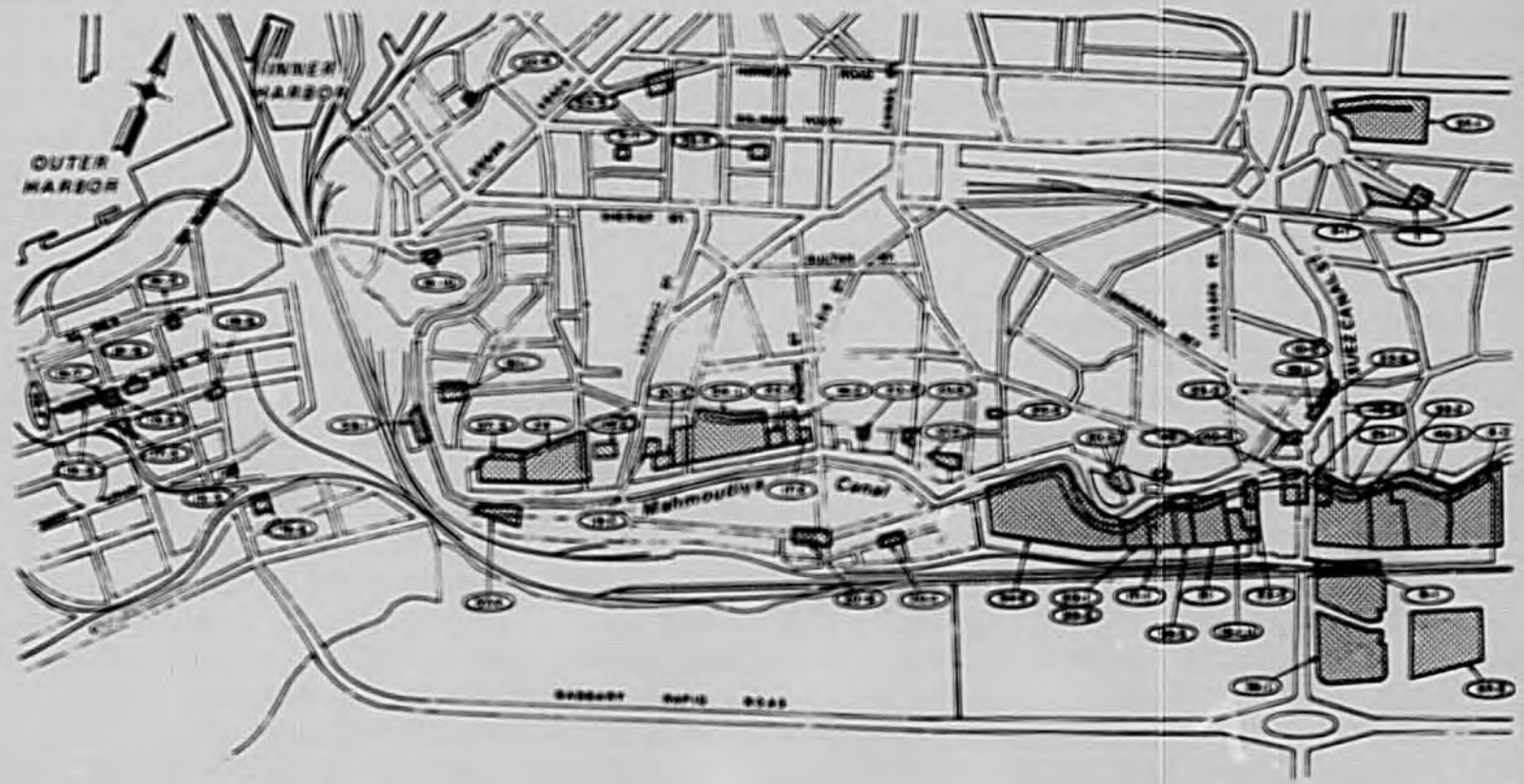


NOTES
SOURCE 1979 MASTER PLAN
SEE TABLE D-E FOR NUMBER
DESIGNATION

1979 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE D-1

MAJOR INDUSTRIES IN ALEXANDRIA



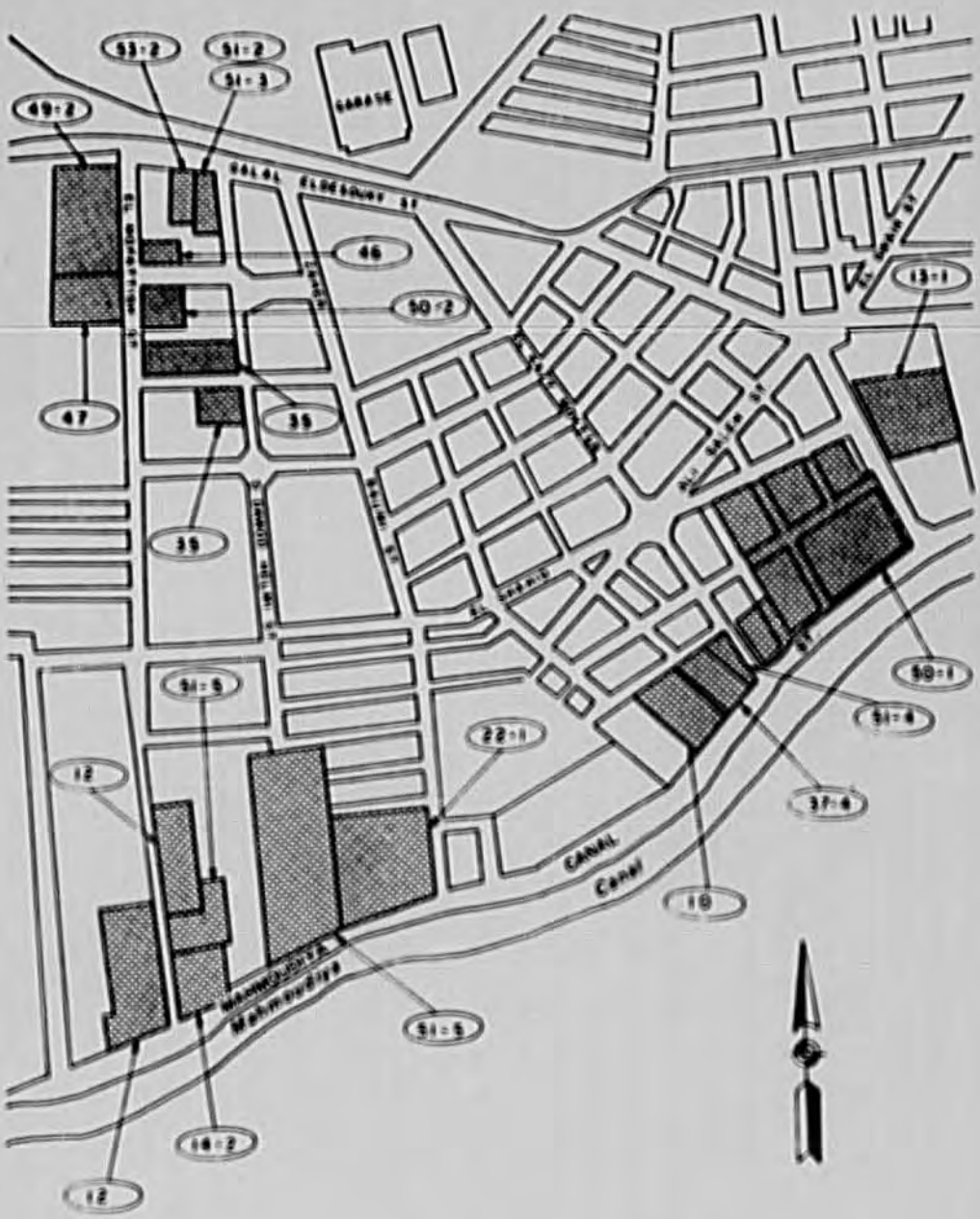
NOTES:
SOURCE: 1978 MASTER PLAN REPORT
SEE TABLE D-2 FOR NUMBER DESIGNATION

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE D-2

MAJOR INDUSTRIES ALONG MAHMOUDIYA CANAL

A/SGMD WWCG W/ECG



NOTES:
SOURCE: 1978 MASTER PLAN REPORT
SEE TABLE D-2 FOR NUMBER DESIGNATION

1977 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE D-3

MAJOR INDUSTRIES IN THE HADARA DISTRICT

A/GOND WWCG W/ECG

APPENDIX E

EAST TREATMENT PLANT EVALUATION

INTRODUCTION

The East Treatment Plant, located near the Hydrodrome in the Smouha area, is proposed to provide secondary treatment for the Nouzha area wastewater regardless of the wastewater [illegible]ment option selected. Discharge will be to the Qala Drain le[illegible] Lake Maryout. This is the recommended arrangement in the 19[illegible] Plan and is appropriate because the plant capacity closely ma[illegible] the year 2000 projected wastewater flows from the Nouzha area, [illegible] is, an average flow of 45 Ml/day and a peak flow of 70 Ml/day. The evaluation reported in this appendix is based on these flow conditions.

Description

The East plant provides primary clarification and activated sludge secondary treatment. Figure E-1 shows a layout of the existing facilities at the East plant, and Figure E-2 is a flow schematic of the plant. Both figures represent the plant when the present construction program is completed.

The incoming wastewater is screened by a manually cleaned bar screen in the influent channel preceding flow measurement by a Venturi flume. Next, grit is removed in six aerated grit chambers and spread on the land around the plantsite. The wastewater then is to be pre-aerated in six new pre-aeration tanks for removal of scum and other floating debris by flotation and skimming. It then receives primary clarification and activated sludge secondary treatment prior to discharge.

Scum from pre-aeration will be trucked to a landfill, and sludge will be air dried and made available as a soil amendment to area farmers. Sludge from the primary clarifiers includes both primary and waste activated sludge.

Basis of Evaluation

The evaluation of the East plant and recommendations for improvements are based on the following influent wastewater characteristics:

- Flow
  - Average--45 Ml/day
  - Peak--70 Ml/day

- BOD
  Average conditions--558 mg/l or 25,100 kg/day
  Peak conditions--1,570 kg/hr

- Suspended solids (ss)--550 mg/l, or 22,500 kg/day

- Ammonia nitrogen--35 mg/l, or 1580 kg/day

The desired effluent quality is 20 mg/l BOD and 30 mg/l SS.

Cost Basis

Capital costs of improvements to the East plant are based on quantity estimates prepared from preliminary designs of the recommended improvements. General conditions concerning capital cost estimating, e.g., time basis for monetary values, foreign exchange rates, customs, construction techniques, etc., are discussed in Appendix B.

Because the East Treatment Plant has operated only periodically, a firm basis for actual operating and maintenance costs does not exist. Therefore, these costs are also estimated according to procedures discussed in Appendix B.

EVALUATION

The following evaluation of the plant's unit processes proceeds according to the wastewater flow through the plant. The text describes the components, evaluates their capabilities, and suggests improvements.

Inlet Works

Influent wastewater enters a riser box and then is discharged into an influent channel. Plant operators report that incoming wastewater splashes over the riser box walls. This should be controlled by covering the riser box with a concrete slab, or redesigning the inlet.

The manually cleaned bar screen in the influent channel is constructed of mild steel and will be deteriorated beyond usefulness in a short time. This bar screen should be replaced with two mechanically cleaned units.

Immediately downstream from the bar screen is a Venturi flume for plant flow measurement. Hydraulic conditions in the inlet channel cause grit to settle at this point; the deposits extend through the flume and

likely render the influent flow measurement erroneous. The flow recording device is also inoperable. The Venturi flume should be abandoned and flow measurement accomplished at the effluent weir at the chlorine contact chamber farther downstream. The entire inlet channel should be covered with grating and surrounded with handrails.

Grit Removal

From the influent channel, the wastewater is discharged to six aerated grit chambers. Stop logs at the inlet and outlet of each chamber are used for isolation. The stop logs can also be removed from the ends of the influent channel to bypass flow around the grit chambers. Control of the water level in the grit chambers is from the primary clarifier flow splitter structure with assistance of the stop logs on the outlet side of the chambers. Operation of the grit chambers would be improved by installation of handwheel-operated slide gates on the inlets and outlets of the chambers, and in the bypass.

Detention time in the chambers at average flow is only 2 minutes, which is less than desired, but no changes are recommended as anaerobic digestion or mechanical dewatering are not used. The grit handling system originally included four grit pumps that discharged to two hydrocyclones located above wheeled vehicles. Ordinarily, one pump served three grit chambers and the second pump on each side was a standby. The pumps are still in place, but the hydrocyclones have been removed. Pumps are of the vertical centrifugal closed-impeller design, but nonclog, recessed impeller pumps would have been more appropriate for this service. The hydrocyclones were reported to have been removed from service because grit-water separation could not be obtained, and the pumps were subject to frequent plugging. Possibly the pump output was not properly matched to the hydrocyclone capacity, as this system has been proven elsewhere, when the grit system is preceded by adequate screening.

From all reports, the grit chambers provide adequate grit removal when operating. The grit handling system should be upgraded by installing the system depicted on Figure E-3. In the improved system, six air-operated diaphragm pumps would continually discharge to two grit concentrators. These would in turn discharge to two grit classifiers followed by wheeled grit storage containers.

Grit chamber air requirements are 740 m^3/hr. At present, air from the same supply as the pre-aeration basins and secondary plant is utilized. The grit chambers and pre-aeration basins, discussed in the following section, should be provided with an air system separate from the secondary plant because of the significantly different air pressure require-

ments of the systems. The individual air diffusers and drop piping in the grit chambers are in poor condition, and should be replaced with manufactured diffusers of the leafspring type. Finally, handrails should be provided around the periphery of the grit chambers complex and the individual basin openings.

Pre-aeration

The pre-aeration system is now under construction. Operation of these units will assist plant operation and performance by facilitating the removal of floatable materials. This will enhance primary clarifier operation. Air requirements will be 2900 m^3/hr, and the air supply for these units is planned to be from the same source as that for the grit chambers and secondary treatment system. This arrangement should be changed so that the air for the grit basins and pre-aeration basins is from a supply separate from the secondary plant. The construction drawings indicate that detention time at peak flow will be nearly 25 minutes. This detention time is sufficient for the intended operation.

If handrails are not to be provided in the current design, they should be added around the entire unit as well as the individual basin openings.

Primary Clarifier Flow Splitting

Discharge from the new pre-aeration basins will be to the primary clarifier splitter structure. Flow splitting is by submerged broad-crested concrete weirs located on the periphery of the concrete riser well. Each quadrant weir discharges to a separate clarifier. The flow split between clarifiers is unequal, because the weirs are submerged. The nearest clarifiers, Nos. 2 and 3, receive the highest flow. The poor flow split, however, should not compromise the primary clarifiers' performance since they are conservatively loaded.

Primary Clarifiers

The primary clarifier tanks appear to be in good condition. They are conservatively loaded with an overflow rate of 16.5 m/day at 45 Ml/day and one unit out of service. The V-notch weirs are made of thin steel, are badly corroded, and should be replaced. The weirs are installed in a peripheral, concentric configuration, and the weir loading is 42 m^3/m/day at a flow of 45 Ml/day. Flow from the center feed well enters the clarification zone through a vertically slotted baffle wall. The openings between the baffles are too narrow and tend to plug. New baffles with larger openings should be installed.

Sludge removal from each clarifier is accomplished by two steel scrapers supported from a rim drive traveling bridge which turns one revolution every 45 minutes. The mechanisms are badly in need of repair and maintenance. The thin, corroded, steel grating on the bridge walkway should be replaced with fiberglass grating. All submerged metal should be sandblasted, primed with polyamid type primer, and covered with a finish coat of coal-tar epoxy paint. Nonsubmerged metal should be sandblasted or wirebrushed and coated with an appropriate enamel paint. The scraper is supported on wheels riding on the clarifier bottom which is badly scored. The wheels should be replaced with a shoe riding on a rail embedded in the clarifier bottom. Also, the space between the clarifier bottom and the scraper needs to be lessened. This could be accomplished by applying a thin layer of grout on the bottom and installing a piece of neoprene rubber or chemically resistant material on the scraper mechanism. These items will be investigated, and a final recommendation made in the basis of design report.

Scum is removed from the clarifier surface by steel scum arms attached to the rotating bridge and is discharged to a steel scum trough in each clarifier. Flushing water is added to the troughs as needed for cleaning. The scum arms and troughs need sandblasting, priming, and a coat of coal-tar epoxy.

Primary Sludge and Scum Pumping

Valves adjacent to each clarifier can be operated to control either sludge or scum discharge. The only motive force available is hydrostatic head. Sludge and scum must flow through nearly 100 m of pipe, 150 to 250 mm in diameter, from the most distant clarifier to a wet well at the sludge pump building. From there, it is pumped to sand drying beds for dewatering. Flushing water is provided to clean the lines. Waste activated sludge (discussed later) is returned to the plant influent for co-settling with primary sludge in the primary clarifiers. Combined sludge from the primary clarifiers and primary scum is discharged to the sand drying beds.

The usual operation technique at this plant is to waste sludge from the clarifiers when the sludge (after settling 30 minutes in a graduated cylinder) occupies 50 percent or more of the sample volume. Problems are experienced with gravity sludge flow to the wet well if sludge is allowed to thicken to a much greater degree in the clarifiers. WWCG's investigation indicates that the wasted sludge flow averages only about two percent solids.

The major problem with the system is caused by the thixotropic nature of sludge. When sludge flow ceases, it settles in the pipes. More energy

than is available is required to restart the flow. Once the sludge begins to flow, the required energy is reduced significantly. This problem intensifies with increased sludge solids concentrations. The available energy is about 2 m of hydrostatic head. Apparently, this head is insufficient to re-establish sludge flow once it stops.

Sludge, concentrated to about four percent solids, could be removed from the clarifiers if the sludge transport system was altered. A schematic of a recommended system is shown in Figure E-4. Four new piston pumps would be needed, each rated at 0.50 m^3 per minute. Two pumps would serve each set of two clarifiers. The piping would be arranged so that either pump in each set of two could pump from either clarifier.

Activated Sludge

The evaluation of the secondary aeration system assumed certain revisions would be made to the aeration system influent and effluent distribution system. At the present time, each aeration bank (a set of six aeration tanks and one pre-aeration tank) receives flow from two of the four primary clarifiers, and discharges to two of the secondary clarifiers. When a primary clarifier is out of service, the other half of the plant, with both clarifiers operating, must handle approximately two-thirds of the flow. The most significant adverse impact of this condition is the additional load placed on the shallow secondary clarifiers. Another adverse impact occurs when one secondary clarifier is out of service. This condition causes the loading to double on the remaining clarifier serving that aeration bank.

The impact of these adverse conditions could be lessened if the effluent from all primary clarifiers is combined and evenly distributed to all aeration tanks in service. Then all operating aeration tanks would receive the same loading, at a given flow, regardless of the number of primary clarifiers in service.

Similarly, if the effluent from the aeration tanks is combined and equally distributed to all secondary clarifiers, the adverse impact of clarifiers being out of service would be minimized. For example, the increased loading to each clarifier (if one other unit is inoperable) would be 33 percent if the changes are made. Under present conditions, the loading is increased by 100 percent to the remaining clarifier serving a bank of aeration tanks when the other clarifier is out of service.

Consequently, WWCG recommends that the effluent from the primary clarifiers be combined and equally distributed to all operating aeration tanks. Further, the effluent from both banks of aeration tanks should

be combined and equally distributed to all four secondary clarifiers. The following evaluation of the secondary aeration system is based on the assumption that these modifications will be made.

In analyzing the aeration system, the following criteria were used:

- Average day plant flow = 45 Ml/day
- Average BOD to aeration = 418 mg/l or 785 kg/hr
- Average BOD to aeration, peak 8 hrs/day = 1200 kg/hr
- Nitrification occurring in the aeration tanks
- Average oxidizable nitrogen to aeration = 25 mg/l
- Average suspended solids to aeration = 200 mg/l
- Suspended solids that are 68 percent volatile
- Volatile suspended solids that are 40 percent non-biodegradable
- For purposes of calculating system stability, all primary and secondary clarifiers and all aeration tanks in service

The activated sludge system is designed for step-feed operation. Because of the shallow secondary clarifiers (a 2-m sidewater depth), the system should be operated at a mixed liquor solids concentration of from 1300 mg/l to 1700 mg/l at the discharge from the basins to prevent overloading the clarifiers with sludge. Based on this condition and a 5000 mg/l return sludge concentration, the following conclusions were drawn:

- The average sludge age should be maintained at approximately 4.4 days when the re-aerated sludge mass is included in the calculation.
- The food-to-microorganism ratio (BOD/aeration basin mixed liquor volatile suspended solids) should average 0.38 when including sludge under re-aeration.
- The average oxygen requirement will be 695 kg/hr per set of aeration tanks, or 1290 kg/hr total.
- The oxygen requirement during the peak 8 hours during the summer will be 800 kg/hour per set of aeration tanks, or 1600 kg/hr total.

- Aeration equipment in each aeration tank should be designed to distribute air at the rate of 260 m^3/min at 0.22 atmospheres (atm) gage pressure based on an actual oxygen transfer rate of 3.28 percent with an aerator submergence 2.3 m deep.

- Air piping in each of the two sets of aeration tanks should be capable of distributing air at the rate of 1600 m^3/min.

- The air supply system must be capable of delivering 190,000 m^3/hr of air at 40°C and one atmosphere pressure for secondary plant requirements.

- The dry weight of waste activated sludge will be approximately 13,800 kg/day at a concentration of 0.5 percent solids from the secondary clarifiers.

Peak air requirements at the East plant will be about 190,000 m^3/hr at 40°C and 1 atmosphere pressure. The four new compressors that are being installed at this plant are each rated at 30,000 m^3/hr at an intake temperature of 40°C. and shaft brake horsepower of 810. The compressor motor is rated at 924 hp, however, and actual operating conditions at the plant will require using this available power in order to satisfy the secondary plant air requirements. Because the required discharge pressure may be only as much as 50 percent of the rated pressure rise, inlet throttling to prevent overloading the motors will be required. Ammeters to monitor the power draw should be provided. By making full use of the available power, approximately 20 percent more air can be delivered from the compressors. This would amount to a total per compressor of 36,000 m^3/hr. Consequently, to satisfy the air requirement, seven compressors, including one standby unit, are needed. Therefore, three more compressors should be purchased.

From the previous analysis of the activated sludge system, the following recommendations are made:

- Increase the air supply to the indicated level.

- Provide new or supplementary air supply and air distribution piping. Calculations suggest that if the air piping is operating under approximately 0.25 atmosphere gage pressure, piping requirements would be:

  - Primary air supply = 1.42 m in diameter

  - Main feeder to each aeration bank = 1.07 m in diameter

- Air header, each tank = 0.6 m diameter

- Install new air diffusion equipment to transfer the indicated oxygen needs.

- Provide three additional 36,000 m^3/hr blowers with 1000-hp drives. These blowers should be chosen to conform to the actual system requirements.

- Provide a water spray system at the aeration tanks to suppress foam.

- Install new slide gates at the aeration basins or repair existing ones.

- Install handrails around all tanks and the periphery of both aeration tanks.

Secondary Clarifiers

The four secondary clarifiers are each 34 m in diameter and have a sidewater depth of 2 m. These units should be adequate to meet plant needs if there is little downtime for any of them and if the mixed liquor suspended solids concentration is maintained between 1300 and 1700 mg/l.

The clarifiers are more shallow than is normal for activated sludge clarifiers when receiving commonly accepted solids loadings of up to 82 kg/m^2/day at average flow. For this reason, the secondary plant should be operated so that mixed liquor to the clarifiers will be in the 1300 to 1700 mg/l range. This will result in solids loadings of 28.5 kg/m^2/day with all units operating. The previously recommended changes to the aeration basin effluent distribution should be made to equalize the loading to each clarifier when one unit is out of service. If these changes are not made and one unit is removed from service, then the second unit, serving that half of the plant would receive a solids loading of 60 kg/m^2/day when the mixed liquor suspended solids concentration is 1500 mg/l and the return sludge concentration is 5000 mg/l. This is somewhat high for the shallow clarifiers and may result in excessive solids loss over the weirs into the final effluent. If the changes are made, then the loading to each clarifier will only be 38 kg/m^2/day, which will lessen the chance of excess solids being discharged to the effluent.

Clarifier surface loadings are comfortably low at 12.5 m/day at average flow with all units in service. Peak loadings will be 19.4 m/day with

all units in service. If the aeration basin effluent distribution modifications are not made, then with one unit out of service and a second unit receiving half of the plant flow, the peak loading will be 39 m/day. This is an uncomfortably high loading, considering the associated high solids loading of 60 kg/m^2/day, for the shallow clarifiers. Again, sludge blanket loss may occur. However, if the aeration basin effluent distribution changes are made, then the surface loading to all units would be 26 m/day, which could be accommodated by the existing units with little or no effluent deterioration.

The clarifiers are provided with double concentric, peripheral weirs. Weir loadings will be conservatively low under all loading conditions. Care must be taken to maintain the weirs at the same elevation in all of the clarifiers. Failure to do so will result in unequal loadings to the shallow clarifiers. Since the units are so shallow, the sludge blanket will not be far below the weirs and localized high weir flows will likely cause some sludge blanket loss. Since an equal flow split among the secondary clarifiers is dependent upon equal hydraulic conditions, the clarifier weirs must be at the same elevation.

Each clarifier is equipped with two steel sludge scrapers supported from a bridge that is equipped with a rim drive. This type of sludge removal mechanism is not suited for its intended use. Once relatively efficient and stable operation is achieved in the secondary plant, nitrification will almost surely occur in the aeration tanks. Unless the sludge can be rapidly removed from the shallow secondary clarifiers, denitrification will occur, resulting in sludge flotation and loss over the effluent weirs. The threat is compounded by the warmth of the wastewater during the summertime, making nitrification and denitrification almost a certainty in this plant. For this reason, the existing sludge removal mechanisms should be modified to permit rapid sludge removal or replaced by rapid sludge removal "suction" type machines.

Secondary Sludge Pumping

When the secondary plant was operated in the past, return sludge was pumped only intermittently to the re-aeration tanks. Return sludge pumping was alternated with waste sludge pumping. This practice will not result in satisfactory operation; rather, sludge return must be continuous and at a rapid rate. Sludge wasting should be used to maintain a preset mixed liquor solids inventory under aeration and may be continuous or intermittent. If intermittent, it should occur at least 3 times per day for at least 2 hours per event.

Firm return sludge pumping capacity should be 455 l/sec. The average return sludge flow requirement is expected to be over 220 l/sec. Return

sludge flow rate should be automatically varied with plant influent flow rate to maintain a return sludge solids concentration of approximately 5000 mg/l. The existing secondary sludge pumping system should be replaced with a new facility similar to that depicted schematically on Figure E-5. Each of the four return activated sludge pumps should be a screw pump rated at 230 l/sec. The waste activated sludge pumps should be rated at approximately 20 l/sec each. Some or all of the existing sludge pumps may be usable in the recommended system. This possibility should be investigated as part of the study for the basis of design report.

Effluent Disinfection

Effluent disinfection is not practiced now, but it should be provided for public health reasons. Consequently, disinfection capabilities should be installed using the following criteria:

- Method--chlorination
- Maximum dose at peak flow--20 mg/l, 1400 kg/day
- Average dose--5 mg/l, 225 kg/day
- Feed system--evaporators and chlorinators with the dose manually set and automatically paced according to plant flow
- Evaporators required--2 (one redundant)
- Chlorinators required--2 (one redundant)
- Chlorine contact volume--970 m^3 in two chambers of minimum length to width ratio of 40:1

Sludge Management

After completion of the 36 sand drying beds now under construction, there will be 32,400 m^2, or just over 3 ha in the total 72 beds. Anticipated sludge will be 13,500 kg/day primary and 13,800 kg/day of waste activated sludge for a total daily dry solids wastage of 27,300 kg/day. Assuming 2 percent solids in the waste sludge withdrawn from the primary clarifiers, as is believed to be presently the case, 18 ha of sand drying beds will be required, including support facilities. In addition, 5 ha will be needed for storage of sludge for 1 to 2 years to permit decay of pathogenic organisms prior to distribution to farmers. Therefore, a total of approximately 23 ha will be needed. Only about

20 ha are available at the present plantsite on which these facilities can be located. For this reason, the suggested changes to the primary sludge pumping system must be made so that a higher sludge concentration can be obtained. If a 4-percent solids concentration can be obtained, then the drying bed and support facilities would have to be expanded to only 9.2 ha. With 5 ha needed for long-term storage, the total land requirement is 14.2 ha. Sand drying beds similar to the current design are appropriate.

Electrical System

Figure E-6 is a one-line diagram of the existing plant power distribution system. Electrical power is supplied to the East Treatment Plant through two underground aluminum cables by the Alexandria Company for Electric Distribution (utility) from a 10.5-kV power distribution center located on the plantsite. One of the existing underground cables is being replaced and the second cable is not connected. The power distribution center is supplied power from only one substation.

The current carrying capability of each of the two existing cables at 10.5 kV is approximately 300 amperes. This will not provide adequate capacity to supply the future load requirements of the plant. The two cables are connected from fused interrupter switches in the power distribution center to main oil circuit breakers in a 10.5-kV switchboard located in the blower building. The 10.5-kV switchboard is clean and appears to be in excellent condition. However, because the d.c. system is inoperative, the only fault current protection that currently exists is provided by fuses in the main circuit breaker assembly. The switchboard is capable of meeting plant needs now but not in the future. The switchboard can be connected to either service cable or both at once, but since the switchboard bus is not divided, the loads cannot be selectively divided between the two services. The main bus in the switchboard is copper with a current carrying capability of approximately 230 amperes continuously. The oil circuit breakers are rated 400 MVA, 600 amperes at 10.5 kV. Overcurrent and fault current protection is supposed to be provided by a d.c.-operated control scheme that is presently inoperative; it must be repaired or replaced.

The 10.5-kV switchboard and the service cables from the power distribution system do not have adequate capacity to supply the needs of the treatment plant in the future. The copper bus bars in the switchboard must be replaced with new bus bars that have a minimum current carrying capacity of 400 amps. The service cables must be replaced with new cables that have matching current carrying capacity. The settings of all protective relays will need to be changed to correspond with the requirements of the new equipment ratings. Two new 10.5-kV oil circuit

breakers will be required for two additional transformers that must be installed.

3000-Volt System. The 10.5-kV switchboard presently feeds three transformers to provide power at 3000 volts for operation of the blowers. One of the transformers is rated 750 kVA, 10.5 kV-3000 volt, three-phase, and the other two are rated 800 kVA, 10.5 kV-3000 volt, three-phase. Each transformer has voltage taps at ± 2½ and 5 percent. The transformers and their associated primary and secondary cables are not of adequate size to serve the additional blower loads previously recommended.

A design and construction program is currently underway that will install four new 685-kW blower motors and four 1000-kVA transformers; see Figure E-7 for a one-line diagram of the proposed system. These transformers will not have the capacity needed to power the six operating blowers that will be required. Two additional 1000-kVA transformers will be required to assure that adequate capacity will be available to operate the six blowers with one transformer out of service. See Figure E-7 for a one-line diagram of the proposed system. Care will need to be taken when specifying the transformers and installing the cable systems to assure that all of the transformers can be operated in parallel.

The 3000-volt transformers can all be connected in parallel to a 3000-volt switchboard. The 3000-volt switchboard is clean and appears to be in excellent condition and adequate to supply present plant needs. The switchboard has three main circuit breakers, three blower feeder circuit breakers, and two spare circuit breakers. The circuit breakers are oil type rated 400 MVA, 600 amperes at 3000 volts. Overcurrent and fault current protection is intended to be provided by a d.c.-operated control scheme that is presently inoperative. The switchboard has copper bus bars capable of carrying 1000 amperes continuously.

The configuration of the existing switchboard will be modified during the construction program to install new blowers. Figure E-7 is a one-line diagram of the system showing the new transformers. Figure E-8 is a one-line diagram of a system that will provide power to seven blower motors. The system will require seven additional oil circuit breakers or seven high-voltage motor starters, one of the starters would be connected to two bus sections by a two-position nonload break selector switch. The existing switchboard will also require other modifications.

380-Volt System. The switchboard presently feeds three transformers to serve the miscellaneous 380 wye/220 volt loads. The transformers are rated 500 kVA, 10.5 kV-380 wye/220 volts, three-phase and have voltage taps at ±2½ and 5 percent. The feeder cables from the 10.5-kV switch-

boards to the transformers each have a current carrying capability of approximately 120 amperes or 1250 kVA at 10.5 kV. The secondary feeders from the transformers to the existing switchboards consist of two conductors per phase with a current carrying capacity of approximately 500 amperes or 328 kVA at 380 volts. The capacities of these feeders are adequate for present and future loads.

Two of the three 380 wye/220 volt transformers can be connected in parallel to a 380-volt main switchboard at any time. The 380-volt main switchboard is of an old design that uses manually operated switches in series with fuses as feeder protective devices. The doors of the switchboard must be opened, exposing open bus bars, for the switch handles to be accessible. The switchboard includes a group of plug fuses for lighting and outlet circuits and a d.c. distribution section for control power for the 10.5-kV and 3000-volt switchboards. The 380-volt switchboard should be replaced with a newer, more modern one that has sufficient capacity for future plant needs.

Direct buried underground cable feeders are provided from the 380-volt switchboard in the blower building to a switchboard and motor control panel in the sludge pumping building, to control centers in the grit pump stations, and to distribution panels in other buildings where 380-volt or 220-volt power is required.

Grit Chambers. The grit chamber pumps are supplied power from and controlled by control panels located in each grit pump room. The control panels are old and in bad condition and should be replaced with new equipment of modern design. The associated electrical wiring should be replaced when the motors and control panels are replaced. Electrical switchboards and control panels would be provided for the grit handling equipment as part of the electrical system modifications.

Grit Basin and Pre-aeration Basin Compressors. The proposed electrical system modifications will include switchboards and control panels for the new pretreatment blowers.

Primary and Secondary Clarifiers. The primary clarifiers can be controlled from either the motor starters located in the sludge pumping building or from control stations located on the platform between each pair of clarifiers. The controls at the clarifiers are mostly disconnected and the wiring is generally inadequate. New control stations should be provided at the clarifiers and connected into new control circuits provided as part of new motor starter assemblies.

The slip ring assemblies on the clarifiers are worn but appear to be serviceable. New brushes should be installed, the contact surfaces dressed, and the wiring reconnected. Existing control and power cables

should be reused unless megger readings taken at the time of reconstruction indicate that insulation failure is likely.

Sludge Pumping Building. The sludge pumping building houses the only standby engine generator at the plant as well as the switchboard and motor control panel supplying the clarifiers, sludge pumps, and other miscellaneous pumps. The switchboard and motor control panel are old, of antiquated design, in questionable condition, and are inadequate in capacity and space to accommodate the needs of the proposed modifications. They should be replaced with a new motor control center having adequate capacity and space for present and future needs of the plant, and tailored to the needs of the proposed modifications. These modifications include the new primary sludge pumps, return activated sludge pumps, and the waste activated sludge pumps.

Existing control and power cables should be reused where existing motors and equipment are retained unless megger readings taken at the time of reconstruction indicate that insulation failure is likely. New control and power cables should be installed for all new or replaced equipment.

Underdrain Pump Station. The existing motor control panel in the underdrain pump station is in poor condition and should be replaced. Existing power cables appear to be in good condition and should be retained unless megger readings taken at the time of reconstruction indicate that insulation failure is likely.

Disinfection. The necessary electrical system components for the chlorination system would be provided under the proposed system modifications.

Standby Power. Standby electrical power is currently provided at the East Treatment Plant in two ways. First 380-volt, three-phase power is provided for operation of the sludge pumps and clarifier drives by a 400-kW engine generator located in the sludge pumping building. The engine generator must be started manually. Second, electrical power can be supplied to the plant's main switchboard through either of two underground primary cables from the utility at 10.5 kV. Three transformers, each with their own primary and secondary breakers, have been provided to supply electrical power at 3000 volts for blower operation. Any two of these transformers are capable of providing adequate capacity to operate all three existing blowers. Three transformers, each with their own primary and secondary breakers, have been provided to supply electrical power at 380 wye/220 volts for general equipment operation. Any one of these transformers has capacity to supply all of the remaining plant loads.

The reliability of standby power supplied by duplication of equipment is limited because of several reasons:

- Power to both underground 10.5-kV service cables is supplied by one underground feeder from the utility's substation.

- Both 10.5-kV services and all of the transformers are connected to the same 10.5-kV switchboard. The likelihood of failure within the switchboard is low, however.

- All three 3000-volt transformers and all three blowers are connected to the same 3000-volt switchboard. The likelihood of a failure within this switchboard is also very low.

- All three 380 wye/220 volt transformers and all of the low voltage loads are connected to the same 380-volt switchboard. Again, likelihood of failure within the switchboard is low.

Continuous operation of the blowers supplying air to the aeration basins and the pumps returning sludge to the aeration basins from the secondary clarifiers is critical to the success of an activated sludge waste treatment plant. To assure continuous operation of the blowers, either an onsite power generation station rated at approximately 7000 kW or a reliable power supply to the site is essential.

WWCG recommends that two dedicated feeders from two separate utility substations be provided to assure a reliable power supply for the site. Each utility substation has several transformers that are separately supplied from a 66-kV double circuit transmission loop along the south side of the city.

To further increase the reliability of the system, WWCG proposes that the 10.5-kV switchboard bus and the 3000-volt switchboard bus be divided and the loads reconfigured as shown in Figure E-8. Either one of two existing 500 kVA, 10.5 kV-380 wye/220 volt transformers, one connected to each half of the 10.5-kV bus, will provide adequate reliability for the low voltage needs of the plant. Three of the six new 1000 kVA, 10.5 kV-3,000 volt transformers should be connected to each half of the 10.5-kV bus. Three transformers connected in parallel can operate three blowers in a non-overloaded condition. Three transformers can operate four blowers for a limited time in an overloaded condition. Key interlocking will have to be provided to assure that the utility services cannot be connected together through the plant's electrical system.

Plant Instrumentation. The existing plant instrumentation installed as part of the plant construction is inoperative. Much of the equipment is of antiquated design and spare parts are not readily available.

New, modern instrumentation, easily operated and maintained, will be installed as part of the proposed reconstruction and upgrading project.

The new system will include equipment to monitor all critical functions and to present the information for the operators' use where it is required for control and at a central location where the operation of the entire plant can be monitored. Recording instruments will be installed for critical flows.

PLANT CAPABILITY

If the suggested changes to the East plant are made, it should be capable of accommodating the following wastewater characteristics:

o	Average dry weather flow	45 Ml/day
o	Peak flow	70 Ml/day
o	BOD, average	25,100 kg/day
o	BOD, peak 8-hour	1570 kg/hr
o	Suspended solids	22,500 kg/day
o	Ammonia nitrogen	1580 kg/day

Under these conditions, achievable effluent quality should be in the range of 20 to 30 mg/l for both BOD and suspended solids. Table E-1 shows the design criteria for the East plant's unit processes with the recommended improvements, and Figure E-9 shows the plant layout.

RECOMMENDED IMPROVEMENTS

Based on the previous analysis of the East plant, the following improvements are recommended:

- o Inlet works
  - Storage containers for screenings (2)
  - Handrails and grating
  - Cover or redesign inlet
- o Grit chambers
  - Handwheel-operated gates on inlets, outlets and bypass (13)
  - Air-operated diaphragm pumps for grit (8)
  - Concentrators (2)
  - Classifiers (2)
  - Storage containers (3)

- New air drop piping
- New air diffusers
- Handrails
- New air supply system

o Pre-aeration basins
  - Handrails
  - New air supply system

o Primary clarifiers
  - New weir plates
  - New inlet baffles with 10-cm openings
  - New fiberglass grating for bridges
  - Mechanism painting
  - Scraper modifications

o Primary sludge pumping
  - Sludge and scum wells (4)
  - Piston pumps (4)

o Modifications to aeration basin influent and effluent distribution

o Plant air supply
  - New blowers with a 36,000 m^3/hr capacity (3)

o Secondary aeration system
  - Air supply main piping
  - Air supply piping to aeration banks
  - Air supply piping to aeration tanks
  - Air diffusers
  - Water spray system to control foam
  - Repair or replace slide gates
  - Handrails

o Secondary clarifiers
  - New rapid sludge removal mechanisms (4) or a retrofit of existing units

o Return sludge pumps
  - Screw pumps (4)

o Waste activated sludge pumps
  - Nonclog centrifugal pumps (3)

o Effluent disinfection
  - Contact chamber

- Evaporators (2)
- Chlorinators (2)

o Sand drying beds
  - An additional 5.6 ha including support facilities

o Long-term sludge storage
  - An additional 3 ha

o Electrical system improvements
  - 10.5-kV switchboard additions and modifications
  - 3000-V switchboard additions and modifications
  - 10.5 kV-3,000 volt transformers (3)
  - Repaired d.c. power system
  - 380-volt switchboard
  - Primary and secondary clarifier control stations
  - Repaired clarifier slip ring assemblies
  - New control and power cables
  - Pump house motor control center
  - Grit pump controls
  - Underdrain pump station motor control center

o Instrumentation and control system
  - A complete new system

COST ESTIMATES

Table E-2 lists the capital costs for the recommended improvements to the East plant. The costs are in terms of end-of-1980 Egyptian pounds (LE) and are of conceptual level accuracy. They are based on quantity estimates of materials and equipment required for the improvements.

Table E-3 shows the estimated operation and maintenance costs for the East Treatment Plant with the recommended improvements.

Table E-1
PROPOSED DESIGN CRITERIA FOR THE
EAST TREATMENT PLANT

Item	Design Criteria
General Criteria	
Treatment capacity	45 Ml/day average, 40 Ml/day peak
Standby power	Sufficient for all plant processes; redundant power source
Power supply	3000 volt and 380 volt, 3-phase, 50 HZ
Inlet Works	
Bar screens	
Process	Mechanically cleaned
Clear space	2 cm or less
Velocity, max.	1.2 m/s
Depth, max.	1.5 m
Width of screen	0.85 m
Screenings discharge	Wheeled container
Drainage	Returned to plant flow
Disposal	Landfill
Flow measurement	Effluent weir
Grit removal	
Process	Aerated grit chambers, existing
Removal equipment	Pump suction, existing design
Chamber depth	3 m, existing design
Length:width ratio	2.34:1, existing design
Detention time	2 minutes, existing design
Air supply	6 l/s per metre of tank length
Grit cleaning and dewatering	By concentrator followed by a classifier
Oil and grease flotation	
Process	Aeration
Detention at peak flow	18.6 minutes with 1 of 6 basins out of service

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Table E-1 (continued)
PROPOSED DESIGN CRITERIA FOR THE
EAST TREATMENT PLANT

Item	Design Criteria
Oil and grease flotation (continued)	
Chamber depth	2.95-m water depth, existing design
Chamber width	5.24 m, existing design
Chamber length	20 m per basin, 6 basins
Air supply	1.0 m^3/m^3 wastewater/day
Skimming discharge	To tank truck
Skimming disposal	To landfill
Primary Clarification	
Process	Circular, existing design
Diameter	34 m
Side water depth	2 m
Surface loading	16.5 m/day @ average flow, 1 unit down 25.6 m/day @ peak flow, 1 unit down
Detention time, average	6 hrs
Weir loading, average	42 $m^3/m/day$
Sludge/scum removal	Rotating rim-drive scraper and skimmer, existing
Sludge pumping	Piston pumps
Scum discharge	With sludge
Scum disposal	To drying beds with sludge
Activated Sludge System	
Process	Step-feed with sludge re-aeration, existing design
Mixed liquor suspended solids	1300 mg/l-1700 mg/l, limited by shallow depth of existing clarifiers
Sludge age	4.4 days including re-aerated sludge
Food-to-microorganism ratio	0.38 based on aeration basin and re-aeration basin mixed liquor volatile suspended solids
Detention time	10.6 hrs
Nitrification	In aeration basins; oxidizable nitrogen = 25 mg/l

Table E-1 (continued)
PROPOSED DESIGN CRITERIA FOR THE
EAST TREATMENT PLANT

Item	Design Criteria
Secondary Clarification	
Process	Circular, existing design
Diameter	34 m
Side water depth	2 m
Surface loading	16.5 m/day @ average flow, 1 unit down 25.6 m/day @ peak flow, 1 unit down
Detention time average	6 hrs
Weir loading, average	42 m^3/m/day
Sludge removal	Rapid sludge removal type
Firm return sludge pumping rate	39 Ml/day
Return sludge rate control	Automatically varied with influent flow
Return sludge pumping	Screw pumps
Firm waste sludge pumping rate	60 l/sec
Waste sludge pumping	Nonclog centrifugal
Effluent Disinfection	
Process	Chlorination
Dose, max. at peak flow	20 mg/l
Dose, average	5 mg/l
Feed system	Evaporators and chlorinators manually set, automatically paced to influent flow
Chlorine storage on site	10 days in tonne cylinders
Chlorine contact time	20 min at peak flow
Contact basin length to width ratio	40:1
Sludge Dewatering	
Process	Sand beds
Loading	3.17 m/yr total
Underdrainage	Return to plant
Dewatered Sludge Storage	1 to 2 years
Sludge Disposal	Use on farmland

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Table E-2
CAPITAL COST ESTIMATE OF RECOMMENDED
EAST TREATMENT PLANT IMPROVEMENTS
(end of 1980)

Item	Estimated Cost (LE)[a]
Inlet works	455,000
Grit chambers	192,000
Pre-aeration basins	85,000
Primary clarifier	89,000
Primary sludge pumping	122,000
Aeration basin influent and effluent distribution	387,000
Secondary plant air supply	1,983,000
Secondary aeration system	548,000
Secondary clarifiers	789,000
Effluent disinfection	384,000
Return and waste activated sludge pumping	58,000
Sludge management	5,947,000
Electrical and instrumentation and control system improvements	202,000
Subtotal	11,241,000
Contingency @ 25%	2,810,000
Subtotal	14,051,000
Engineering, legal, and administrative costs @ 15%	2,108,000
Total	16,159,000

[a]Excluding customs charges

Table E-3

EAST TREATMENT PLANT ESTIMATED ANNUAL O&M COST SUMMARY

Item	1980 Quantity	1980 Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	11,125		11,125		11,125	
Cost		72		72		72
Power						
1,000,000 kWh	28,845		28,845		28,845	
Cost		256		256		256
Fuel						
1000 litres	12		12		12	
Cost		1		1		1
Chemicals						
Tonnes	93		93		93	
Cost		35		35		35
Materials						
Cost		506		506		506
Total						
Cost		870		870		870

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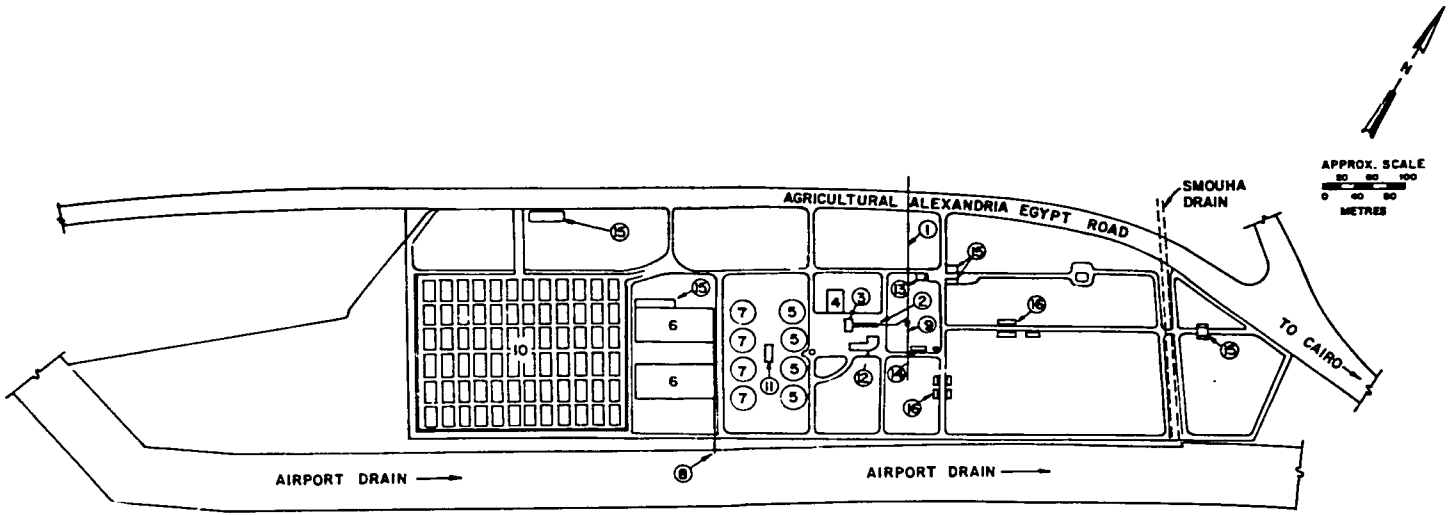

1. INFLUENT FORCE MAIN
2. INLET WORKS
3. AERATED GRIT BASIN
4. PRE-AERATION BASINS
5. PRIMARY CLARIFIERS
6. AERATION BASINS
7. SECONDARY CLARIFIERS
8. EFFLUENT LINE, EXISTING
9. BYPASS TO DRAIN
10. SLUDGE DRYING BEDS
11. PUMP HOUSE
12. MACHINERY BUILDING
13. LABORATORY AND ADMINISTRATION BUILDING
14. INDUSTRIAL LAB
15. STORAGE BUILDING
16. HOUSING
17. CHLORINE BUILDING, PROPOSED
18. CHLORINE CONTACT BASIN, PROPOSED
19. EFFLUENT LINE, PROPOSED

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FIGURE E-1

EXISTING LAYOUT OF THE EAST TREATMENT PLANT

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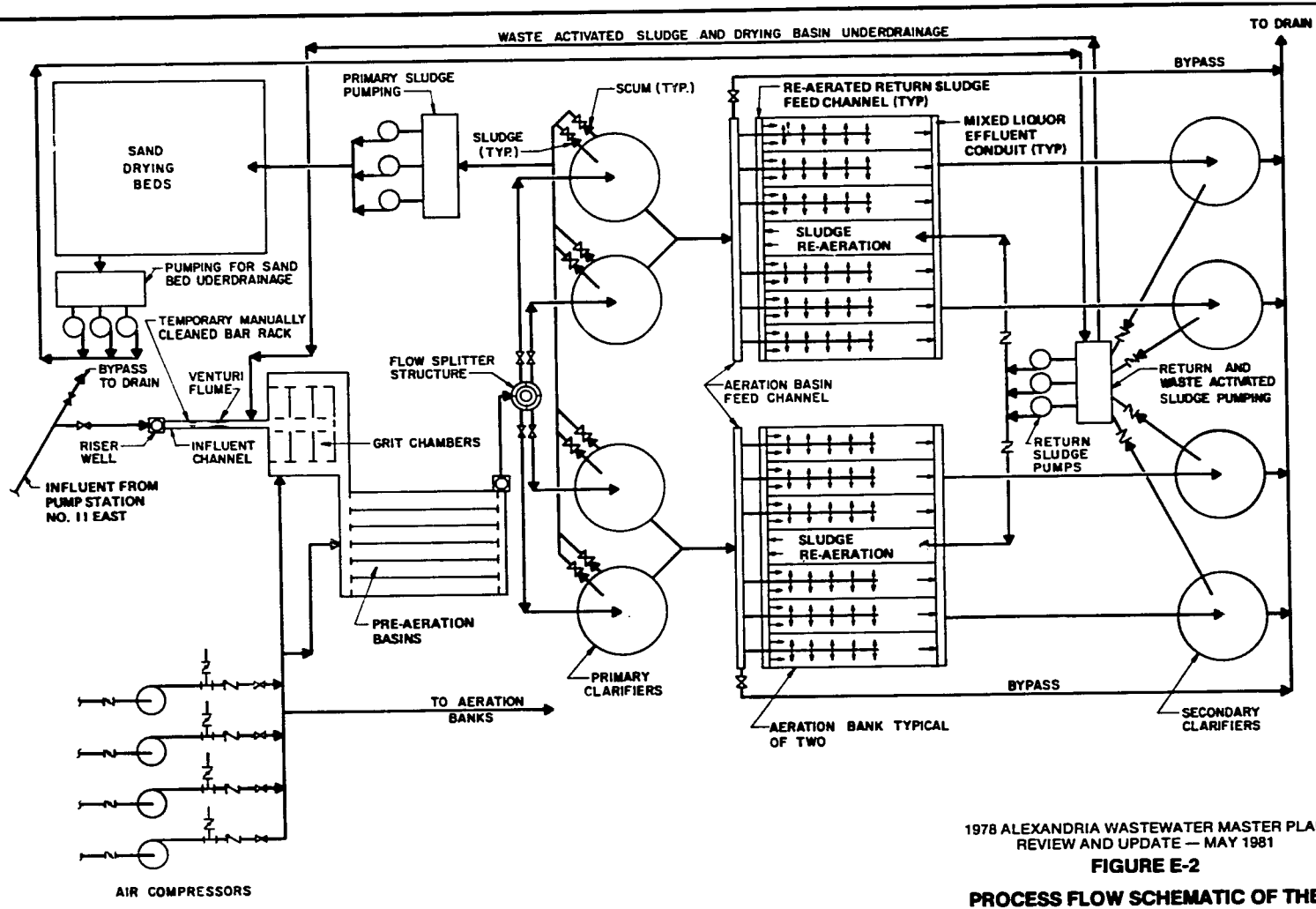

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FIGURE E-2

PROCESS FLOW SCHEMATIC OF THE EAST TREATMENT PLANT

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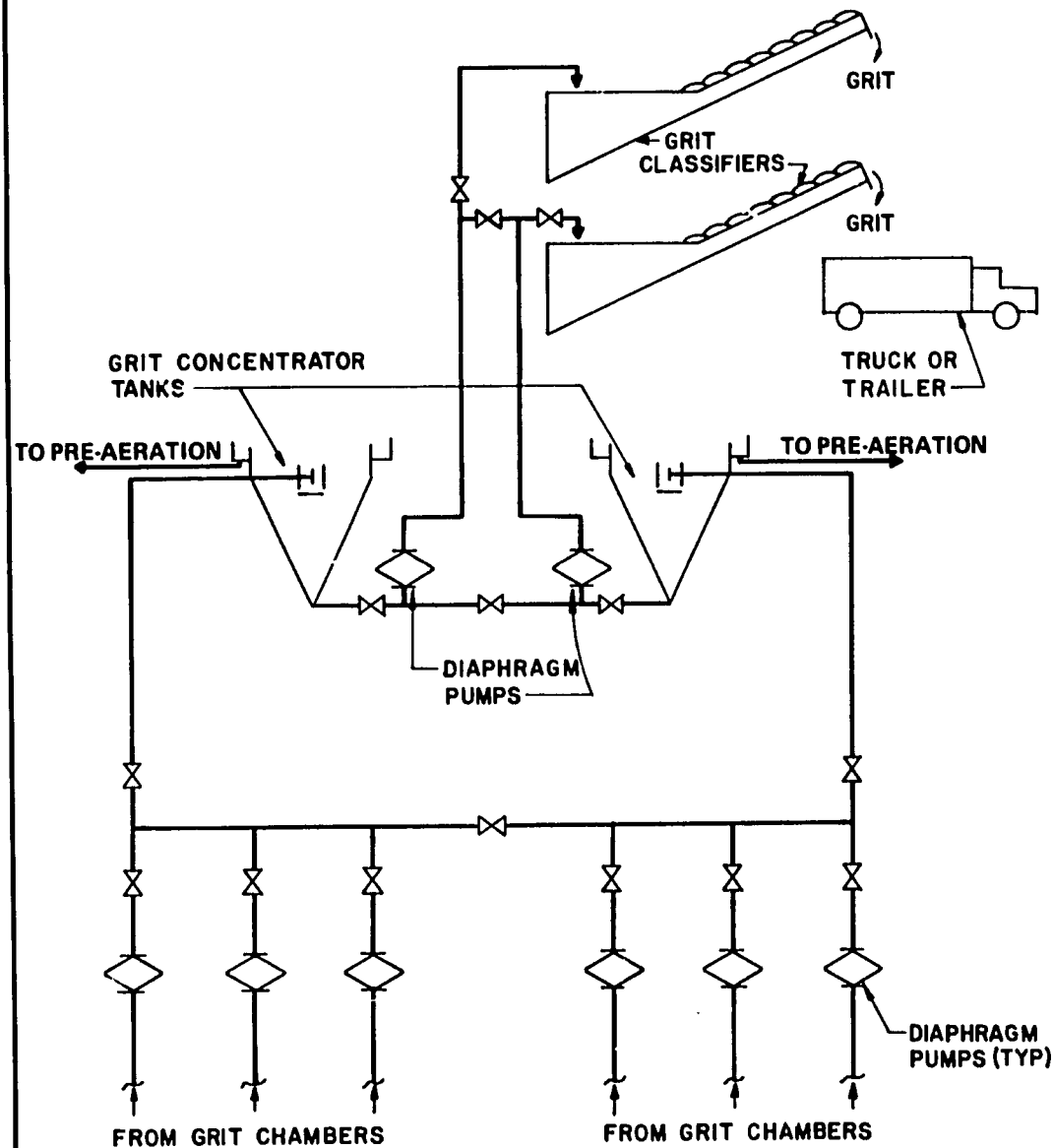

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FIGURE E-3

PROPOSED GRIT HANDLING SYSTEM

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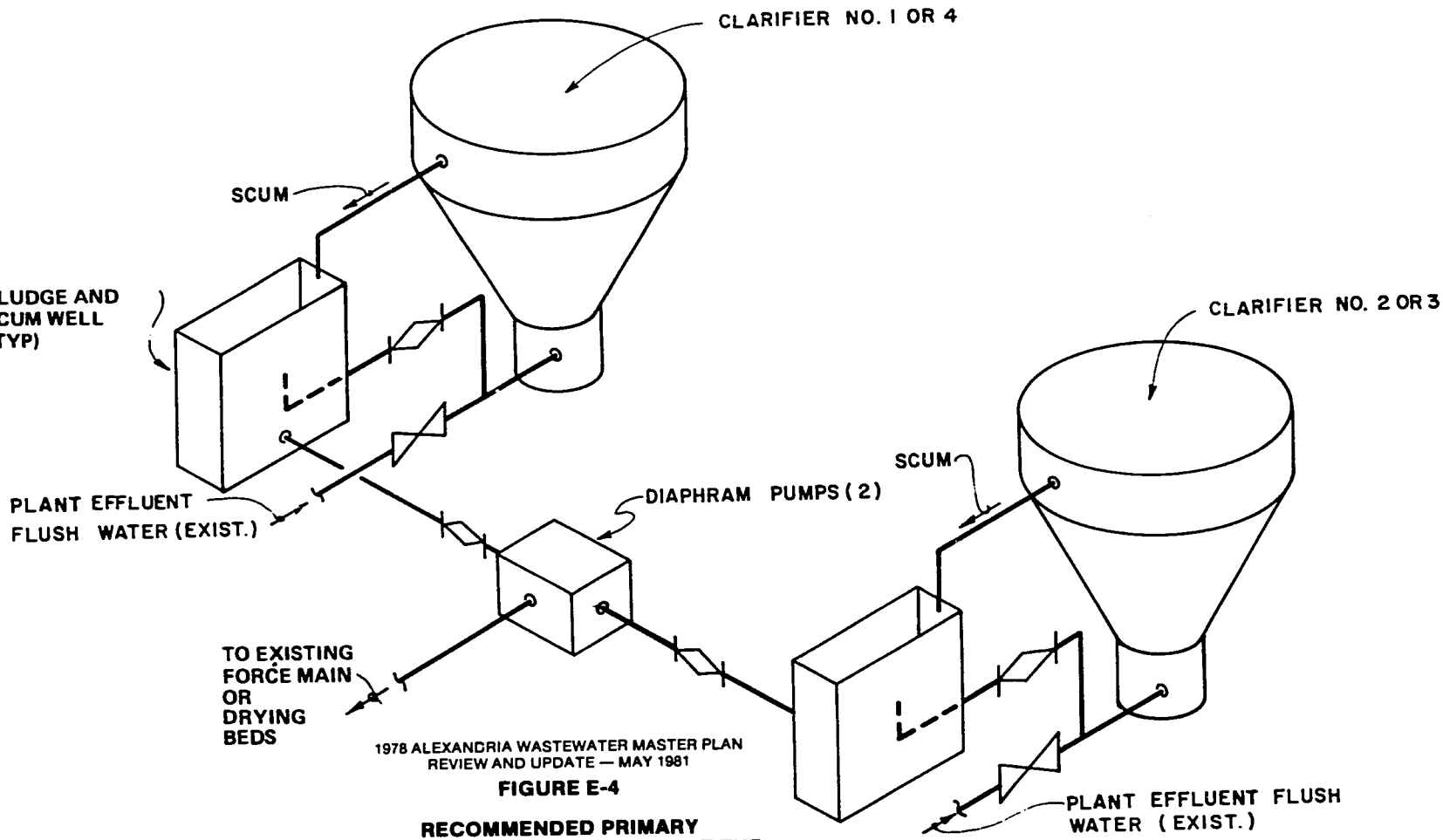

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FIGURE E-4

RECOMMENDED PRIMARY SLUDGE TRANSPORT SYSTEM AT THE EAST TREATMENT PLANT

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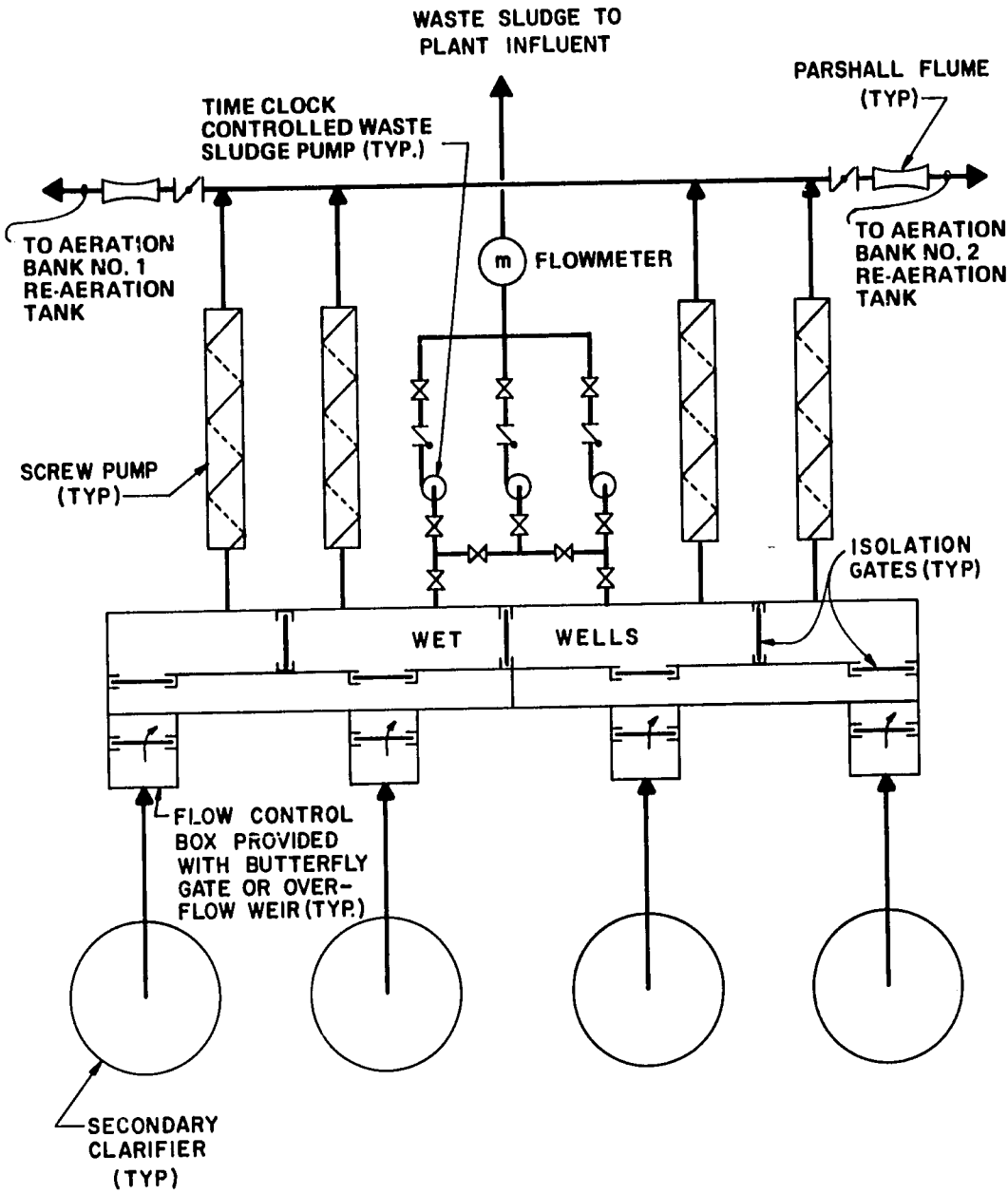

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FIGURE E-5

PROPOSED SECONDARY SLUDGE PUMPING AT THE EAST TREATMENT PLANT

A/GOSD WWCG W/ECG

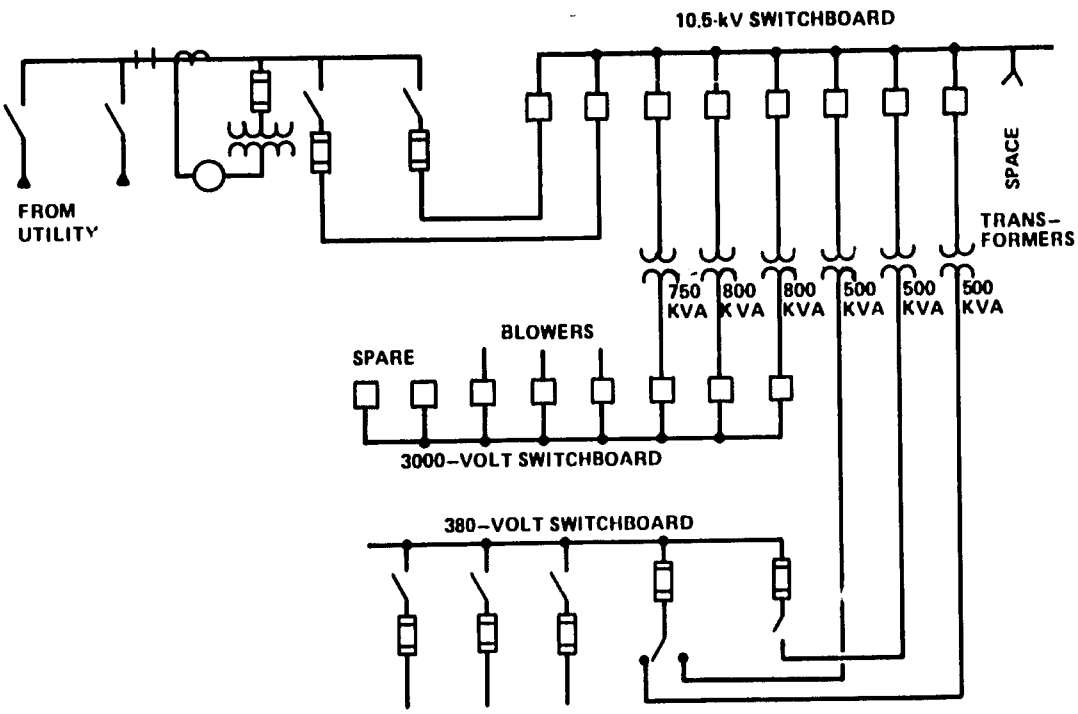

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FIGURE E-6

ONE-LINE DIAGRAM OF THE EAST TREATMENT PLANT ELECTRICAL SYSTEM

A/GOSD WWCG W/ECG

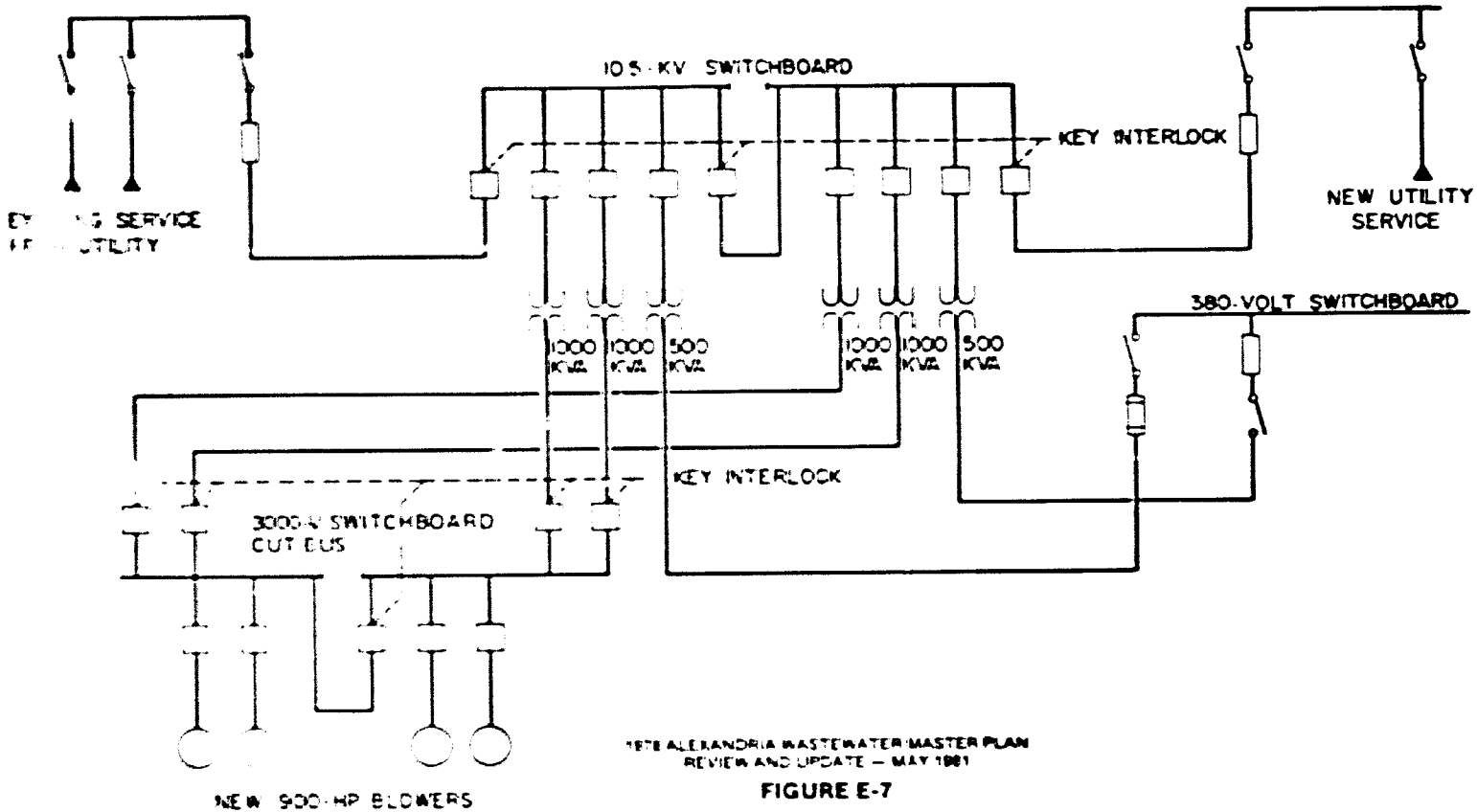

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FIGURE E-7

EAST TREATMENT PLANT
ONE-LINE DIAGRAM AT COMPLETION OF
BLOWER INSTALLATION

A/GOSD WWCG W/ECG

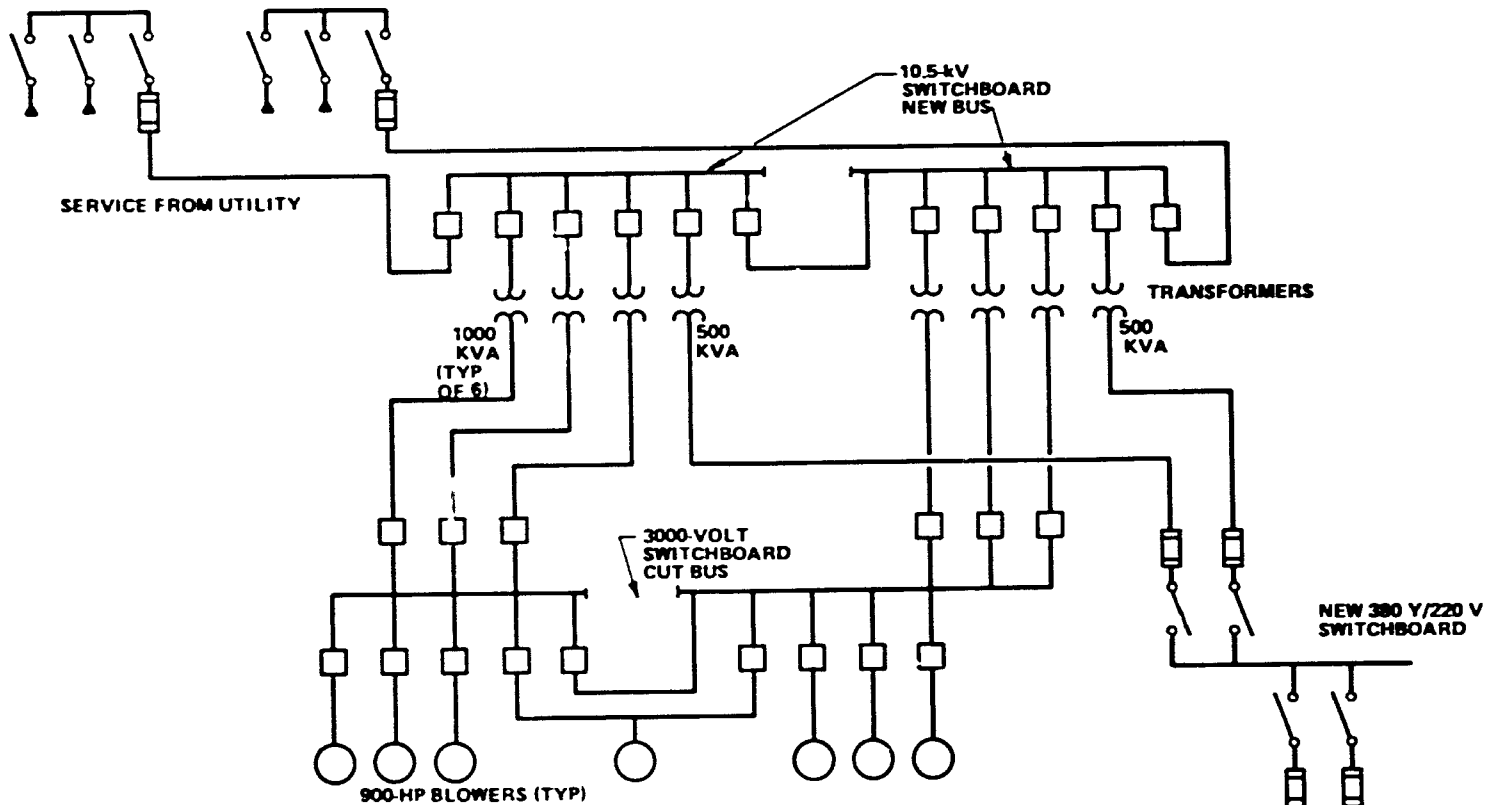

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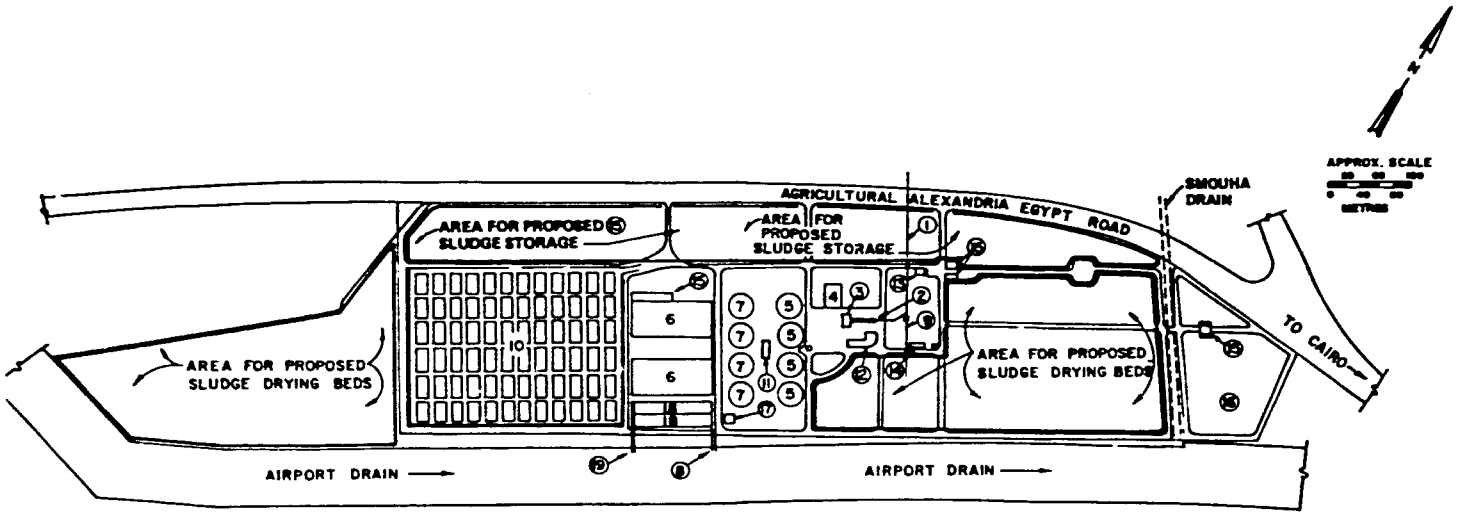
FIGURE E-8

PROPOSED ONE-LINE DIAGRAM FOR THE EAST TREATMENT PLANT

A/GOSD WWCG W/ECG



LEGEND:

1. INFLUENT FORCE MAIN
2. INLET WORKS
3. AERATED GRIT BASIN
4. PRE-AERATION BASINS
5. PRIMARY CLARIFIERS
6. AERATION BASINS
7. SECONDARY CLARIFIERS
8. EFFLUENT LINE, EXISTING
9. BYPASS TO DRAIN
10. SLUDGE DRYING BEDS
11. PUMP HOUSE
12. MACHINERY BUILDING
13. LABORATORY AND ADMINISTRATION BUILDING
14. INDUSTRIAL LAB
15. STORAGE BUILDING
16. HOUSING
17. CHLORINE BUILDING, PROPOSED
18. CHLORINE CONTACT BASIN, PROPOSED
19. EFFLUENT LINE, PROPOSED

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FIGURE E-9

EAST TREATMENT PLANT LAYOUT WITH RECOMMENDED ADDITIONS

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APPENDIX F

WEST TREATMENT PLANT EVALUATION

INTRODUCTION

The West plant, located on the north shore of Lake Maryout, is designed to provide primary treatment to an average wastewater flow of 85 Ml/day. This plant is presently under construction. It will provide primary treatment for a portion of Alexandria's wastewater if sea disposal is the selected management plan and pretreatment of industrial wastewater if land application is selected. If sea disposal is selected, the plant would be expanded to treat an average dry weather flow of 395 Ml/day. If land application is selected, the plant size would remain at 85 Ml/day and provide primary treatment until the land application system is implemented. At that time, the plant would become an industrial treatment facility with its effluent combined with the remainder of Alexandria's wastewater for irrigation. In this case, systems that would add chemicals to the wastewater would be provided to neutralize acidic wastes and aid in precipitation of solids. Such chemicals could include lime and ferric chloride.

The plant evaluation reported in this appendix only assumes the capability to provide primary treatment for the 85 Ml/d wastewater flow for which it was designed. Modifications to the plant to accommodate different wastewater flows and provide different treatment methods are discussed in Appendix J.

Description

When completed, the West plant will provide pretreatment by grit removal, screening or comminuting, and pre-aeration; and primary treatment by sedimentation. Figure F-1 is a layout of the existing West plant facilities and Figure F-2 is a schematic diagram. Incoming wastewater will be measured by a Venturi meter and then comminuted. Alternatively, it may be screened by manually cleaned bar screens. The flow will then be degritted in aerated grit chambers and pre-aerated for scum removal. The wastewater will then be treated by primary sedimentation prior to the planned interim discharge to Lake Maryout. Scum and grease removed in pre-aeration is to be trucked to a landfill. Primary scum and sludge will be air dried on sand drying beds, and the dried sludge will be made available to local farmers as a soil amendment.

Basis of Evaluation

The evaluation of the West plant and recommendations for improvements are based on the following influent wastewater characteristics:

Flow	85 Ml/day average flow 100 Ml/day peak flow
BOD	573 mg/l or 48,700 kg/day
Suspended Solids (SS)	590 mg/l or 50,200 kg/day

Desired pollutant removals through the plant are 60 percent for SS and 25 percent for BOD.

Cost Basis

Capital cost of improvements to the West plant are based on quantity estimates prepared from preliminary designs of the recommended improvements. General conditions concerning capital cost estimating, e.g., time basis for monetary values, foreign exchange rates, construction techniques, etc., are discussed in Appendix B.

Because the West Treatment Plant has not been constructed, records of actual operating and maintenance costs do not exist. Therefore, these costs are estimated by procedures discussed in Appendix B.

EVALUATION

The following evaluation of the plant's unit processes proceeds according to the wastewater flow through the plant. The discussion describes the components, assesses their capabilities, and suggests improvements.

Inlet Works

When the initial construction is completed, flow will enter the plant through force mains from four pump stations: Industries pump station, the Furn El-Gueraya pump station, the No. 1 West pump station and the No. 3 West pump station. Pumped flow will discharge to a riser well at the plant inlet. Overflow weirs and troughs around the riser well will collect any well overflow and direct it to Lake Maryout. Wastewater from the inlet well will flow through an inlet channel that includes a Venturi flume for flow measurement. Downstream from the flume, the

channel forms the grit chamber inlet/bypass channel. The inlet and flow measurement works appear adequate; however, grit buildup will occur in the influent channel, restricting capacity and rendering flow measurements inaccurate. To correct this situation, WWCG recommends that a Parshall flume be located upstream from the existing flume for flow measurement. The influent riser well walls will have to be raised and aeration of the channel downstream from the Parshall flume will be necessary to keep grit suspended in the flow.

Grit Removal

Six grit chambers are provided as indicated on the plant flow schematic, Figure F-2. With five chambers in service, the hydraulic detention time will be 2 minutes at peak flow. This is less than desired; however, in this case, no changes are recommended because the primary sludge will not be anaerobically digested or mechanically dewatered, so some grit in the sludge should not be a problem. Air requirements are estimated at 750 m^3/hr.

According to the current design, operators can use manual valves on either of two pumps serving each bank of three grit chambers to pump grit from each basin. No decision has yet been made as to the disposition of the pumped grit flow. WWCG recommends that the design be modified to pump the grit to two grit concentrators, with the overflow returned to the process flow and the underflow continuously pumped to two grit classifiers. Flow should be continuously pumped to the concentrators, with one pump serving each grit chamber. Figure F-3 is a schematic of the recommended system.

The grit pumps should be of the air-operated diaphragm type. Information describing the types of grit pumps that are to be installed at the West plant was not available at this writing. If they are similar to the grit pumps installed at the East plant, they are unsuited for pumping grit as they have closed impellers and thus have a high tendency to clog. Eight new air-operated diaphram grit pumps should be installed at the West plant, one for each basin and one for each concentrator.

Additional recommendations related to the grit removal operation include (1) replacement of the grit chamber inlets and outlet stop logs with fabricated metal handwheel-operated slide gates, and (2) addition of handrails.

Comminution and Screening

Wastewater discharge from the grit chambers will flow to a channel feeding six comminutors. Four manually cleaned bar screens will be

available when the comminutors are bypassed. Inspection of the comminutors disclosed that they appear to be light-duty units, and operating problems are likely. In the event of operational problems, the manually cleaned bar screens should provide adequate bypass capacity. Comminutor and bar screen inlets and outlets are provided with stop logs. These should be converted to handwheel-operated metal slide gates, and handrails should be provided around the sides and all openings in the deck area.

Effluent from the comminutors and bypass screens will discharge to an open channel which, in turn, will discharge at the south end through a gated bypass opening to the pre-aeration effluent conduit. The north end of the channel will discharge to the pre-aeration feed channel.

Pre-aeration

Wastewater from the pre-aeration feed channel will discharge to six pre-aeration chambers. Flow can also be diverted directly to primary treatment through a sluice gate and bypass line terminating in the primary clarifier inlet structure. If five of the six chambers are in operation, then the detention time at peak flow will be about 16 minutes and will be suitable for the intended purpose.

Pre-aeration air requirements are calculated at 2500 m^3/hr, although up to 3500 m^3/hr may be desirable. Air requirements for grit removal plus pre-aeration are estimated at 3250 m^3/hr to 4250 m^3/hr. New blowers rated at 4500 m^3/hr should be installed at the West plant, with one unit as standby. The 3000-volt blowers planned for installation were intended for use in an activated sludge plant as originally designed. Those blowers should not be installed because of the large expense of installing the 3000-volt and 10.5-kv switchgear that will otherwise not be required. This is discussed later in this appendix.

Scum and grease from the pre-aeration basins is to flow by gravity to a scum well where it will be pumped to a truck for hauling to disposal, i.e., a landfill. Effluent from the individual pre-aeration chambers will discharge through sluice gates to an effluent conduit which, in turn, will discharge to the east and west primary inlet structures.

Primary Sedimentation

Flow from pre-aeration will enter two primary sedimentation inlet structures, flow through a feed channel, and will be discharged into the sedimentation basins. Each of the eight basins will have a concrete scum collector trough and a single concrete effluent weir. With all

sedimentation basins in service at an average daily flow of 85 Ml, the surface overflow rate will be 26.6 m/day and the weir loading will be 1062 m^3/day/m. This weir loading is excessive; however, since the overflow rate is conservatively low, adding another effluent weir is not justified. Primary effluent is to discharge to Lake Maryout through two lined open channels.

Sludge and scum are to be moved by bridge-mounted collectors to double hoppers and a single trough, respectively, in each basin. The sludge hoppers are 1 m square by 0.6 m deep. Two rail-mounted bridge collectors are provided, one to serve each set of four basins. Sludge is to pass from each sludge hopper by hydrostatic head through a pipe to the sludge channel 1.5 m wide, sloped at 1 to 300 in the direction of the sludge drying beds. Scum from the scum trough is to flow to the main sludge channel (see Figure F-2). Once the main sludge channel leaves the basin structure it will narrow to 1 m wide, with a depth of 0.8 m and a slope of 1 to 300.

Evaluation of the sludge and scum removal and transfer systems indicates that the systems will not operate properly. Sludge and scum flows will certainly be more dilute than the 4 percent solids concentration required to minimize sand drying bed needs. However, if the 4-percent concentration can be achieved through careful operation of the scum and sludge removal and transfer systems, the sludge will not be transmitted to the sand drying beds.

Several modifications to the sludge and scum removal and transport system are recommended:

- Enlarge the individual sludge hoppers to 5 m by 2 m by 3.5 m deep. This will result in sludge discharging to the hoppers and not to the flat area adjacent to the hoppers where it would be lifted by gas generation.

- Provide a piston pump for each clarifier, with the pumps located in the sludge channel. The pump suctions would be valved so that sludge would be taken alternately from each hopper in any one basin. The pumps and suction valves could be controlled by time clocks at a preprogrammed rate throughout the day.

- Mount discharge piping from the sludge pumps in the sludge channel, which the channel will serve as a pipe chase.

- Construct scum wells at both the east and west ends of the scum trough. Scum would be pumped from the wells to trucks for hauling to a landfill for disposal.

Disinfection

The need for disinfection at the West plant will be determined by the wastewater management plan selected. It will be needed for the sea disposal option but not for the land application option. Further discussion on this subject can be found in Appendix J, Treatment Alternatives. Addition of disinfection facilities for the 85 Ml/day should be held in abeyance until a comprehensive plan for Alexandria's wastewater treatment and disposal has been selected.

Sludge Management

A total of 28,000 m^2 of sand drying bed surface is provided in the existing 112 beds. Because the West plant handles a large amount of industrial waste, the plant influent could contain 590 mg/l suspended solids. At a 60-percent solids removal through the clarifiers, the estimated daily dry solids sludge production is 30,100 kg. At 4 percent solids, this translates to 755 m^3 sludge per day. This sludge flow will require a sand drying bed area of 99,900 m^2, or an additional 71,900 m^3 (7.2 ha) of sand drying beds including support areas. Storage area to hold the sludge for 1 to 2 years onsite will require an additional 7 ha for a total additional sludge drying and holding area of about 14 ha. Storage is highly desirable to ensure adequate pathogen decay prior to public distribution of the sludge as a soil amendment. The additional drying bed and sludge storage facilities would be located east of the existing sand drying beds and south of the primary sedimentation tanks, administration and other laboratory building, and staff housing. To accommodate the drying bed-storage area, land south of the immediate plantsite would have to be filled. The currently constructed facilities include a dried sludge sales building. Sludge, after drying and storage for pathogen decay, can be moved into the building for final storage and distribution to farmers.

According to the current design, sand bed underdrainage is to be returned to the plant influent. The underdrain quality should be adequate, however, to allow for its discharge to the primary clarifier effluent.

Because of the extent of construction needed for tne additional drying bed storage area, a preliminary cost analysis was performed to compare that alternative with vacuum filter dewatering followed by composting. The result indicated drying bed storage could be as much as 19 percent less costly. The difference is mostly due to the larger energy and chemical requirements of the vacuum filter-composting operation. Land costs have not been included in the analysis and would be more

significant for the drying bed alternate. However, much of the required land may already be owned by AGOSD in the vicinity of the West plant, though some minor filling may be needed.

Electrical System

Construction of the electrical system at the West Treatment Plant has not been started. An electrical power distribution center has been located on the plantsite by the Alexandria Company for Electric Distribution (utility), but no underground cables, transformers, service entrance switchboards, or motor control panels have been installed. Construction of the facilities and installation of mechanical equipment has not progressed to the point where installation of electrical equipment and cables would be appropriate.

The electrical equipment for the project was delivered in 1964 and has been stored ever since. Since the equipment furnished was for use in a secondary treatment plant similar in design to the East Treatment Plant, we believe that the electrical system designed for the West plant was similar to the one installed at the East plant. The 10.5-kV switchboards and the 3000-volt switchboards are stored out-of-doors in their original shipping crates. Lighting fixtures, lamps, motor starters, electrical cables and miscellaneous control panels are stored in three different buildings at the East Treatment Plant. Several 380-volt switchboards and control panels have been stored at the West plant.

The switchboards stored out-of-doors in their original shipping crates appear to be well protected. Only one of the crates viewed had been opened. The rest of the equipment is stored in unfinished areas that are very dusty. Most of the equipment has been unpacked or at least opened, and consequently a large amount of dust has accumulated inside the equipment, in control cabinets, and on circuit breakers, relays, starters, etc.

The intended purpose of all the equipment viewed is not obvious, and much of it cannot be used in completing the construction because the treatment process has been changed. The original equipment is not properly rated or configured for the present needs.

WWCG recommends that the electrical equipment provided in 1964 not be installed and new equipment of modern design, tailored for the project, be purchased and installed. This recommendation is made because the existing equipment is already 16 years old and its projected useful life has been greatly reduced by aging of insulation and because dust accumulations in relays, motor starters, circuit breakers, etc., will cause wear. Also, the existing equipment will need to be modified before it can be used at the plant. Completion of the system as originally

designed includes installation of high-voltage equipment and purchase and installation of high-voltage transformers that are no longer required.

The original equipment was designed to supply a plant having three 100-hp, 3000-volt blowers and several hundred horsepower in pumping capability. The total process load at the plant as it is being completed will require less than 250 hp and no 3000-volt or 10.5-kV power.

The proposed system eliminates the need for the existing 10.5-kv and 3000-volt switchboards and eliminates the need to purchase two transformers with 3000 volt secondaries. It will receive power from the Utility at 10.5 kv, three-phase and will supply all power requirements of the plant at 380 wye/220 volts.

See Figure F-4 for a one-line diagram of a recommended power supply system to serve plant needs. Standby power will be provided at the plant by an engine-generator set. Redundant equipment will be divided between two switchboards and it will be possible to manually select either normal power or standby power as the source for each switchboard. The proposed system will provide maximum reliability with a minimum of equipment.

Instrumentation and Control Systems

Construction of the instrumentation and control systems at the West Treatment Plant has not been started. Construction of the facilities and installation of mechanical and electrical equipment has not progressed to the point where installation of delicate instruments, primary elements, control panels, and interconnecting cables would be appropriate.

The instrumentation and control system equipment also has been stored at the East Treatment Plant since 1964. When the equipment was viewed it was found that many of the items have been unpacked and some were broken. The areas where the equipment is stored are very dusty, and consequently there is dust on the relays and in the instruments.

WWCG recommends that existing instrumentation and control system equipment not be installed and new equipment of modern design, tailored for the project as it will be completed, be purchased and installed. This recommendation is made for the following reasons:

- The existing equipment was provided for a secondary treatment plant and much of it is not suitable for use in the primary

plant as it will be completed. Control panels will need to be modified with many instruments left out.

- The existing equipment is 16 years old and is already obsolete. Repair and replacement parts probably are not readily available.

- The existing equipment has been stored in areas that are not environmentally suitable. Dust and moisture over the years have probably caused the delicate instruments to be inoperative and major maintenance could be required to put them into operating condition.

- Modern equipment has more capability than the existing equipment and is more reliable. Also, repair and replacement parts would be more readily available.

- A new system could be tailored to meet the needs of the processes of the plant under construction.

PLANT CAPABILITY

If the suggested modifications to the West plant are made, it should be capable of accommodating the following wastewater characteristics:

o	Average dry weather flow	85 Ml/day
o	Peak flow	100 Ml/day
o	BOD average	48,700 kg/day
o	Suspended solids average	50,200 kg/day

Under these conditions, the plant should be able to remove 60 percent of the suspended solids and 25 percent of the BOD. Therefore, effluent quality should be 430 mg/l and 236 mg/l BOD and suspended solids, respectively. Table F-1 shows the design criteria for the West plant's unit processes with the recommended improvements.

RECOMMENDED IMPROVEMENTS

Based on the previous analysis of the West plant, the following improvements are recommended:

- Inlet works

  - Parshall flume (1)
  - Handrails and grating
  - Aeration downstream from flume

- Grit Chambers

  - Air-operated diaphragm pumps (8)
  - Grit concentrators (2)
  - Grit classifiers (2)
  - Handwheel-operated gates on basin inlets, outlets, and bypass (13)
  - Handrails around grit basins
  - Storage containers (2)

- Pre-aeration basins

  - Handrails
  - Blowers (2)

- Comminuting and screening

  - Handwheel-operated gates (10)
  - Handrails around top of comminutor-screening facility

- Primary clarifiers

  - Enlarged sludge hoppers
  - Piston pumps (8)
  - New glass-lined sludge piping
  - New scum wells (2)
  - Piston pump at each scum well (2)

- Sludge management

  - Drying beds (additional 7.2 ha including support areas)
  - Storage area (7 ha)

- Electrical system
  - Switchboards
  - Transformers
  - Motor starters
  - Control panels
  - Standby engine generator (1)
- Instrumentation and control system - complete

COST ESTIMATES

Table F-2 lists the capital costs for the recommended improvements to the West plant. The costs are in terms of end-of-1980 Egyptian pounds (LE) and are of budget level accuracy. They are based on quantity estimates of materials and equipment required for the described improvements.

Table F-3 shows the estimated operation and maintenance costs for the West Treatment Plant with the recommended improvements.

Table F-1
PROPOSED DESIGN CRITERIA FOR THE
WEST TREATMENT PLANT

Item	Design Criteria
General Criteria	
Treatment capacity	85 Ml/day average, 100 Ml/day peak
Standby power	Engine generator
Power supply	380-volt, 3-phase, 50 HZ
Inlet Works	
Flow measurements	Parshall flume
Grit Removal	
Process	Aerated grit chambers, existing design
Removal equipment	Pump suction, existing design
Chamber depth	3 m
Length: width ratio	2.34:1
Detention time	2 minutes
Air supply	6 l/s per metre of tank length
Grit cleaning and dewatering	Concentrator followed by classifier
Sewage Grinding	
Process	Comminutors
Number	6
Bypass Screening	
Process	Manually cleaned
Clear space	2 cm or less
Velocity, max.	0.55 m/s
Depth, max.	1.0 m
Width of screen	0.78 m
Screenings discharge	Wheeled container
Drainage	Returned to plant flow
Disposal	Landfill
Oil and Grease Flotation	
Process	Aeration, existing design
Detention at peak flow	16 minutes with 1 of 6 basins out of service.
Chamber depth	2.95-m water depth
Chamber width	5.24 m
Chamber length	20 m per basin, 6 basins
Air supply	1.00 m^3/m^3 wastewater
Skimmings discharge	To tank truck
Skimmings disposal	To landfill

Table F-1 (continued)
PROPOSED DESIGN CRITERIA FOR THE
WEST TREATMENT PLANT

Item	Design Criteria
Primary clarification	
Process	Rectangular, existing design
Length: width ratio	4:1
Length	40 m
Width	10 m
Surface loading, average	26.6 m/day
Side water depth	3.68 m
Detention time, average	3.32 hr with all 8 units in service, 2.91 hr, with one unit down
Weir loading, average	1062 m^3/day/m at weir length
Scum removal	Skimmer with discharge to trough
Scum discharge	To well and then pump to tank truck with piston pumps
Scum disposal	Truck to landfill
Sludge removal	Travelling collector
Sludge pumping	Piston pumps
Effluent disinfection	Depends on wastewater management plan selected; no recommendation to be made until total plan is selected
Sludge dewatering	
Process	Sand beds
Loading	3.17 m/yr
Underdrainage	Return to plant
Dewatered sludge storage	1 to 2 years
Sludge disposal	Use on farmland

Table F-2
CAPITAL COST ESTIMATE OF RECOMMENDED
WEST TREATMENT PLANT IMPROVEMENTS
(end of 1980)

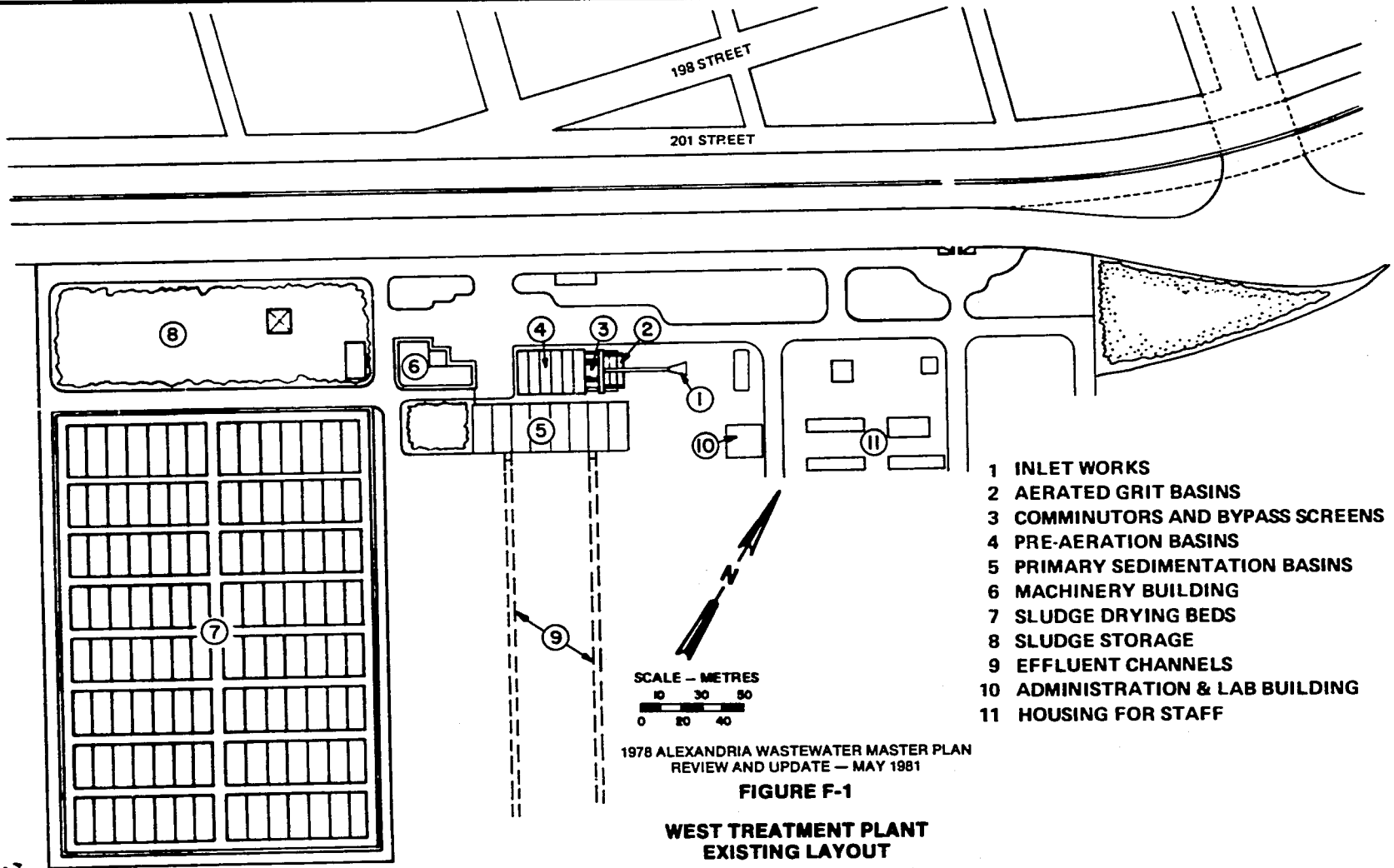
Item	Estimated Cost (LE)[a]
Inlet works	25,000
Grit removal	174,000
Pre-aeration	106,000
Comminuting and screening	26,000
Primary clarifiers	266,000
Sludge pumping	54,000
Sludge management	9,186,000
Electrical and instrumentation and control system	117,000
Subtotal	9,954,000
Contingency @ 25%	2,489,000
Subtotal	12,443,000
Engineering, legal, and administrative costs @ 15%	1,867,000
Total	14,310,000

[a]Excluding customs charges

Table F-3
WEST TREATMENT PLANT
ESTIMATED ANNUAL O&M COST SUMMARY
(end of 1980)

Item	1980[a] Quantity	1980[a] Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	8000		8000		8000	
Cost		53		53		53
Power						
1,000,000 kWh	0.389		0.389		0.389	
Cost		4		4		4
Fuel						
1000 litres	11		11		11	
Cost		1		1		1
Chemicals						
Tonnes						
Cost	--	--	--	--	--	--
Materials						
Cost		482		482		482
Total						
Cost	--	540	--	540	--	540

[a]Annual O&M costs indicated for 1980 used as basis for estimating subsequent O&M cost (and costs in following years to 1990) by interpolation between 1980 and 1990 estimated annual costs.



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE F-1

WEST TREATMENT PLANT
EXISTING LAYOUT

A/GOSD WWCG W/ECG

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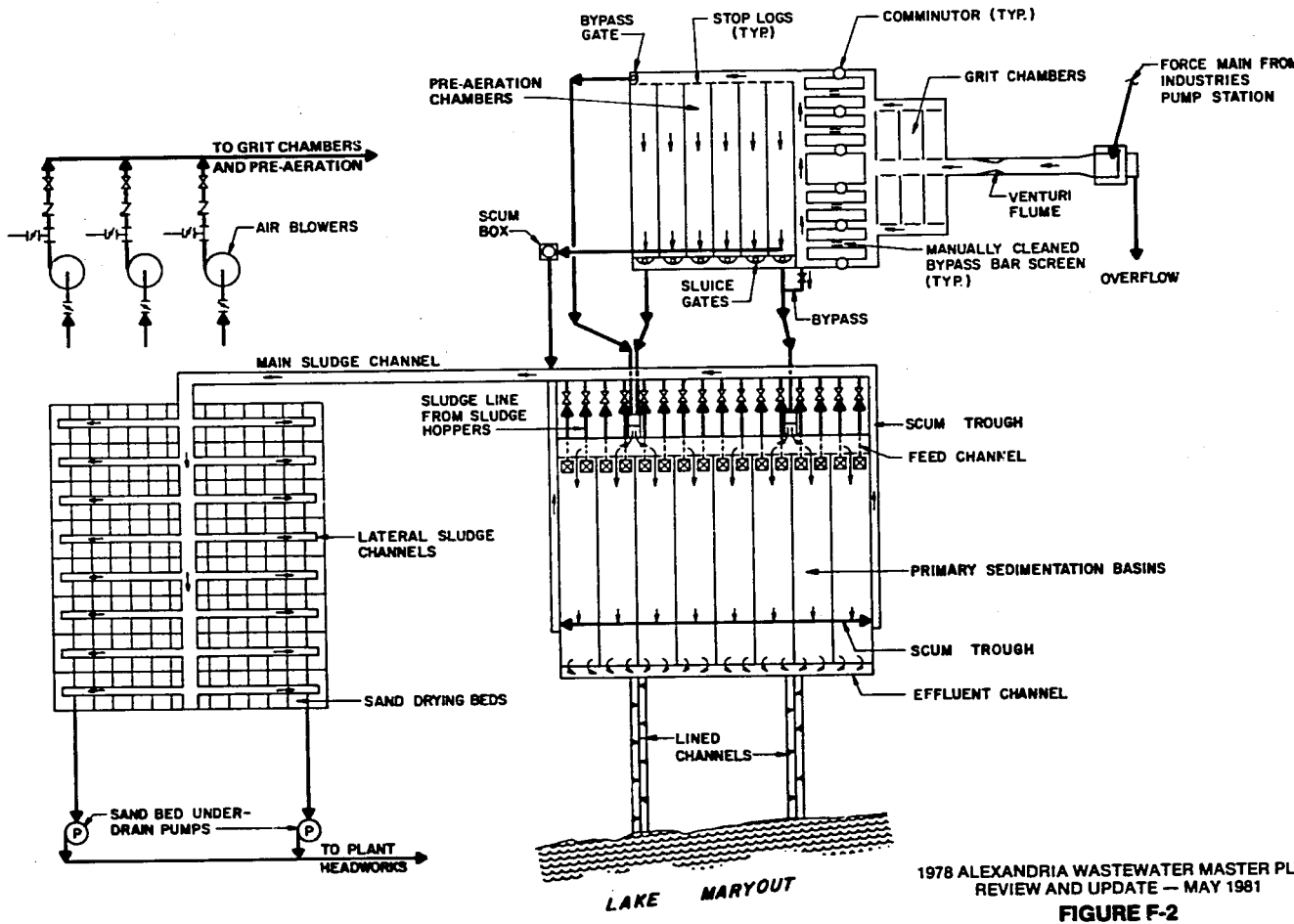

1978 ALEXANDRIA WASTEWATER MASTER PLAN
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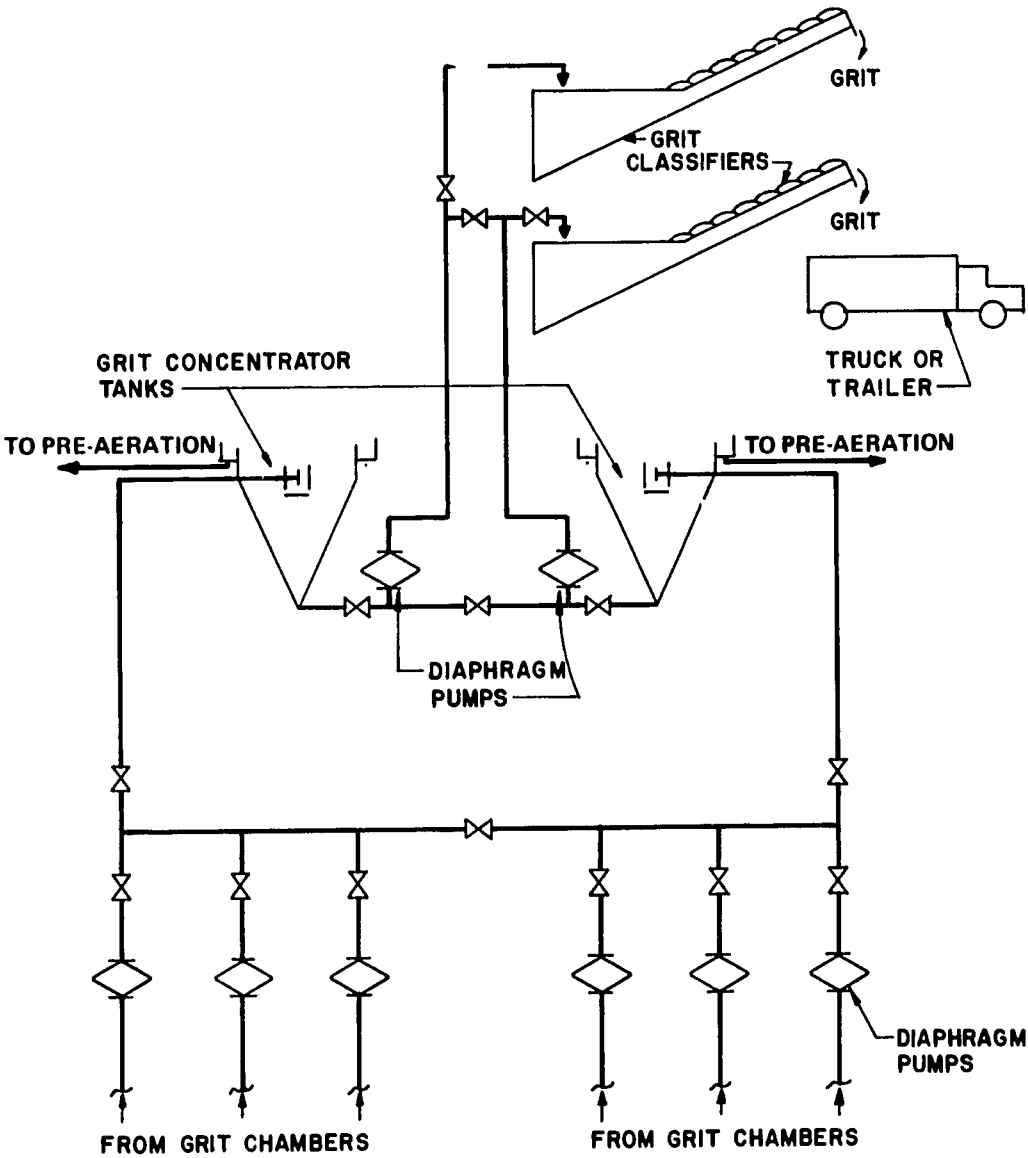
FIGURE F-2

WEST TREATMENT PLANT PROCESS FLOW SCHEMATIC

A/GOSD WWCG W/ECG

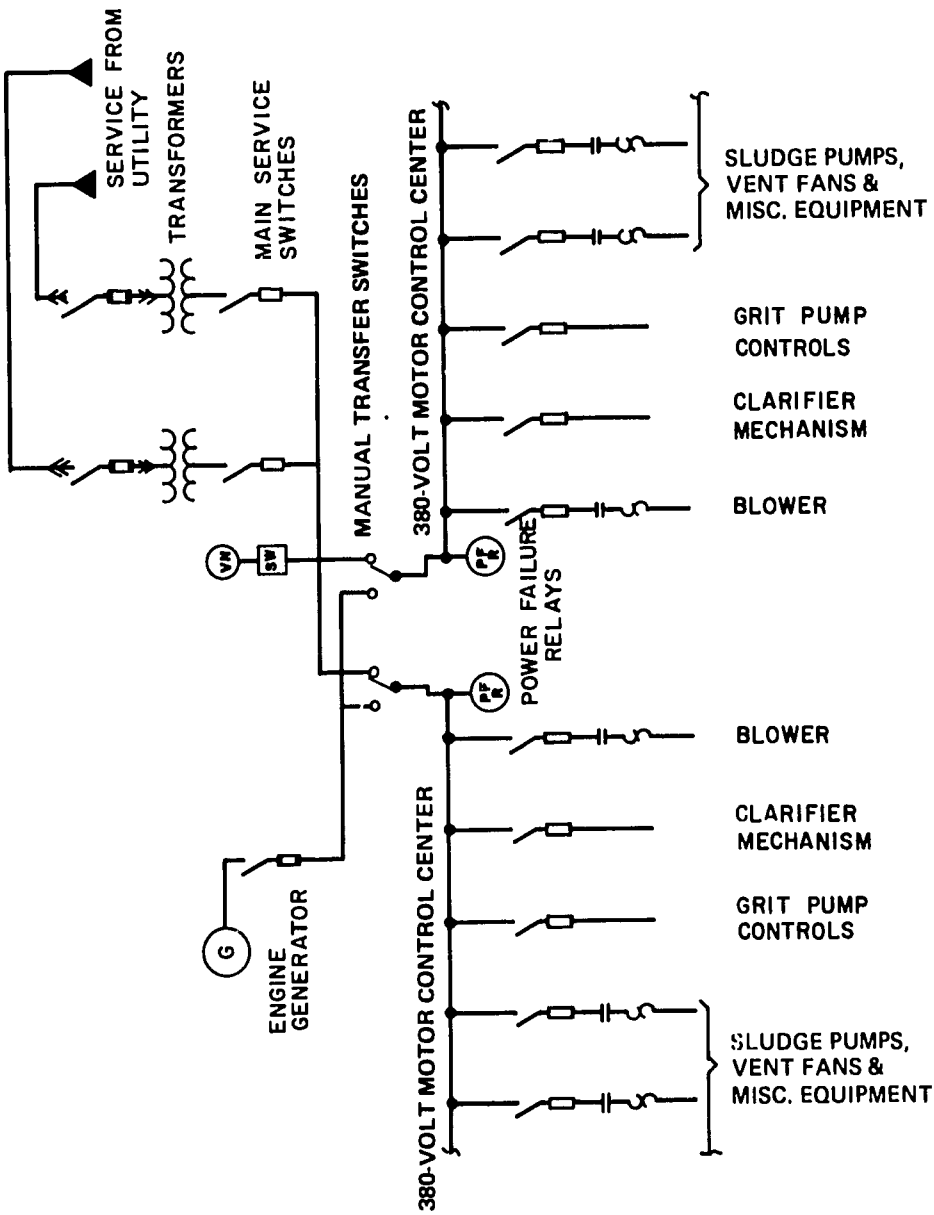


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE F-3

WEST TREATMENT PLANT
PROPOSED GRIT HANDLING SYSTEM

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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FIGURE F-4

WEST TREATMENT PLANT
PROPOSED ELECTRICAL SYSTEM

A/GOSD WWCG W/ECG

APPENDIX G

PUMP STATION EVALUATION

INTRODUCTION

This appendix evaluates the present condition of each of the existing pump stations and force mains. Deficiencies are identified, and suggestions are made for corrective action to facilitate safe operation and accommodate the present flow requirements of the system until a long-term plan is implemented. This appendix also recommends the corrections of deficiencies that are not impacted by selection of a long-term plan.

To accomplish this evaluation, the 1978 Master Plan and related documents were reviewed and 15 of the existing 32 stations were inspected. The following sections are a compilation of information gathered by personal inspection and review of the 1978 Master Plan and related documents. Future requirements for each pump station will be determined at a later date based on the findings of this Master Plan Review and subsequent basis of design reports. During the interim period, prior to implementation of long-term system improvements, most existing pump stations must continue to function.

REVIEW OF PREVIOUS REPORTS

Review of the 1978 Master Plan, Special Reports No. 2 and No. 4, and the design reports for the recommended improvements indicates that an extensive review of the existing pump stations has already been accomplished. Our conclusions at the completion of the inspection of 15 pump stations follow very closely the recommendations of the previous reports. These reports should be used as a guide by A/GOSD personnel in upgrading existing facilities. They can be used as a checklist to restore the pump stations to reliable operating and safe working conditions. Table G-1 is a summary of the recommendations from the previous reports concerning each of the pump stations.

Many improvements have already been made since the reports were written. These improvements are noted herein. Some of the work suggested in the reports may be excessive and these areas of concern are also noted along with comments from the present operating and supervisory personnel.

The most pressing need at the present time is not further investigation, but rather, corrective action. Repair, reconditioning, and in some

cases, rebuilding are urgently required. Many of the extremely costly sewer cleaning requirements within the Alexandria system are directly attributable to improper and inefficient pump station operation. Failure to properly operate the stations causes gravity sewers to surcharge. Surcharge in turn causes poor flow velocity resulting in frequent sewer blockages, reduced capacity, and the need for more cleaning.

PUMP STATION AND FORCE MAIN INVENTORY

The following inventory and review is based mostly on information contained in the aforementioned reports and supplemented by inspections of nearly half of the facilities. Figure G-1 shows the location of the major facilities.

Central Zone

Four pump stations are located in the central zone: Sporting, Bisiry, Sinin Pasha, and Kait Bey.

The Sporting station discharges either stormwater or combined stormwater and wastewater to an overflow to the sea, or wastewater or combined flow to a force main to the gravity interceptor system paralleling the Eastern Harbor shoreline. The station has five pumps: two at 720 m^3/hr and 35 m of head, two at 900 m^3/hr and 25 m of head, and one at 600 m^3/hr and 20 m of head. The force main is 410 mm in diameter and 760 m long.

The basic purpose of t[illegible] station is to prevent local flooding from the surcharged [illegible]tor. The station pumps wastewater to the collector ge[illegible] through a force main 210 m long and 510 mm in diameter [illegible]ed capacity is 600 m^3/hr at 15 m total head.

The Sinin Pa[illegible] local flooding in the Mohamed Ali Square are [illegible] local collectors is pumped to the collector gener[illegible] a force main 510 mm in diameter and 70 m long. T[illegible]y of the station is 600 m^3/hr at a total head of [illegible]

The Kait Bey [illegible] is a major facility that pumps into the existing [illegible] a rated capacity of 3000 m^3/hr at a total head of 5 m. [illegible] force main; rather, the sewage is lifted to a receiving [illegible]mber to provide sufficient head to discharge by gravity through the [illegible]tfall.

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Western Zone

Western zone pump stations include:

- No. 1 West
- No. 2 West
- No. 3 West
- New Furn El-Gueraya
- Old Furn El-Gueraya
- Industries
- Gheit El-Enab Main
- Gheit El- Enab Auxiliary
- Mohsein Pasha

The No. 1 West station is not presently operating because the West Treatment Plant is not yet completed. When it becomes operational, the No. 1 West pump station will lift sewage from the Gabbary collector to the plant headworks. The station has a rated capacity of 1440 m^3/hr at a total head of 13 m. It discharges through a force main 1300 m long and 610 mm in diameter.

At present, the No. 2 West pump station pumps directly to the Western Harbor. When the West Treatment Plant is completed, however, it will pump directly to the No. 3 West pump station, which in turn, will pump to the plant. The rated capacity of the station is 576 m^3/hr or 436 m^3/hr at 15 m or 8.7 m of total head, respectively, depending on high-speed or low-speed single-pump operation. The proposed force main to the No. 3 West pump station is 1150 m long with a 356-mm diameter.

When the West Treatment Plant is completed, the No. 3 West pump station will discharge to its headworks. At that time this station will also receive wastewater from the No. 2 West pump station. Until completion of the West Treatment Plant, the No. 3 West pump station will continue to pump to a nearby agricultural drain that flows to Lake Maryout. The station's rated capacity is 1440 m^3/hr at a total head of 14 m. The force main is 780 m long and 559 mm in diameter.

The new Furn El-Gueraya pump station is intended to replace the old Furn El-Gueraya station. It will pump both wastewater and combined

wastewater and stormwater. The wet well at the station is compartmentalized, one for dry weather flow and a second for wet weather flow. The dry weather flow will be pumped to the West plant when it is completed, while wet weather flow will be discharged to the Gabbary collector for final discharge to Lake Maryout. The rated capacity of this station is 720 m^3/hr at a total head of 10 m. Special Report No. 2 for the Alexandria Wastewater Facilities Development Program indicates that the new Furn El-Gueraya station will not be able to pump wastewater to the new waste treatment plant when it is complete because of improper pump selection. According to this report, the estimated head required to pump to the West plant at rated discharge is about three times the rated pump head and exceeds the pump shutoff head. At the present time, flow from the Gheit El-Enab collector, which is tributary to the new pump station, is diverted to the old Furn El-Gueraya station because the West plant is not completed.

The old Furn El-Gueraya pump station receives wastewater from a collector serving the Furn El-Gueraya tributary area and a portion of Gheit El-Enab tributary area. All sewage thus collected is pumped northward underneath the Mahmoudiya Canal to the Mahmoudiya east collector where it flows by gravity toward the Mohsein Pasha pump station. However, most of the sewage returns to the south side of the canal through the Mohsein Pasha bypass siphon located just south of the Mohsein Pasha pump station. The sewage then flows by gravity past Gheit El-Enab main pump station into an open drain that discharges into Lake Maryout. The station's rated capacity is unknown, but it contains two 300-m^3/hr pumps and one 600-m^3/hr pump. Its force main is 204 mm in diameter and about 150 m long. This pump station will be abandoned when the new Furn El-Gueraya station is placed in operation.

The Industries pump station pumps mostly untreated industrial wastewater to Lake Maryout, at present through a temporary force main that is 700 m long and 610 mm in diameter. When it is completed, it will pump wastewater to the West Treatment Plant through a force main 3500 m long; approximately 1600 m of the force main is 610 mm in diameter, and 1900 m of it is 914 mm in diameter. The pump station's rated capacity is 2160 m^3/hr and 1440 m^3/hr at high and low speeds, respectively, and the heads are, respectively, 23 m and 10 m.

The Gheit El-Enab main pump station receives sewage from the eastern part of Gheit El-Enab and lifts it to an open pit adjacent to the station structure. The sewage then flows by gravity in an open drain leading to Lake Maryout. When the lake is high, it overflows into a sewer near the Industries pump station, which returns it to the station's wet well. Two horizontal pumps then pump this reverse flow into the open drain adjacent to the station. The pump station's rated capacity is unknown, but the six pumps vary in capacity from 300 m^3/hr at 4 m of total head to 900 m^3/hr at 15 m of head.

The Gheit El-Enab auxiliary pump station has two pumps, each with a rated capacity of 600 m^3/hr at about 10 m of total head and one pump (900 m^3/hr) at about 25 m of total head. This station serves the south-central portion of Gheit El-Enab and is intended to pump wastewater through 1200 m of force main 254 mm in diameter to the Gheit El-Enab main pump station. Due to problems with the force main, this station has apparently never worked as designed. Instead, sewage is pumped to the Furn El-Gueraya collector, which currently conveys it to the old Furn El-Gueraya pump station.

The Mohsein Pasha pump station receives wastewater from a sewer in Mohsein Pasha Street serving the Mahmoudiya east and west collectors and local lateral sewers. Sewage is then pumped through either of two force mains, each 610 mm in diameter, for about 370 m to the Salah El-Din collector, from which it is eventually conveyed by gravity to the Kait Bey pump station for disposal in the sea. Flows in excess of the station's capacity flow south through a siphon beneath the Mahmoudiya Canal to an open channel that discharges into Lake Maryout. The pump station has four pumps: two at 1200 m^3/hr at 15 m of head and two at 480 m^3/hr at 11 m of head.

Eastern Zone

The eastern zone has 19 pump stations: Numbers 1 East through 11 East, Montazah, Mamoura, Hospitals, Sidi Bishr, Sarwat, Glym, Sidi Gaber, and Smouha.

The No. 1 East pump station receives sewage from its own tributary area and the Montazah, Mamoura, and Hospitals pump stations. Sewage is pumped directly to the No. 5 East pump station through about 2000 m of force main 406 mm in diameter before entering a force main 559 mm in diameter, which also conveys sewage from the No. 3 East pump station. In addition, No. 1 East pumps stormwater through about 70 m of force main 356 mm in diameter to a force main 508 mm in diameter that discharges to the Mediterranean Sea at Mandara Beach. The latter also conveys stormwater from the No. 2 East pump station. Sewage from Mamoura can also be diverted directly to the No. 2 East pump station in order to reduce the peak demand on No. 1 East during the summer. The station's rated capacity is 648 m^3/hr or 428 m^3/hr at 34 and 15 m of total head, respectively, with high-or low-speed, single-pump operation. The stormwater pumps have rated capacities of 516 or 436 m^3/hr at 15 and 8.7 m of head, respectively, at high or low speeds, respectively.

The No. 2 East pump station receives sewage from its own tributary area and pumps it directly to the No. 6 East pump station. Sewage is pumped through about 1250 m of force main 356 mm in diameter before entering a

force main 508 mm in diameter, which can also convey sewage from the No. 4 East pump station. In addition, No. 2 East pumps stormwater from its own tributary area and through about 1250 m of force main 406 mm in diameter before entering a force main 508 mm in diameter, which also conveys stormwater from the No. 1 East pump station. Stormwater from both stations is discharged directly to the Mediterranean Sea at Mandara Beach. The station's rated capacity is similar to No. 1 East.

The No. 3 East pump station receives sewage from its tributary area and pumps it directly to the No. 5 East pump station. Sewage is pumped through about 100 m of force main 356 mm in diameter before entering a force main 559 mm in diameter, which also conveys sewage from the No. 1 East pump station. The No. 3 East pump station also pumps stormwater from its own tributary area and the Nos. 5 and 4 East pump stations. This stormwater is pumped through a force main 559 mm in diameter directly to the Mediterranean Sea at Bir Masoud. The rated capacity of the No. 3 East pump station is 540 m^3/hr at 17 m of head for the sewage pumps and 1440 m^3/hr at 10 m of head for the stormwater pumps.

The No. 4 East pump station pumps sewage from its own tributary area and directly to the No. 6 East pump station. Sewage is pumped through about 50 m of force main 356 mm in diameter before entering a force main 508 mm in diameter, which also conveys sewage from the No. 2 East pump station. The No. 4 East pump station also pumps stormwater from its own tributary area directly to No. 3 East pump station. Stormwater is pumped through about 600 m of force main 356 mm in diameter before entering a force main 406 mm in diameter, which also conveys stormwater from No. 5 East. The pump station's rated capacity is 540 or 410 m^3/hr at a total head of 17 and 9.8 m and high- or low-speed, single-pump operation, respectively.

The No. 5 East pump station receives sewage from its own tributary area and from the Nos. 1 and 3 East pump stations. Sewage is pumped through a force main 559 mm in diameter and 2950 m long to the Abu Soliman siphon, which passes under the Montazah Canal and discharges into the Abu Soliman collector. The sewage then flows by gravity to the No. 11 East pump station. The No. 5 East pump station also pumps stormwater from its own tributary area through a force main 305 and 406 mm in diameter, and 970 m to the stormwater wet well at the No. 3 East pump station. The station's capacity for sewage is 1440 or 1102 m^3/hr at 25 or 15.5 m of head, with high- or low-speed, single-pump operation. For pumping stormwater, the capacity is 360 or 248 m^3/hr at heads of 20 and 9 m, respectively, at high- or low-speed, single-pump operation.

The No. 6 East pump station receives sewage from the Nos. 2 and 4 East pump stations and its own tributary area. Sewage is pumped through a force main 509 mm in diameter through the Abu Soliman siphon to the Abu

Soliman collector. The sewage then flows by gravity to the No. 11 East pump station. The No. 6 East pump station also receives stormwater from its tributary area and pumps it through a force main 405 mm in diameter directly to the Mediterranean Sea at Sidi Bishr. The pump station's capacity for pumping sewage is the same as for the No. 5 East pump station. For pumping stormwater, the rated capacity is 540 or 410 m^3/hr at 20 or 9 m of head and at high- or low-speed, single-pump operation, respectively.

The No. 7 East pump station receives sewage from its own tributary area and pumps it directly to the Abu Soliman collector. Sewage is pumped through about 300 m of force main 356 mm in diameter before entering one 457 mm in diameter, which also conveys sewage from the No. 8 East pump station. Once in the Abu Soliman collector, sewage flows by gravity to the No. 11 East pump station. The No. 7 East pump station does not have separate stormwater pumping capability. Its rated capacity is the same as for the No. 4 East pump station.

The No. 8 East pump station receives sewage from its own tributary area and pumps it directly to the Abu Soliman collector. Sewage is pumped through about 530 m of force main 406 mm in diameter before entering a force main 451 mm in diameter, which also conveys sewage from the No. 7 East pump station. Once in the Abu Soliman collector, sewage flows by gravity to the No. 11 East pump station. The No. 9 East pump station does not have separate stormwater pumping capability. Its rated capacity is the same as for No. 4 East.

The No. 9 East pump station is intended to pump sewage from its own tributary area to the No. 10 East pump station, but it has never been placed into operation. Flow tributary to this station is diverted through the Hagar El-Nawatiya collector to the Smouha Drain. According to Special Report No. 2 for the Alexandria Wastewater Facilities Development Program, the difference in elevation between the influent sewer invert at the No. 9 East pump station and the influent sewer invert at the No. 10 East pump station would allow construction of a gravity sewer with capacity equal to the rated pump discharge. The need for the No. 9 East pump station is therefore not apparent. Its rated capacity is 396 and 263 m^3/hr at 53 and 10 m of head, at high- or low-speed, single-pump operation, respectively.

The No. 10 East pump station receives sewage from its own tributary area and pumps it through a force main 508 mm in diameter directly to the wet well of the No. 11 East pump station. It is also intended to receive sewage from the No. 9 East pump station, which has not been placed into operation. The No. 10 East pump station was also intended to pump directly to the Hagar El-Nawatiya collector near the No. 9 East pump station; however, the stormwater force main has not been completed and

the stormwater pump is not used. Therefore, any stormwater received at this pump station is eventually discharged to the No. 11 East pump station. The rated capacity of the station for pumping sewage is 1296 and 871 m^3/hr at 10 and 4.5 m of head at high- or low-speed, single-pump operation, respectively. For stormwater pumping, the rated capacity is the same as for the No. 6 East pump station.

The No. 11 East pump station receives sewage directly from the No. 10 East pump station and the Abu Soliman collector. The Abu Soliman collector receives sewage from its own tributary area and the remainder of the eastern zone through the Nos. 5, 6, 7 and 8 East pump stations. Discharge from the Montazah, Mamoura, and Hospitals pump stations also eventually reaches the No. 11 East pump station. Sewage thus collected is pumped through about 4000 m of force main 1200 mm in diameter directly to the headworks of the East Treatment Plant (when operating) or to the Smouha Drain when the plant is bypassed. The rated station capacity at high-speed, single-pump operation is 4320 m^3/hr at 28 m of head, and at low-speed, single-pump operation the capacity is 2898 m^3/hr at a head of 11.8 m.

The Montazah pump station receives sewage from its own tributary area and pumps it about 450 m through a force main 294 m in diameter to a sewer located in Horreya Avenue. Sewage then flows by gravity to the No. 1 East pump station. The Montazah pump station also pumps stormwater from its own tributary area through about 550 m of force main 294 mm in diameter directly to the Mediterranean Sea near the western wall surrounding the Montazah Palace grounds. For pumping sewage or stormwater, the rated station capacity at high-speed, single-pump operation is 288 m^3/hr at 25 m of head. At low-speed, single-pump operation, the capacity is 191 m^3/hr at 11 m of head.

The Mamoura pump station pumps sewage from its own tributary area through about 3500 m of force main 356 mm in diameter directly to the wet well of the No. 1 East pump station. Originally, output from this pump station was not included in the design criteria for the No. 1 East pump station and, because of the influx of summer residents, its discharge contributes to that station's capacity deficiency. A second force main leading to the No. 2 East pump station provides the flexibility to pump to either station. Rated capacity is 432 m^3/hr at a total head of 35 m.

The Hospitals pump station receives sewage from several hospitals and conveys it through a force main 152 mm in diameter to the No. 1 East pump station. Its rated capacity is 288 m^3/hr at a total head of 15 m.

The Sidi Bishr pump station serves a small tributary area along the Mediterranean Sea and pumps sewage through a force main 254 mm in diameter to a gravity collector about 350 m west along the Corniche.

This station can also pump directly to the sea by using a second force main 254 mm in diameter, which discharges about 220 m west of the station location. The rated capacity is 300 m^3/hr at about 8 m of head.

The Sarwat pump station receives sewage from its tributary area, which includes the Sidi Bishr pump station and lifts it to an adjacent manhole located in the Corniche. The sewage then flows by gravity to the Glym pump station. The station's rated capacity is similar to that of the Sidi Bishr pump station.

The Glym pump station receives sewage from its own tributary area and the Sarwat pump station (including Sidi Bishr), and pumps it through either of two parallel force mains 457 mm in diameter to a gravity collector about 170 m west along the Corniche. The sewage then flows by gravity to Horreya Street past the old Sidi Gaber pump station and eventually into the Smouha Drain. Ultimately, this sewage is pumped into Lake Maryout. The station has four pumps: one rated at 480 m^3/hr at 15 m of head, two rated at 480 m^3/hr at 10 m of head, and one rated at 540 m^3/hr at 25 m of head.

The new Sidi Gaber pump station will have two pumps that will be able to pump up to 121,000 m^3/day at 18 m of total head. This pump station will be able to pump flows through either of two force mains 610 m in diameter directly to the Mediterranean Sea. However, according to Special Report No. 2 (April 1977), this pump station is unnecessary. The flows that this station would be pumping could be accommodated by the Cromer collector, which has a capacity of 260,000 m^3/day, respectively. Therefore, the Cromer collector can accept the full capacity of its two principal contributors without becoming surcharged.

The Smouha pump station receives sewage and industrial waste from its tributary area and pumps it through about 1050 m of force mains 660 mm in diameter to the Horreya-New collector. Sewage then flows by gravity to the existing Kait Bey pump station for disposal in the sea. Flow in excess of station capacity continues by gravity to the Mohamed Ali collector, which conveys it to the Smouha Drain. This sewage is ultimately discharged into Lake Maryout. The pump station has two pumps at 900 m^3/hr and two at 1800 m^3/hr rated at a total head of 26 m.

PUMP STATION AND FORCE MAIN EVALUATION

Overall Conditions

All of the pump stations in Alexandria could not be inspected in the time allotted. The pump stations that are most likely to remain in service are the ones that were chosen for inspection. Information from

the reports previously mentioned supplemented the inspections. Pump stations inspected were: Nos. 1 through 7 East, and 11 East, Montazah, Mamoura, No. 2 West, Glym, Sporting, and Kait Bey.

One general finding is that all steel and asbestos cement force mains should be inspected and replaced if deficient. If force mains for pump stations that will remain in extended service are replaced, they should be constructed of ductile iron. Design criteria should be in accordance with Appendix A. For those pump stations which are to be abandoned, less costly materials with shorter service lives (perhaps salvaged pipe) might be used if conditions warrant.

WWCG's onsite investigations supplemented by the previous work addressed in Special Reports Nos. 2 and 4 indicate numerous deficiencies in several categories including wet wells, pumping equipment, dry wells/control rooms, control, power supply, and operation and maintenance programs.

Table G-2 lists the deficiencies of wet wells. All pump stations have inadequate, sometimes dangerous, access to wet wells; more have wet well lighting or ventilation. Most wet well designs are insufficient and exhibit poor suction inlet design.

Table G-3 lists deficiencies of pumping equipment components. All except one pump station have undersized suction and/or discharge piping and many have inappropriately selected pumps. Fault current protection is typically inoperable. Several stations have inadequate pumping capacity.

Table G-4 lists pump station dry well and control room deficiencies. These typically include inadequate lighting and ventilation, lack of discharge flow metering, interconnected wet and dry wells and lack of automatic sump pump operation.

Table G-5 lists operation and maintenance deficiencies. Of these, the most serious is operation causing surcharge of incoming sewers. Nearly all pump stations would benefit from better recordkeeping, operator training, operation and maintenance manuals, and an inventory of spare parts.

Power Supply

All of the eastern zone pump stations excepting Mamoura and Montazah are connected to a 10.5-kV power network. These, as well as several others in other zones, were constructed as part of the main drainage project during the years 1965-1974 and have similar electrical equipment. The

10.5-kV switchboards at these pump stations were constructed with draw-out type "minimum oil" circuit breakers rated 350 MVA at 10.5 kV for overcurrent and fault current protection. The circuit breakers have adequate capacity for the system requirements and generally appear to be in good, clean condition. Failure of the d.c. power supplies for the circuit breaker control systems has rendered the protective relays for the circuit breakers inoperative. The breakers are now used as disconnect switches to connect the switchboards and do not provide the protection required by the system. In addition, a number of interconnecting cables are reportedly faulted. This reduces the flexibility and reliability of the system.

Each 10.5-kV switchboard on the loop supplies power to its pump station through two feeders; each feeder has its own transformer. The transformers are suitable for outdoor installation but are enclosed in rooms. Consequently, they appear to be in excellent condition. The cabling to and from the transformers also appears to be in good condition. The oil level in the transformers at each pump station inspected was found to be within the appropriate limits. The sudden pressure relays on most of the transformers were not connected into the circuit breaker trip circuit. Thus low-level, phase-to-ground faults in the transformer windings could go undetected until they became large enough to trip phase overcurrent protection devices in the system.

The low-voltage switchboard in each of the pump stations inspected had at least two main switch assemblies and two services from either the utility or the onsite 10.5-kV transformers. Several switchboards had a third switch assembly for connection of an emergency engine-generator. The switchboards also include a switch assembly to provide power to a remotely located starter for each pump in the station, controls and fuses for auxiliary equipment in the station, and fuses for lights and outlets. Each switch assembly consists of a manually operated 3-pole knife switch, three fuses, and an air interrupter switch. The interrupter switch can be manually closed and either manually or automatically opened.

The 380-volt switchboards are of an old design but are functional for the present. The switch assemblies have adequate capacity and appear to be in relatively good condition except that the automatic trip systems for the air interrupter switches do not operate. They receive d.c. power from the same power supply as the 10.5-kV systems, and the failure of those has rendered all fault current protection in the stations inoperative except for the fuses. The protection provided by the existing fuses is questionable and installation of new fuses of modern design is recommended.

Control of small pumps is provided by across-the-line starters; large pumps are controlled by wye-delta starters, and units 200 hp and larger

are started using resistor banks, which are connected through slip rings into the rotor circuits of the motors. The existing starters have had considerable use and in some cases have been replaced by new equipment.

RECOMMENDED IMPROVEMENTS

Operation

Discharging sewage into the Mediterranean near the Alexandria beaches should be stopped as soon as possible. This is particularly true for the summer season. In addition, even though many of the pump stations may be abandoned or upgraded regardless of which alternative treatment and disposal method is selected, they must continue to operate in the interim. For these reasons, the pump stations were specifically investigated as to their ability to meet current and near future (through 1988) needs.

Most pump stations are fed by gravity collectors that are designed to maintain fluid velocities at 0.6 m/s or greater. This velocity will keep solids in suspension but occurs only when flow is nonrestricted. When the flow is restricted, as happens when the wet well level is allowed to rise above the invert of the inlet sewer, then collector velocities are slowed and suspended solids settle. A velocity of about 1.1 m/s, or almost twice the design velocity, is required to resuspend these solids. Thus, surcharging is extremely detrimental to overall system operation. Maximum wet well liquid levels should never be allowed to reach the level of the incoming sewer invert. In nearly all cases, keeping the wet well level below the invert of the incoming sewer will require more frequent pump operation, which will increase costs for repair parts and labor. However, the money saved by not having to clean the sewers as frequently will far outweigh the increased repair costs.

To assure that wet well levels are maintained within acceptable limits, the liquid level in each should be continuously monitored and recorded on a 30-day strip chart. Alarms at the high and low extremes of acceptable limits should be included to assure operator attentiveness.

Pump stations that are manned 24 hours per day should be operated in three 8-hour shifts with at least two people per shift. The operators should be required to keep records of all events that take place in the station. The records should include the times each pump is started and stopped, the number of pumps running, total run time of each pump, wet well levels at which pump operation is changed, special happenings, and changes in operators. The record can be used to assure that the operation of the pumps is alternated and that the total run time on each

motor is balanced so the equipment wear is even and routine maintenance can be scheduled.

Wet well fluctuation records will allow the supervisory personnel to accurately determine pumping demands and forecast future needs of the stations. The records can be used by the area supervisor to monitor maintenance programs and forecast future financial requirements to assure continuous successful operation.

Maintenance

General maintenance, which is common to all pump stations, may best be carried out by one crew of men who work on all stations in their area. One truck and two men could carry supplies and visit all stations on a regular schedule or by emergency phone call. The crew would be responsible for replacing lights, windows, and fixing door locks. Broken windows should be replaced as soon as possible to decrease dust and water vapor contact with electrical and mechanical equipment. Lights should be replaced as needed to facilitate accident-free nighttime work. All exposed electrical wires should be covered to prevent accidental contact during normal work routines. General maintenance should also include periodic pump checks with pressure gauges to check impeller wear, wearing ring spacing, and force main plugging.

Cleanliness of each station presently depends upon the man in charge of that station. There does appear to be a difference in cleanliness from station to station. Since there is a person responsible for all the stations, a set of minimum standards should be prepared. All station supervisors should be required to conform to these standards. Unannounced periodic visits may be required for a time at some stations to see that adherence to standards is accomplished. This set of minimum standards should include:

- Floors should be grease-free, so workers can walk without slipping. An emulsifier such as a detergent or solvent must be used to clean the floors.
- Motors should be clean and free of excess grease.
- Ladders and handrails should be clean and dry.
- Screenings should be properly stored in containers prior to disposal.
- Burned out light bulbs should be replaced.

- Personal belongings should be properly stored.
- Floors and grounds should be free of trash.
- Dry sawdust or powdered clay should be used to soak up oil spills. Sand does not work as well.
- Walls should be cleaned and brightly painted, in order to reflect available light. Windows should be cleaned to allow more light to enter.

Minor repairs and improvements in housekeeping techniques are needed in all stations. Pump stations appear to be in better condition than was reported following the inspections made during 1977. Safety is still an item that needs to be emphasized.

The structural condition of most of the stations is fairly good and no major work is required.

Electrical Systems

The electrical system of all pump stations inspected by WWCG is very deficient in the area of fault current protection. Similar deficiencies probably exist at other stations. Most of the equipment is in fairly good condition, but the failure of the d.c. power supplies has rendered the protective relays systems inoperative. The following electrical improvements should be made:

- Repair or replace deficient d.c. power supplies for the 10.5-kV and 380-volt switchboards.
- Check the operation of all protective relays and circuit breaker control systems and recalibrate the protective relays.
- Modify the existing 380-volt fuse holders to accept time delay fuses of modern design suitable for starting motors.
- Locate and repair damaged underground electrical cables in the eastern zone 10.5-kV distribution system.
- Replace all faulty current transformers used for protective relay operation.
- Repair all strip chart recorders, obtain new ink and paper charts, and put recorders back into service.

- Cover all exposed electrical connections and replace or repair wiring where loss of insulation has exposed bare conductors.
- Replace all burned out lamps and rewire or replace all inoperative fixtures.
- For all pump stations that are not provided with redundant power supplies, a standby engine generator, either portable or stationary, should be provided.

Mechanical Systems

The following mechanical system improvements are recommended:

- In all pump stations to be retained, replace present ladders to dry well floors with circular stairs or stairs mounted on the dry well wall.
- Extend dry well exhaust fan ducts within 30 cm of the floor in all stations.
- Replace or repair any defective dry well exhaust fans.
- Repair or replace all existing dry well sump pump float systems and install automatically controlled sump pumps.
- Pump stations scheduled to remain in service for an extended period of time should have effluent flow measuring and recording devices installed. For those which require new pumps, accurate flow rates and discharge head requirements should be determined.
- Pump stations scheduled to be rebuilt or upgraded and remain in service should have the discharge piping reworked to eliminate sewage pump and check valve plugging.

Safety

The following safety improvements are recommended:

- Purchase new dry chemical fire extinguishers for all stations.
- Institute a fire extinguisher use and training program.

- Replace broken window glass in pump stations.
- Remove excess grease from all pumps and motors.
- Stock mops, pails, and detergents for cleaning floors.
- Install lockable cabinets for storage of cleaning supplies, strip charts, light bulbs, and other general maintenance materials.

Specific Improvements

The following improvements are based on Special Report No. 2 and supplemented by WWCG's inspection of nearly half of the pump stations.

Sporting. Special Report No. 2 recommends replacement of this pump station. WWCG concurs with that recommendation. The need for this station is not affected by the selection of alternative wastewater management plan. The top priority project for construction of this facility should proceed. This station should not experience a capacity problem in the near future. It has sufficient pump capacity and, if the force main can be kept in service, there should not be a problem in discharging peak flows.

Bisiry. Regardless of the wastewater management plan selected, the Bisiry pump station may remain in operation, as it is required to relieve local flooding. If it is retained, ventilation, lighting, and screening should be installed in the wet well. New pumps should be installed or the existing ones modified to meet the present head conditions. Standby power should be provided. Also, the dry well should be provided with better lighting and ventilation than at present. An automatic sump pump, a discharge flowmeter, and a modified access to provide safety to operating personnel should be installed.

Sinin Pasha. The Sinin Pasha pump station also may remain in operation regardless of the recommended wastewater management alternative selected. It may be needed to relieve local flooding. The same improvements recommended previously for the Bisiry pump station apply to the Sinin Pasha station.

Kait Bey. The existing Kait Bey pump station will be abandoned in the future regardless of which treatment and disposal alternative is selected. The station will, however, need to be maintained and operated until the selected alternative is constructed. For this reason, WWCG recommends installation of a more adequate power supply, and repair of the discharge flowmeter.

No. 1 West. No conclusion about the fate of the No. 1 West pump station was reached in Special Report No. 2 because its operation was dependent on completion of the West Treatment Plant. This plant is still not operable. Regardless of the wastewater management plan selected, WWCG recommends that this station be abandoned when the plan is implemented. In the meantime, it is not needed until at least the ongoing West Treatment Plant construction is completed.

No. 2 West. WWCG recommends that this station be retained and rebuilt for extended future use regardless of the wastewater management plan selected. The station now discharges to the Western Harbor. Regardless of the selected plan, No. 2 West will pump to a gravity sewer. In the interim, (upon completion of the current West plant construction) it should pump to the No. 3 West pump station. It currently has sufficient capacity. Future needs depend on the plan selected. In the meantime, the following interm improvements are recommended: repair pumps as needed so all units are operable, provide a discharge flowmeter, and repair or replace the electric service cable between the transformer and the 380-V switchboard.

No. 3 West. This pump station will not be needed after implementation of the selected plan. It will be needed in the interim period between completion of the current West plant construction and the completion of the selected plan. This may require replacement pumps because the present ones have not been maintained. In the interim, WWCG recommends constructing a temporary pipeline to convey the discharge directly to Lake Maryout instead of the nearby agricultural drain. This pipeline would also receive flow from the Mohsein Pasha, old Furn El-Gueraya, and Gheit El-Enab pump stations, which are also to be abandoned.

New Furn El-Gueraya. This station cannot be operated until the West Treatment Plant is completed, and it will not be needed after implementation of the selected plan. Currently, the installed pumps do not have the capability necessary to pump to the West plant. During the interim period, after completion of the current West plant construction and before completion of the selected plan, pumps with higher head capability will be required.

Old Furn El-Gueraya. This pump station has been abandoned and will not be needed in the future regardless of the wastewater management plan selected.

Industries. This pump station will be retained and rebuilt for extended future use regardless of the plan selected and implemented. Prior to plan implementation or completion of the current West plant construction, interim repairs and reconditioning are recommended. These include: install new manually cleaned screens, provide a discharge flowmeter, and repair any leaking pipes.

Gheit El-Enab Main and Auxiliary, and Mohsein Pasha. Regardless of the wastewater management plan selected, WWCG recommends abandonment of these pump stations. Until the selected plan is implemented, these stations must continue in use and will require interim improvements. Recommended improvements include the temporary discharge pipeline to Lake Maryout previously described. For the Gheit El-Enab main pump station, WWCG recommends replacing any unusable pumps.

No. 1 East. This pump station will be retained and rebuilt for extended future use regardless of the wastewater management plan selected. To meet the immediate needs of the 1981 summer peak flow, sufficient power to run two pumps at high speed is required. Existing pumps are too small for present peak flows and force main velocity is excessively high. Diversion of Mamoura summer flows to No. 2 East should enable meeting 1981 peak requirements. Additional interim improvements recommended include: inspect and replace (if required) the section of steel force main 40 m long under the Corniche at Miami, install additional pump capacity to meet peak summer demand, repair the existing or install a new discharge flowmeter.

No. 2 East. Regardless of the wastewater management plan selected, the No. 2 East pump station may remain in operation for stormwater only. In the meantime, until a plan is implemented, it must continue to operate. Immediate 1981 needs can possibly be met by diverting Mamoura wet weather flows to No. 1 East and completing repair of the second high-speed pump. In the interim, additional pump capacity to meet peak summer demand should be installed. The new pumps presently on order for this pump station do not match the present system head requirements and, unless a larger force main is installed, use of these pumps will not materially increase the station's capacity.

No. 3 East. This pump station will be replaced with a larger facility regardless of the wastewater management plan selected. The station has sufficient capacity to accept present peak flows and the force main is amply sized. The present pumps will require replacement before 1987. The backflow prevention values should be inspected and replaced if required.

No. 4 East. This pump station will be retained and rebuilt for extended future use regardless of the wastewater management plan selected. The following interim improvements are recommended: install new pumps on order, provide a separate standby stormwater pump, and provide a discharge flowmeter. Current capacity is sufficient for immediate needs; therefore, immediate pump replacement other than for reasons of wear is not urgent.

No. 5 East. Regardless of the wastewater management plan selected, the No. 5 East pump station may remain in operation for stormwater only.

Until a plan is implemented, it must continue to operate. To meet peak summer flow conditions, two high-speed pumps are required. In the interim, the force main should be inspected and any damaged portions should be repaired. Additional transformer capacity should be provided. This latter improvement would permit operating two pumps at high speed to provide needed station capacity. This could be accomplished by supplying two new 500-kVA transformers, or connecting the existing 300-kVA transformers in parallel and purchasing a standby 300-kVA unit. The latter is the least costly alternative. Finally, the installation of the wye-delta starter should be completed. Depending on when the selected plan is implemented, larger interim pumps and force main replacement may be required.

No. 6 East. This pump station will be retained and rebuilt for extended future use regardless of the wastewater management plan selected. In the interim, a discharge flowmeter should be installed. The section of damaged force main that crosses the Montazah Canal should also be repaired.

No. 7 East. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. In the meantime, it must continue to operate. Until then, a new emergency discharge should be constructed. This would permit better operation when the regular force main is out of service or the No. 11 East pump station is down.

No. 8 East. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. In the meantime, it must continue to operate and will require a new emergency discharge.

No. 9 East. Implementation of any of the conveyance system alternatives discussed in Appendix M will result in abandonment of this pump station. It is not used now and can remain out of service.

No. 10 East. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. In the meantime, until a plan is implemented, it must continue to operate. Construction of the emergency force main should be completed. Another recommended interim improvement is installation of additional pumping capacity to meet existing requirements.

No. 11 East. This pump station will be replaced with a larger facility regardless of the wastewater management plan selected. The March 1981 report by WWCG on the evaluation of No. 11 East pump station recommended the following interim improvements: install new screens, replace defective piping and valves in the dry well and header outside the valve vault,

investigate and install surge control devices, consider installing variable speed pumping capabilities. Other recommended improvements are to repair the force main leak at the Mahmoudiya Canal undercrossing, and complete construction of the emergency force main. Surcharging the collection system cannot be prevented because the pump volume is set too high. However, the station should be operated in a manner to minimize this surcharging.

Montazah. Regardless of the wastewater management plan selected, the Montazah pump station may remain in operation for stormwater only and must continue to operate until a plan is implemented. In the interim, the following improvements should be made: install the pumps that are on order as soon as warranted by existing pump wear, and revise the existing standby pumping capabilities to permit discharge to the force main instead of the sea during power outages. The station currently has sufficient capacity to meet needs through the interim period.

Mamoura. This pump station will be retained and rebuilt for extended future use regardless of the wastewater management plan selected. Recommended interim improvements until an alternative conveyance system is implemented are as follows: provide discharge flow metering, install at least one of the pumps that are on order, modify the existing standby pumping capabilities to permit discharge to the force main instead of the drain during power outages, and replace the failing asbestos cement force main. The station currently provides sufficient capacity for present flow conditions. The discharge force main should be valved to permit diversion of summer peak flows to No. 2 East and wet weather flows to No. 1 East.

Hospitals. The currently inoperable Hospitals pump station should be replaced as an interim improvement or connected to the Ras El-Soda system now under construction. The capital cost of an interim replacement is included in Table G-6. Connection to the Ras El-Soda system would require early construction of a portion of the Mohandes collector.

Sidi Bishr. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. The following interim improvements are recommended: install additional pumping capacity to meet summer requirements beyond the year 1981, provide ventilation for standby power system, and repair water leaks into the dry well during winter storms. The practice of resting the pumps in the winter should be eliminated to prevent sewage discharge to the sea.

Sarwat. Implementation of any alternative plan will eliminate the need for the Sarwat pump station. The following improvements are recommended for the interim period: install new manually cleaned screens and add variable speed pumping capacity to meet summer demand. Without addi-

tional capacity the station cannot meet anticipated 1981 summer flows. The practice of resting pumps in the winter season should be eliminated to prevent sewage being discharged to the sea.

Glym. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. In the meantime, it must continue to operate. The following interim improvements are necessary and recommended to meet current peak discharge requirements: provide new manually cleaned screens and new, larger pumps for summer demand. The practice of resting the pumps should be eliminated to prevent sewage being discharged to the sea. Finally, the construction of the new fiberglass force main should be completed.

Sidi Gaber. The Sidi Gaber pump station presently pumps only stormwater and is expected to continue operating in the near future. No short-term improvements are recommended.

Smouha. Regardless of the wastewater management plan selected and implemented, WWCG recommends eventual abandonment of this pump station. In the meantime, it must continue to operate. The following interim improvements are recommended: replace screens, install new pumps that are on order, and repair any leaking valves. The steel force main under the bridge in Ibrahimia should be inspected and repaired if faulty.

COSTS OF IMPROVEMENTS

Estimated capital costs for improvements are listed in Table G-6.

The cost of general improvements such as maintenance, safety items, and overall upgrading is included for all 32 pump stations. The cost estimates for specific improvements does not include pump station replacement costs, other than the replacement of the Hospitals pump station. The long-range future of many pump stations and force mains depends upon the final decisions to be made after the Master Plan Review and the implementation of the selected conveyance system alternative from Appendix M. For this reason, the costs included are for just immediate needs during this interim period.

Table G-1
SUMMARY OF RECOMMENDATIONS FOR PUMP STATIONS IN PREVIOUS REPORTS

Pump Stations	Special Report No. 2	Special Report No. 4	1978 Master Plan Sea Disposal Option	1978 Master Plan Land Application Option
No. 1 East	R, Rst	R	Rst	Rst
No. 2 East	R, Rst	R	Rst	Rst
No. 3 East	R, Rst		Rst	Rst
No. 4 East			Rc	Rc
No. 5 East		R	Rst	Rst
No. 6 East		R	Rc	Rc
No. 7 East		R	A	A
No. 8 East		R	A	A
No. 9 East	A		A	A
No. 10 East		R	A	A
No. 11 East		R	R	A
Montazah	Rc	R	Rc	Rc
Mamoura	Rc		Rc	Rc
Hospitals		R	Rc	Rc
No. 1 West			A	A
No. 2 West			Rc	Rc
No. 3 West			A	A
New Furn El-Gueraya	A		A	A
Industries			A	A
Old Furn El-Gueraya	A		A	A
Ghelt El-Enab Main	RP		A	A
Ghelt El-Enab Auxiliary	A		A	A
Sidi Bishr	RP	RP	RP	RP
Sarwat	RP	RP	A	A
Glym	RP	RP	A	RP
Sporting	RP	RP		
New Sidi Gaber	A			
Smouha	A		A	A
Mohsein Pasha	A		A	A
Bisiry				
Sinin Pasha				
Kait Bey	RP		A	RP

Legend:

A: Abandon
R: Rebuild
RP: Replace
Rc: Retain for combined flows
Rst: Retain for stormwater only
Rw: Retain for wastewater only

TABLE G-2

PUMP STATION WET WELL DEFICIENCIES

WET WELL DEFICIENCY NOTED	SPORTING	BIBAY	SHAHN PASHA	KAIT BEY	1W	2W	3W	NEW FURN EL-GUERAYA	OLD FURN EL-GUERAYA	INDUSTRIES	GHEIT EL-ENAB MAIN	GHEIT EL-ENAB AUX	MOHSEIN PASHA	1E	2E	3E	4E	5E	6E	7E	8E	9E	10E	11E	MONTAZAH	MAMOURA	HOSPITALS	SIDI BISHR	BARWAT	GLYM	SIDI GABER	SMOUHA
INADEQUATE ACCESS	x	x	x	a	x	x	x	x	b	x	c	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
NO VENTILATION	x	x	x	x	x	x	x	x	x	x	c	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
NO LIGHTING	x	x	x	x	x	x	x	x	x	x	c	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
INSUFFICIENT BOTTOM SLOPE	x	x	x	x	x	x	x	x		x	c	x		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	
NO SEWAGE SCREENS	x	x	x						x	x																		x	x	x		
INEFFICIENT DAMAGED OR PARTIAL SCREENS										x										x	x			x								x
POOR SUCTION INLET DESIGN	x	x	x	x	x	x	x	x		x		x		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CRACKED INFLUENT SEWER																							x									

SOURCES WWCG inspection and/or Special Report No 2

a Access is highly dangerous for maintenance personnel

b Access through dry well is very poor

c Wet well was inaccessible during inspection

TABLE G-3

PUMP STATION PUMPING EQUIPMENT DEFICIENCIES

PUMP STATION

DEFICIENCY	SPORTING	BISHRY	SHNIN PASHA	KAIT BEY	1W	2W	3W	NEW FURN EL-GUERAYA	OLD FURN EL-GUERAYA	INDUSTRIES	GHEIT EL-ENAB MAIN	GHEIT EL-ENAB AUX.	MOHSEIN PASHA	1E	2E	3E	4E	5E	6E	7E	8E	9E	10E	11E	MONTAZAH	MAMOURA	HOSPITALS	SIDI BISHR	BARWAT	GLYM	SIDI GABER	SMOUHA
INAPPROPRIATE PUMP SELECTION		x	x					a		x						x	x		b	x	x	x		x	x	x	x		x	x		x
PUMP(S) OPERATE OUT-SIDE RATED RANGE						x	x	x								x								x								
PUMPS/VALVES LEAKING										x																						x
PUMP(S) INOPERABLE AT HIGH SPEED															x			x														
INADEQUATE OR NO STANDBY PUMPS																	x								x			x	x			
PUMP(S) INOPERABLE						x																	c									
HIGH SUCTION AND/OR DISCHARGE VELOCITY	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
ONE POWER SUPPLY	d	d	d	x	d		d	d	d	d		d	d									x			x		d	d	d		d	d
INSUFFICIENT POWER SUPPLY				x														x														
POOR FAULT CURRENT PROTECTION				x		x								x	x	x	x	x	x	x	x	x	x	x								
NO GENERATOR	x	x	x		x	x		x	x	x	x				e		x		x	x	x	x		x	x	x	x	x	x	x	x	x

SOURCES WWCG inspection and/or Special Report No. 2.

a Extremely poor selection. TDH at rated discharge will exceed shut off head.

b For stormwater pumps

c At high speed

d Unknown

e Generator too small and out of service

TABLE G-3 (continued)

PUMP STATION PUMPING EQUIPMENT DEFICIENCIES

PUMP STATION

DEFICIENCY	SPORTING	BISHRY	SHIHM PASHA	KAIT BEY	1W	2W	3W	NEW FURN EL-GUERAYA	OLD FURN EL-GUERAYA	INDUSTRIES	GHEIT EL-ENAB MAIN	GHEIT EL-ENAB AUX.	MOHSEIN PASHA	1E	2E	3E	4E	5E	6E	7E	8E	9E	10E	11E	MONTAZAH	MAMOURA	HOSPITALS	SIDI BISHR	SARWAT	GLYM	SIDI GABER	SMOUHA
GENERATOR INADEQUATE				f			x					x		x	x			x														
GENERATOR NOT AUTO-START														x	x	x																
GENERATOR IMPROPERLY VENTED																												x				
GENERATOR NOT HOUSED							x					x																				
INSUFFICENT PUMPING CAPACITY									g					x	x	x	x						x			x	x	x	x			x
DISCHARGE PIPING GROSSLY MIS-SIZED								x																								
IMPROPER VALVING														x								h										
PUMPING EQUIPMENT WORN OUT									x		x																					x
SUCTION LIFT REQUIRED				x									x											x								
POSSIBLE SURGE PROBLEMS																																

f Reported that only two pumps can be run on generator even though it should be able to run four with step ring starting.

g For wet weather flows

h Discharge piping not connected

TABLE G-4

PUMP STATION DRY WELL AND CONTROL ROOM DEFICIENCIES

PUMP STATION

DEFICIENCY	SPORTING	BISIRY	SIMIN PASHA	KAIT BEY	1W	2W	3W	NEW FURN EL-GUERAYA	OLD FURN EL-GUERAYA	INDUSTRIES	GHEIT EL-ENAB MAIN	GHEIT EL-ENAB AUX.	MOHREIN PASHA	1E	2E	3E	4E	5E	6E	7E	8E	9E	10E	11E	MONTAZAH	MAMOURA	HOSPITALS	SIDI BISHR	SARWAT	GLYM	SIDI GABER	SMOUHA
INOPERABLE VENTILATION	X																													X		
INADEQUATE VENTILATION	X	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	a	X	X		b
SUMP PUMP NOT AUTOMATIC	X	X	X		X	X	X	X		X	X	X		X	X	X		X	X	X	X	X	X	X		X	X		X	X		X
NO DISCHARGE FLOWMETER	X	X	X		X	X	X	X	X	X	X	X	c	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
INADEQUATE LIGHTING	X	X	X		X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X		X
INTERCONNECTED WET/DRY WELLS	X								X					X	X	X	X	X	X				X		X	X		X	X	X		X
DRY WELL ACCESS INADEQUATE/UNSAFE	X	X	X		X	X	X	X		X	X	X			X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	
SUMP PUMP NOT OPERABLE																											X					
MOTOR SUBJECT TO FLOODING							X		X		X																X	X				
ELECTRICAL CONDUIT SUBJECT TO FLOODING									X																		X	X				
SURFACE WATER ENTERS STATION											X																	X				
INADEQUATE SPACE	X										X						X													X		
REGULAR FLOODING							X																				X					

SOURCES WWCG inspection and/or Special Report No. 2.

a None provided

b Not operable

c Has nonfunctional Venturi meter

TABLE G-5

PUMP STATION OPERATION AND MAINTENANCE DEFICIENCIES

PUMP STATION

DEFICIENCY	SPORTING	BISHRY	BININ PASHA	KAIT BEY	1W	2W	3W	NEW FURN EL-GUERAYA	OLD FURN EL-GUERAYA	INDUSTRIES	GHEIT EL-ENAB MAIN	GHEIT EL-ENAB AUX.	MOHSEIN PASHA	1E	2E	3E	4E	5E	6E	7E	8E	9E	10E	11E	MONTAZAH	MAMOURA	HOSPITALS	SIDI BISHR	SARWAT	GLYM	SIDI QABER	SMOUHA
IMPROPER OPERATION CAUSES SURCHARGE	X	X	X			X	X		X	X		X		X	X	X	X	X	X	X	X			X	X	X	X	X	X	X		X
IMPROPER SCREENINGS HANDLING						X			X																							
DISCHARGE POINT CREATES UNNEEDED HAZARD	a					X	X			X	X			b	b	b	b	b	b	c	c		c	c	d	X	X	a	a	a		X
STATION UNATTENDED -- NOT OPERATED		e	e		f			X	X																		X					
NO O&M MANUALS	X	X	X	X		X	X		X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X
NO SPARE PARTS INVENTORY	X	X	X			X	X		X	X	X	X	X	X	X	X		X	X	X	X		X	X	X	X	X	X	X	X	X	X
NO O&M RECORDS	X	X	X	X		X	X		X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X
NO ROUTINE INSPECTION RECORDS	X	X	X			X	X		X	X	X	X		X	X	X		X	X	X	X		X	X	X	X	X	X	X	X	X	X
NO O&M TRAINING PROGRAM	X	X	X	X		X	X		X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X
DETENTION IN WET WELL TOO LONG	X	X	X			X	X	X		X		X		X	X	X	X	X	X	X	X		X		X	X	X	X	X	X	X	X

SOURCES: WWCG inspection and/or Special Report No. 2.

a "Resting" pumps during winter allows unnecessary pollution of the sea

b Winter discharge to sea causes unnecessary pollution

c During power failures

d During winter and during power outages

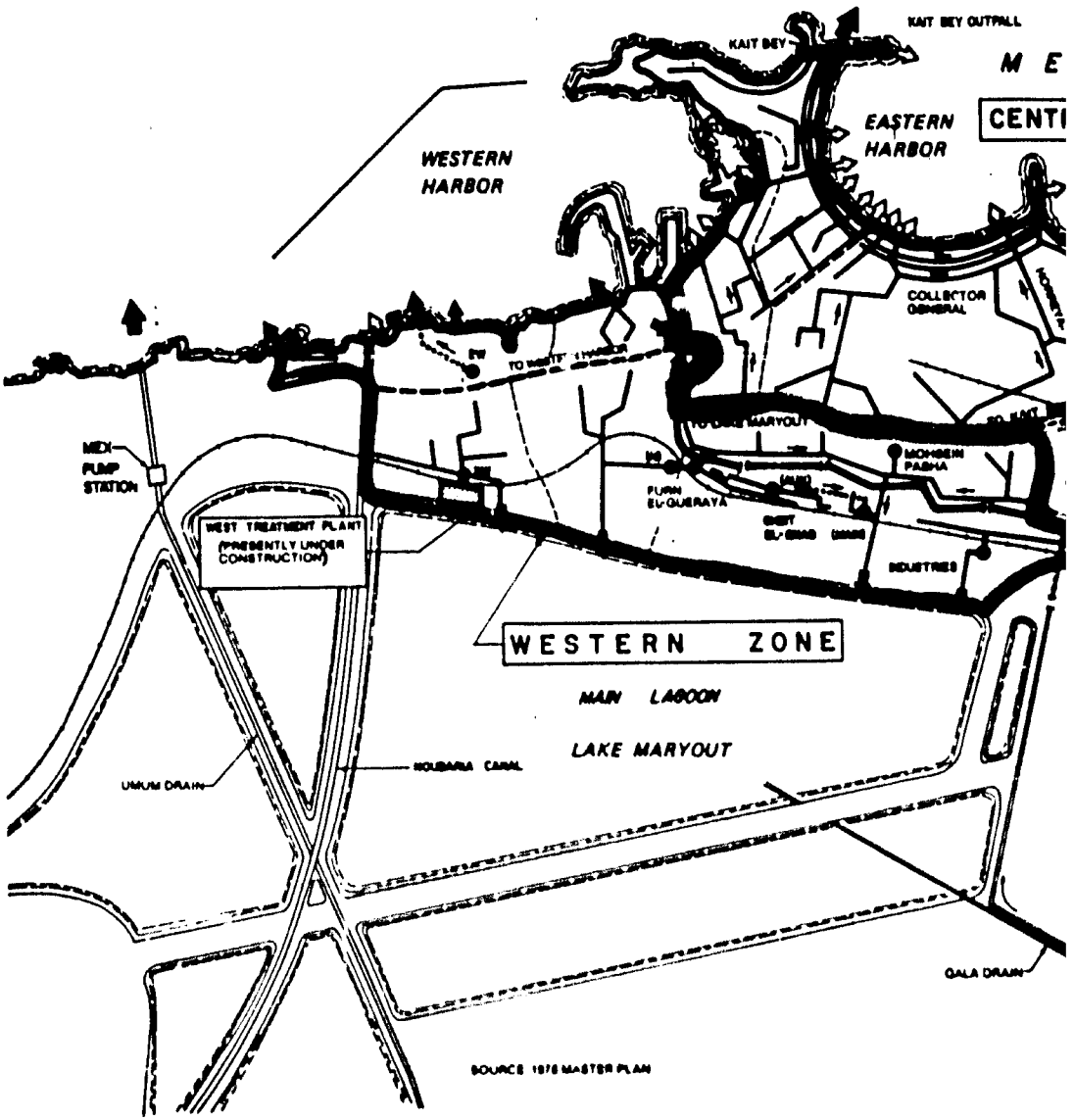
e Custodial services only during dry months

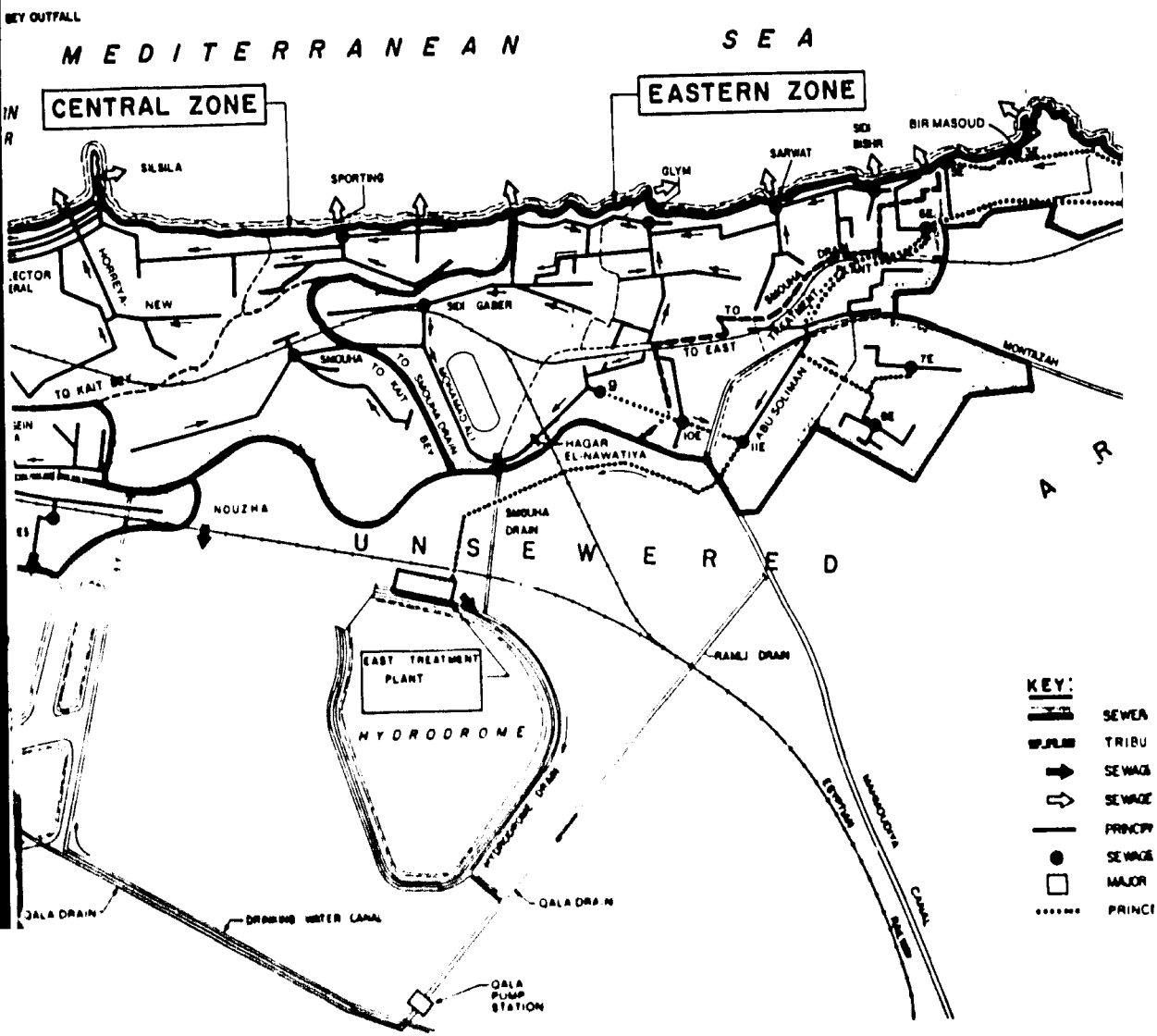
f Only custodial services provided

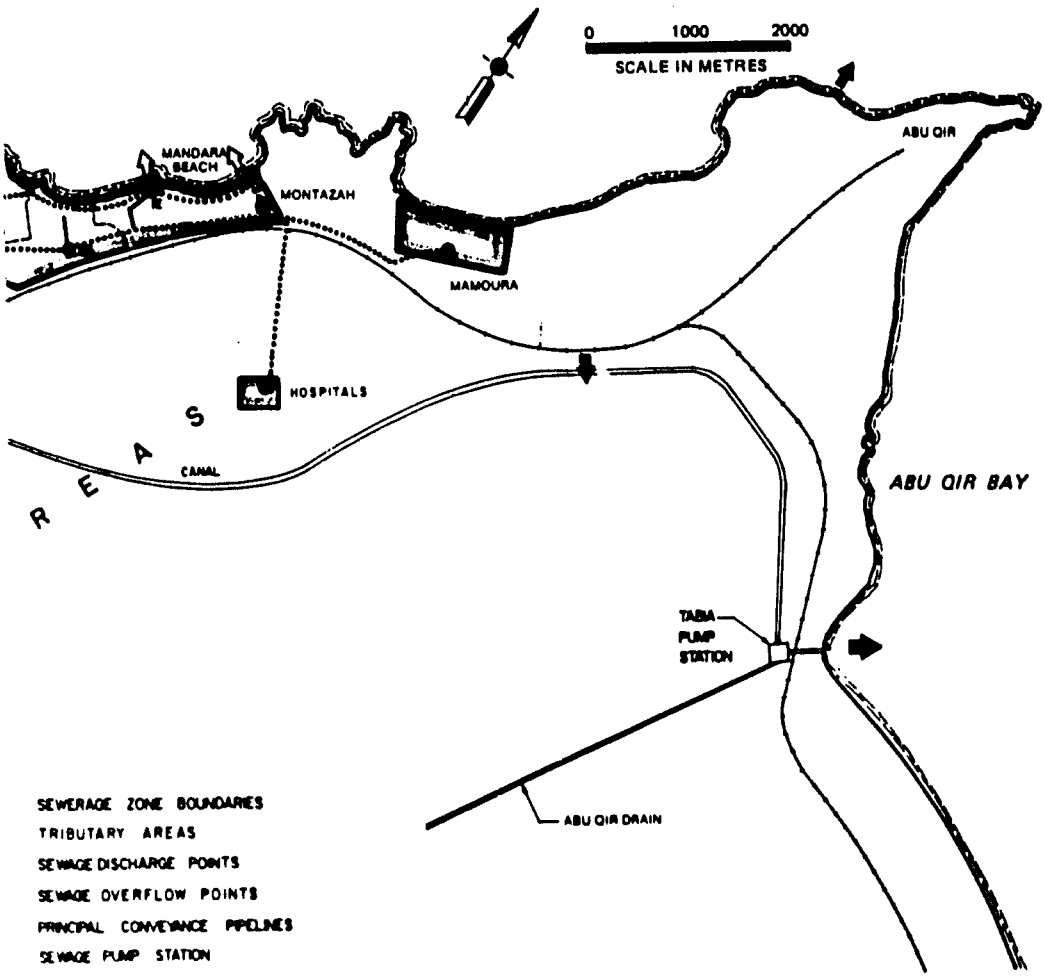
Table G-6
ESTIMATED CAPITAL COSTS FOR
EXISTING PUMP STATION IMPROVEMENTS

Item	Cost[a] (LE)
Electrical	142,000
Mechanical	7,293,000
Maintenance & Safety	92,000
Hospitals Pump Station	104,000
Subtotal	7,631,000
Contingencies @ 25%	1,908,000
Subtotal	9,539,000
Engineering, Legal, and Administrative Costs @ 15%	1,431,000
Total	10,970,000

[a]End-of-1980 cost levels


KAIT BEY OUTFALL
KAIT BEY
M E
CENTI
EASTERN HARBOR
WESTERN HARBOR
COLLECTOR GENERAL
TO WEST HARBOR
TO LAKE MARYOUT
MOHGEIN PASHA
PUMP EL-QUERAYA
MEX PUMP STATION
WEST TREATMENT PLANT (PRESENTLY UNDER CONSTRUCTION)
INDUSTRIES
WESTERN ZONE
MAIN LAGOON
LAKE MARYOUT
UMUM DRAIN
NOUBARIA CANAL
QALA DRAIN
SOURCE 1976 MASTER PLAN


BEY OUTFALL
MEDITERRANEAN SEA
CENTRAL ZONE
EASTERN ZONE
SILSILA
SPORTING
GLYM
SARWAT
SIDI BISHR
BIR MASOUD
NEW
SIDI GABER
TO EAST
TO KAIT BEY
SMOUHA
TO KAIT BEY
SMOUHA DRAIN
HAGAR EL-NAWATIYA
ABU SOLIMAN
MONTAZAH
NOUZHA
SMOUHA DRAIN
UNSEWERED
EAST TREATMENT PLANT
HYDRODROME
RAMLI DRAIN
QALA DRAIN
DRINKING WATER CANAL
QALA PUMP STATION
MAHMOUDIYA CANAL
KEY:
SEWER
TRIBU
SEWAGE
SEWAGE
PRINCIP
SEWAGE
MAJOR
PRINCI



SEWERAGE ZONE BOUNDARIES
TRIBUTARY AREAS
SEWAGE DISCHARGE POINTS
SEWAGE OVERFLOW POINTS
PRINCIPAL CONVEYANCE PIPELINES
SEWAGE PUMP STATION
MAJOR DRAINAGE PUMP STATION
PRINCIPAL FORCE MAINS

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE – MAY 1981

FIGURE G-1

EXISTING SEWERAGE SYSTEM

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APPENDIX H

COLLECTION SYSTEM REVIEW

This appendix reviews the existing Alexandria collection system and recommends short-term projects that will improve the system prior to the implementation of recommended Master Plan projects. Included in this appendix are: a description of the system in general, a review of its present condition, a review of the 1978 Master Plan recommendations, a program of recommended short-term improvements, and a cost estimate for these improvements.

EXISTING SYSTEM

Description

The extent of the drainage area of Alexandria presently served by sewers is indicated on Figure H-1. There are 20 individual tributary areas noted on this plan. These areas are grouped into three zones which correspond to the grouping used by A/GOSD and are called the central, western, and eastern zones. Figure H-1 shows only major interceptors, force mains, and pump stations; a complete inventory of the collectors, pump stations, and force mains, as well as a description of the treatment works are contained in Special Report Number 2 for Alexandria Sewerage Study, dated April 1977.

Central Zone. The oldest sewered area in Alexandria is the central zone. In the western portion of the central zone, two separate systems collect wastewater for conveyance to two separate discharge points. The Port Est system has main collectors radiating out of the Port Est collector, which passes along the Eastern Harbor to the Kait Bey pump station, and then out to sea via the Kait Bey outfall. The remaining portion of the central zone is served by local sewers that discharge directly into the Western Harbor. One of the top priority projects (IPP) recommended by the 1978 Master Plan (now under construction) will intercept the flow now emptying into the Western Harbor and pump it to the Port Est system, where it will be temporarily discharged through the existing Kait Bey outfall.

In the eastern portion of the central zone, three systems convey the wastewater to eventual disposal through the Kait Bey outfall. Two of the systems are served by the Sporting and Smouha pump stations, which pump the flows into the third system. The combined wastewater is then conveyed by gravity to collectors which parallel the Eastern Harbor shoreline and terminate at the existing Kait Bey pump station.

Stormwater that exceeds the capacity of the conduits overflows at several points into the Eastern Harbor. Excess flow at the Sporting pump station flows directly to the Mediterranean from the station while excess from the Smouha pump station flows by gravity to the Smouha Drain.

Western Zone. The western zone system is now in an interim state of construction and operation because the West Treatment Plant has not been completed. Currently, flows from three northern tributary areas flow directly to the Western Harbor, while those from the four southern areas go directly to Lake Maryout. When the West Treatment Plant is completed, all flows are planned to be pumped there for treatment. Prior to implementation of the Master Plan recommendations, effluent from the West Treatment Plant will be disposed of in Lake Maryout.

Eastern Zone. The eastern zone system conveys all its flow to two points to the south. The western portion of the zone flows to the Smouha Drain, a large canal located south of the Mahmoudiya Canal. This drain receives the gravity flows from two large collectors that discharge under the Mahmoudiya Canal through a siphon and eventually to Lake Maryout. The two collectors, the Hagar El-Nawatiya and the Mohamed Ali, serve large areas adjacent to the Smouha Club. The Sidi Bishr, Sarwat, and Glym pump stations serve the areas north along the Mediterranean and pump in series to the Mohamed Ali system. The Sidi Gaber pump station is currently not operating, but is planned to pump stormwater to the sea through a force main.

The eastern portion of the zone is a system which has a series of local collection subsystems, each with its own pump station and force main discharging to the next station westward. These subsystems were designed to be separate, but parallel, sanitary and stormwater conduits. When the twin conduits reach each pump station, separate force mains convey storm flow to the sea. All wastewater was originally planned to flow to the No. 11 East pump station, which pumps to the East Treatment Plant via a 48-inch force main. Currently, all of the system is operating in a surcharged condition, which causes mixed storm and sanitary sewage to be discharged both to the sea and the East Treatment Plant.

Existing System Basic Design Criteria

Criteria reportedly used in the design of the existing system were obtained through discussions with A/GOSD personnel. Main sewers and collectors were designed to provide slopes that will ensure self-cleaning velocities when the pipes are flowing full. The CCC Egypt Inception Report (June 1980) listed design criteria used by the General

Organization for Sewerage and Sanitary Drainage (GOSSD), which was responsible for the Alexandria sewer system prior to A/GOSD being formed. This criteria is summarized in Table H-1. Information as to how much of the existing system was designed using this criteria is not available.

Comparison of the criteria presented in Table H-1 with that discussed in Appendix A, indicates that flows computed by the GOSSD method are significantly larger than those projected in the 1978 Master Plan. This would result in sewers designed by GOSSD criteria having more capacity. However, when infiltration was calculated using the GOSSD equation, flows obtained were less than those given by Figure A-1, especially for shallow sewers, using the lower limit.

Other differences include GOSSD recommending generally smaller manhole spacing, smaller house connections, smaller minimum sizes for sewers, and different pipe and joint materials.

Combined sewers were designed in accordance with the design parameters established in a 1908 report prepared by Mr. Lloyd Davis, Chief Engineer of the Alexandria Municipality. Since then, Special Report No. 4 (1977) determined that for major sewers having a time of concentration in excess of about 15 minutes, the capacities of the existing sewers were far less than the three times dry weather flow, based on present design flows.

Condition of Existing System

Sewers. Some of the sewers serving Alexandria are in good condition. These can be incorporated into an expanded system and should provide many more years of useful service. There are, however, many older interceptors that were constructed at very flat slopes. This causes solids to settle and drastically increases maintenance requirements. In addition, the former practice of matching invert elevations at pipe size changes causes flooding at upper areas when the downstream system flows full.

Several sections of collapsed sewers urgently need repair. Some sewers have been or are currently being replaced under the TPP program, but more sections exist which need to be renovated. A/GOSD staff are currently preparing a list of additional collapsed sewers. A concentrated program should be initiated to start immediate repairs on these sewers.

In addition to older portions of the system which are surcharged, the newer eastern zone system (consisting of dual collectors for sanitary and storm sewage) also operates in a surcharged condition. This

surcharge is caused by pump station design and operation, and not by interceptor deficiency.

Table H-2 summarizes the deficiencies in the present system. It was compiled after analysis of the capacities and velocities of the existing sewers and is based on information obtained through discussion with A/GOSD staff.

Manholes. Information concerning design criteria used for existing manholes is very limited. Through discussions with A/GOSD staff and review of as-built drawings, WWCG determined that existing manholes in the older system are constructed as shown generally in Figure H-2. The barrel, 75 cm in diameter, is too small by today's standards, and the enlarged area at the sewer entrance and exit is too shallow for a man to stand upright. With modern cleaning equipment, this deficiency is not critical. Each manhole should be inspected to determine whether ground water is entering, and corrective measures should be taken to eliminate any leakage found.

In the eastern zone, where a dual pipe system of collection is employed, manhole locations often preclude access by modern sewer cleaning equipment. This should be corrected, as recommended by Special Report No. 4, August 1977.

Many of the sewer subsystems constructed in low-income housing areas have been constructed without the use of any design criteria. Some of these systems have square brick masonry manholes which extend up to 1 m above grade. Covers are variously cast iron, concrete, or missing. Uncovered manholes become receptacles for debris, which quickly plugs the sewers. Missing covers should be replaced immediately and secured to prevent unauthorized entry.

Maintenance. The responsibility for cleaning and maintaining the existing sewers in Alexandria is distributed among several districts, each of which is supervised by a superintendent. The maintenance crews are directed by a foreman and usually consist of three laborers.

Current methods used by A/GOSD to clean sewers are extremely labor intensive. A description of these methods was included in the Report on Sewer Maintenance Operation, January 1980. Some modern sewer cleaning equipment has been acquired by A/GOSD and more is being purchased under a TPP. With this additional equipment and the establishment of proper training programs for maintenance personnel, a comprehensive program can be implemented to ensure that the sewer system is properly cleaned and maintained.

Many of the current problems associated with maintaining the existing system are not related to lack of equipment or labor. Instead, they are

the direct result of dumping extraneous and incompatible wastes into the system. As long as wastes such as mazout residue, cattle manure, septage from septic tanks, solid wastes, gilla, oil, slaughterhouse wastes, and sand are permitted to enter the system, maintenance will be overburdened. Further aggravating this problem is the design and operation of pump stations, which results in system surcharge and causes solids to accumulate. These problems have caused some sewers to require daily or even constant maintenance, thus overburdening the entire system.

REVIEW OF THE 1978 MASTER PLAN

Previous sections of this appendix have described the existing Alexandria sewer system and its deficiencies. The following sections analyze TPP's and the recommendations of the 1978 Master Plan. The review emphasis is on correction of existing deficiencies and the determination of whether or not the 1978 recommendations will provide adequate remedy. Additional needs are identified and a remedial program of needed additional short-term improvements is recommended. Finally, costs for the recommended improvements are presented.

Review of Top Priority Projects

In August of 1977, Special Report No. 4 of the Alexandria Wastewater Facilities Development Program was published. This document identified the following top priority projects and recommended that these be undertaken at the earliest possible opportunity.

- Category I--Institutional and Operational Changes

  Item I-1 Implementation of the sewer use law
  I-2 Collection and disposal of wastes
  I-3 Sewer cleaning program
  I-4 Vehicle replacement
  I-5 Prohibition of building in Dekheila

- Category II--Repairs and Replacements in the Existing System

  Item II-1 Contract sewer cleaning and inspection
  II-2 Collector general relief sewer
  II-3,8,10,14 Western Harbor system
  II-4 Reconstruction of collapsed sewers
  II-5 Manhole cover replacement
  II-6 Smouha stormwater drain
  II-7 Drainage relief--Mohamed Ali Square
  II-9,13 Sporting pump station and force main
  II-11 Bisiry force main extension

II-12,16 Hospitals pump station and force main
II-15 Rehabilitation of the eastern zone pump stations

- Category III--New Service to Unsewered Areas

Item III-1 Ras El-Soda system
III-2 Siouf Keblia system
III-3,9,11,18,19 Mex-Dekheila system
III-4,12,20 Nouzha system
III-5,15,16,22,23 Abu Qir system

The listed TPP's are currently in various stages of development. On some, nothing has been done; on others, construction is in progress. None has been completed, although design reports have been written for almost all of the projects.

As for the deficiencies in the existing system which were identified previously, the TPP's coupled with the 1978 Master Plan recommendations will alleviate the majority of problems. Following is a list of the deficiencies, identified in Table H-2, that will be corrected by the TPP's:

Old Collector General. Flows to this old collector have mainly been diverted to the new collector general. Only a small amount of local drainage is now carried there. In addition, a new relief collector is being designed as a top priority project. Therefore, no additional improvements are recommended.

Chounah. This small collector is very old and in questionable condition. A/GOSD staff believe that the main problem stems from cow manure, which is constantly being dumped into the system via this collector. If such dumping could be stopped, the existing collector would be adequate. The problem can be solved through implementation of the TPP's concerning sewer use law and routine cleaning. Therefore, no additional improvements are recommended.

Mohamed Ali. A new collector to relieve the existing sewer is now being designed as a top priority project. Until construction is complete, routine cleaning will be very important for this collector. No additional improvements are recommended.

Long-Range Program

In general, the 1978 Master Plan recommended eliminating many existing pump stations by construction of deep gravity sewers. This would relieve the surcharged condition in many parts of the system.

Elimination of surcharging would, in turn, reduce solids build-up and allow the system to be operated as designed. Following is a list of the deficiencies identified in Table H-2 that will be corrected by the long-range projects recommended in the 1978 Master Plan and this update of that document.

Port Est Collector. This old collector was identified as having a very flat slope and low velocities. Regardless of the disposal alternative selected, a new collector designed at a steeper slope is recommended. Until selection of the disposal alternative, the existing Port Est collector should be cleaned routinely to assure maximum possible capacity.

Wekalet El-Khoder. This branch of the Port Est collector also has low velocities. A/GOSD staff has indicated, though, that this collector would improve if the Port Est collector were replaced. Therefore, we recommended that this collector be cleaned regularly, as a short-term improvement, and that it be reevaluated after a decision as to the fate of the Port Est collector is reached.

North and South Rue Marine. These two collectors branch off the upstream end of the Wekalet El-Khoder collector and have similar deficiencies. The same recommendations as for Wekalet El-Khoder apply to these two collectors.

Slaughterhouse--Tannery. This collector has flat slopes and is in poor condition. Since flows from the area served are anticipated to be pumped west, we do not recommend that any short-term improvement other than cleaning be initiated.

Road No. 218. This collector will not be used after the West Treatment Plant is operational or if the Master Plan recommendations are implemented. Therefore, no short-term improvements other than cleaning are recommended.

Furn El-Gueraya Connection. This sewer will also be abandoned after the Master Plan is implemented. No improvements are recommended other than cleaning.

Hadara. This large collector was constructed in ground water at a flat grade and is now in very poor condition. If the Master Plan is implemented, a new collector could be constructed at a steeper grade. This is considered a long-term improvement and not recommended at this time. A short-term improvement would be to clean the collector on a routine basis.

Bacos North. This collector flows to the Glym pump station, and is flat, has low velocities, and is subject to flooding. A/GOSD staff have

indicated that flooding is caused by insufficient pump station capacity. Many of the problems will be corrected when the Master Plan is implemented and the pump station is abandoned. Until that time, routine cleaning is recommended as a short-term improvement.

Sarwat Pasha. This collector has problems similar to Bacos North. It would be replaced or at least improved when the new Corniche collector is constructed, as proposed by the Master Plan.

San Stefano. This collector is surcharged due to the inadequate capacity of the Sarwat pump station. It should be improved by the proposed Corniche collector, and therefore, needs no short-term improvements other than routine maintenance.

Glym Discharge Collector. This collector does not have sufficient capacity to accept the influent from the Glym pump station and therefore is surcharged. With the elimination of the pump station, it will have sufficient capacity for the design period. Therefore, no short-term improvements are recommended.

Abu Soliman Collector. This major collector currently operates in a surcharged condition and is constructed at too flat a slope. In addition, it was constructed with perforated pipe to carry ground water. It is proposed that a new 1400 mm gravity line be constructed as soon as possible to relieve the existing collector. This new line would not only relieve the existing collector, but would also allow it to be abandoned when the Master Plan recommendations are implemented, no matter which disposal alternative is chosen. If the sea disposal alternative is used, most of the wastewater which currently flows through the Abu Soliman collector would be redirected to the Ras El-Soda Treatment Plant (see Figure M-4 in Appendix M) and the new relief collector would easily carry the remaining flows. If land application is chosen, the force main from the new No. 3 East pump station could continue all the way to the Abu Soliman pump station and the relief collector would carry the remaining flows. Therefore, we recommend that this relief collector be constructed now to eliminate one of the major deficiencies in the existing system.

EVALUATION AND RECOMMENDED SHORT-TERM IMPROVEMENTS

Of the deficiencies listed in Tables M-2, only those at the Ibrahimia and Gabbary collectors will not be remedied by the completion of the top priority projects or the Master Plan recommendations. Following are recommendations for these two collectors.

The Ibrahimia collector is old and its first 400 m are constructed with inadequate slope, resulting in low velocities. The main portion of flow

is currently diverted upstream for this section, resulting in a need to convey only local drainage flows. Therefore, we recommended that only regular cleaning be provided as a short-term improvement.

Even though the slopes on the Gabbary collector are less than recommended, calculated velocities are greater than 0.6 m/sec. A/GOSD staff concur that velocities are sufficient for this collector. Therefore, no short-term improvements other than cleaning are recommended.

As discussed previously, a sewer cleaning program is necessary to keep the existing collectors free of sediment and at maximum capacity. A report, Recommended Sewer Cleaning Programs for 1980 (February 1980), outlined the steps necessary to a good cleaning operation. The first step listed in the report is to establish a priority area. The main collectors listed above should be given first priority. All subareas associated with the main collectors should also be cleaned before moving on to the next system.

Another TPP that needs to be implemented is the collection and disposal of wastes (Item I-2). Along with it, implementation of the sewer use law (Item I-1) should also be accomplished. These two projects will greatly help keep inappropriate liquid and solid wastes from reaching the sewers, where they clog the pipes and reduce the capacity of the system. Corrosive liquids attack the sewer surfaces and the metal of equipment used in the system, causing corrosion and structural failure. Elimination of these wastes is essential to producing a working sewerage system. Many reports have been written describing procedures and facilities which will help accomplish these tasks. Those recommendations should be implemented as soon as possible. In addition, all cattle should be moved from the city areas to a rural location.

Item II-5 of the top priority projects recommends replacing manholes that are inaccessible to modern sewer cleaning equipment. More than 300 manholes would need to be modified to move the manhole openings over both of the dual sewers in the eastern zone. Also, 29 new manholes are required along one of the twin pipes of the Mohamed Ali collector.

COSTS OF RECOMMENDED SHORT-TERM IMPROVEMENTS

Costs of the proposed projects were given in Special Report No. 4 based on 1977 costs. Table [illegible]-3 presents cost estimates for these projects based on end-of-1980 costs. These projects should be completed as soon as possible to avoid additional costs in the future due to inflation, and to minimize future maintenance costs and problems.

Table H-1
GOSSD DESIGN CRITERIA

Domestic Flow

High	300 lcd
Medium	200-250 lcd
Government	150 lcd
Hospitals	1,000 l/d/bed[a]
Hotels	500 l/d/bed[a]

Infiltration

$Q(m^3/d \text{ per } ln\text{-}km) = (5 \text{ to } 10)h^{2/3}$
where h is the depth of pipe invert below ground water

Gravity Collection Systems

Minimum diameter--180 mm

Design depth of flow:

Severs--1/2 full for maximum discharge
Collectors--2/3 full for maximum discharge

Minimum velocities:

Severs--0.75 to 1.5 m/sec at maximum flow
0.5 m/sec at minimum flow
Collectors--1.5 m/sec at maximum flow
0.6 m/sec at minimum flow

Manhole spacing:

Sewer Diameter	Maximum Spacing
180 - 225 mm	30 m
300 - 450 mm	45 m
600 - 1,000 mm	60 m
Main Collectors	80 m

House connections:

Type--into manholes
Minimum Diameter--125 mm

Pipe materials:

≤ 1,000 mm in diameter--vitrified clay pipe (VCP)
> 1,000 mm in diameter--Cast in place
Joint materials--hemp and mortar with encasement of pipe

Source: CEU Egypt Inception Report, June 1980

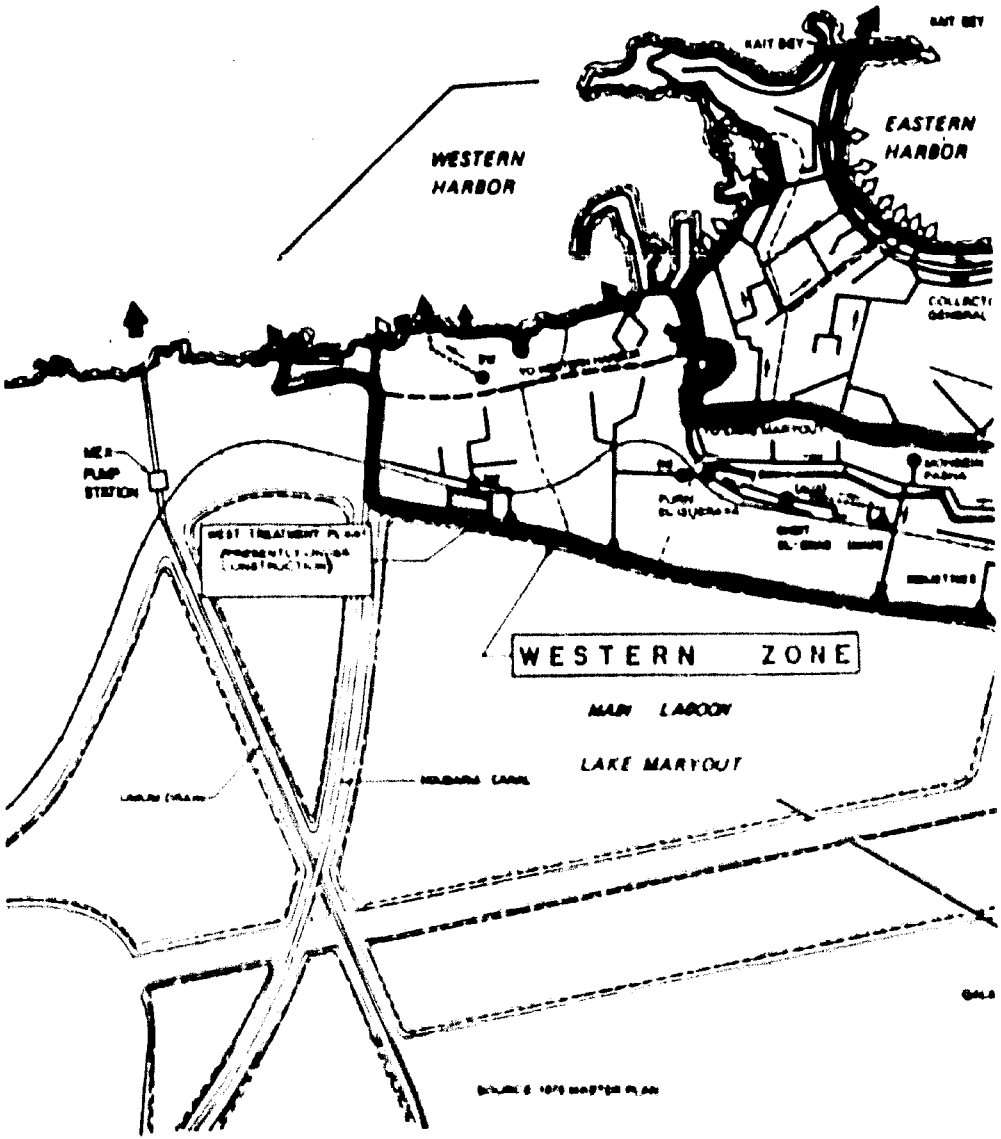
[a]Litre per day per bed

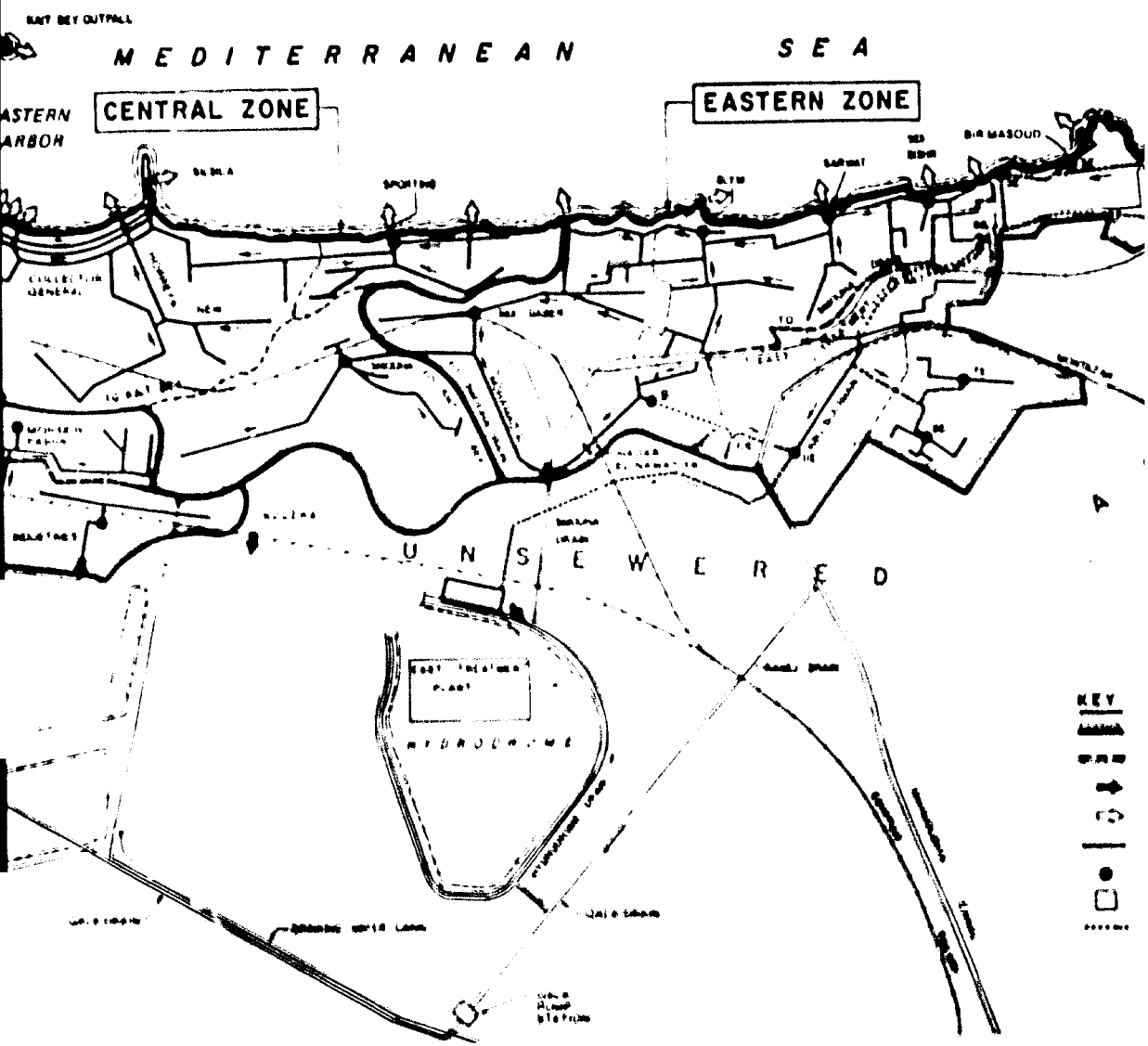
Table H-2
EXISTING COLLECTION SYSTEM DEFICIENCIES

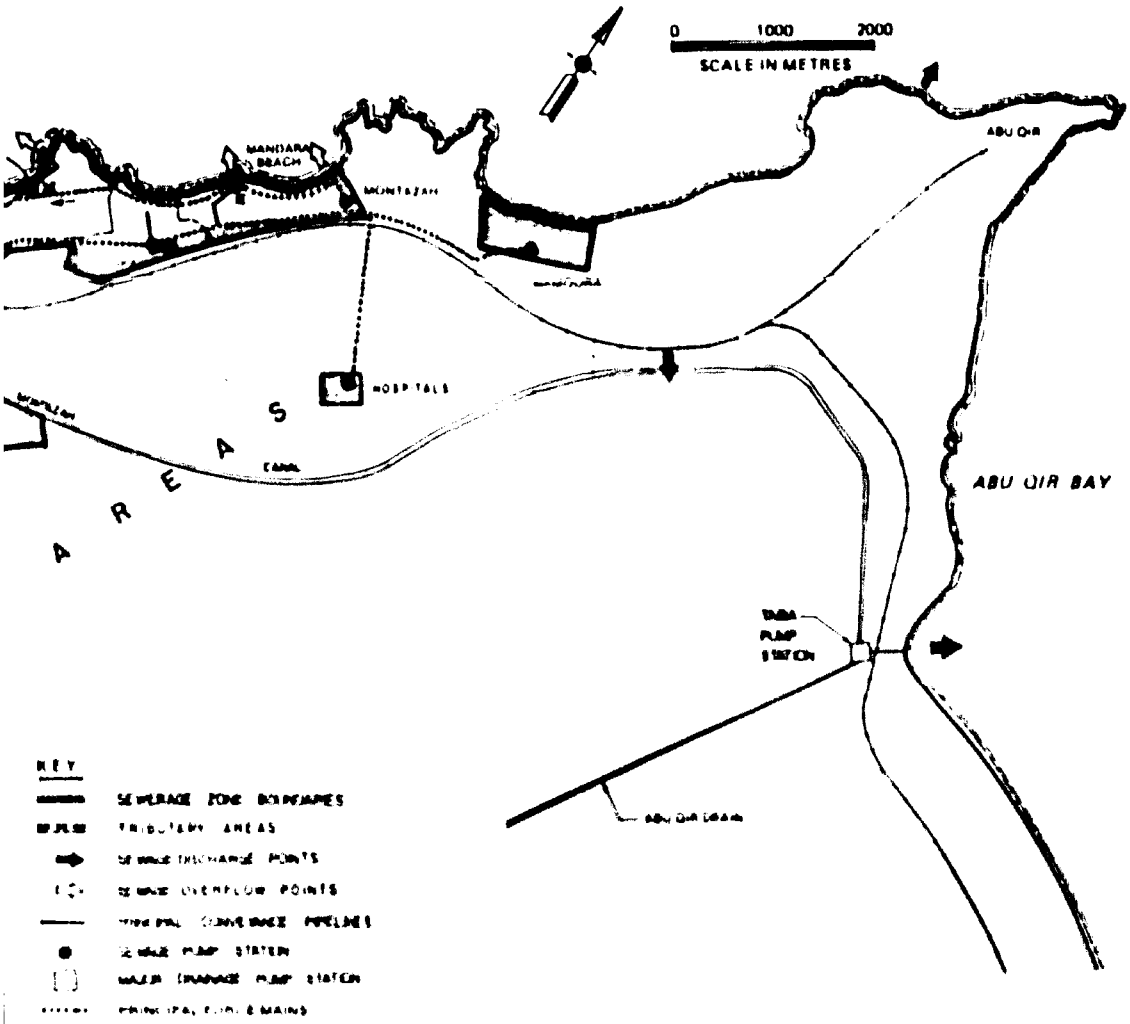
Collectors	Deficiencies
Port Est Collector	Flat, low velocities, old
Old Collector General	Flat, low velocities, old, too small
Wakalat El-Khoder	Flat, low velocities
North & South Rue Marina	Both flat, low velocities
Ibrahimia	Old, flat, low velocities
Slaughterhouse-Tannery	Poor condition, somewhat flat
Gabbary	Old, flat
Road No. 218	Flat, low velocity
Furn El-Guerays Connection	Flat, low velocity
Chouna	Old, bad condition, cow manure problems
Mohamed Ali	Flat, low velocities, perforated pipe
Hadara	Flat, in poor condition, built in ground water
Bacos North	Flat, low velocities, P.S. can't take flow
Sarwat Pasha	Flat, low velocities
Abu Soliman	Flat, surcharged
San Stefano	Surcharged
Glym Discharge Collector	Surcharged

Table H-3
CAPITAL COSTS OF SHORT-TERM IMPROVEMENTS
(1000 LE, end-of-1980 prices)

Project	Estimated Cost
Sewer Cleaning Program	600
Collection and Disposal of Wastes	2,000
Sewer Law Implementation	300
Manhole Replacement	250
Abu Soliman Relief Sewer	1,800


WESTERN HARBOR
EASTERN HARBOR
WESTERN ZONE
MAIN LAGOON
LAKE MARYOUT


MEDITERRANEAN SEA
CENTRAL ZONE
EASTERN ZONE
ASTERN
ARBOR
BIR MASOUD
UNSEWERED
KEY



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE H-1

PLAN OF EXISTING MAJOR INTERCEPTORS AND FORCE MAINS

A. DOBED WWCS W.E.C.S

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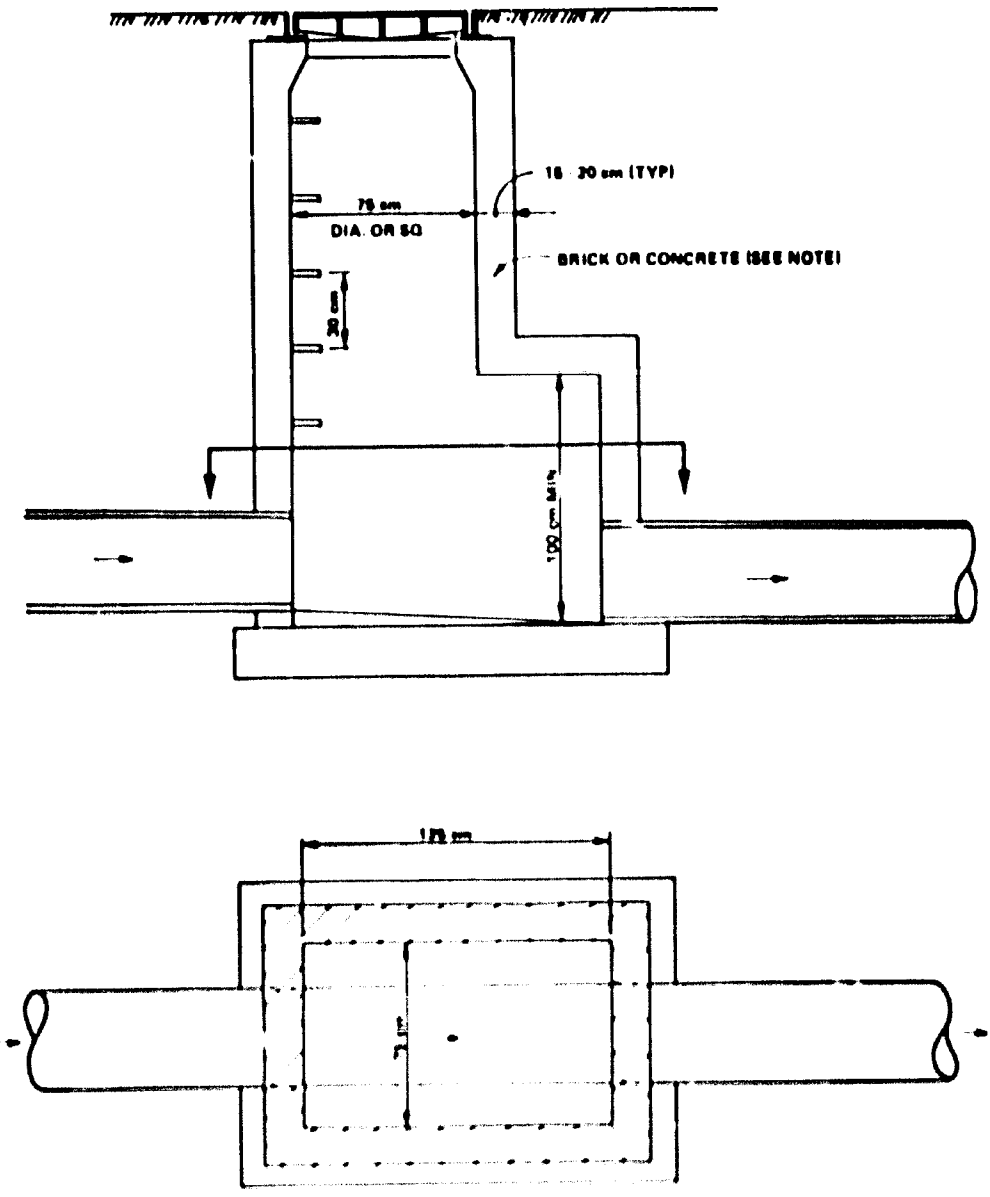

NOTE

OLDER MANHOLES MADE OF BRICK FOR SEWERS < Φ30cm CONCRETE FOR SEWERS >Φ30cm. LATER MANHOLES ALL MADE OF CONCRETE

(SEALE RANCH) A WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE H-2

TYPICAL EXISTING MANHOLE
(OLD SYSTEM)

A (2/2) BOLTON & ECO

APPENDIX I

ENVIRONMENTAL DATA BASE REVIEW

INTRODUCTION

Purpose

The purpose of this appendix is to review and update baseline environmental data. Information presented in this appendix to meet this objective includes:

- o Descriptions of areas [illegible]d by the alternatives under consideration
- o A summary of areas likely to be affected in the future if no action is taken on wastewater collection, treatment, and disposal
- o Additional studies needed

Baseline material is necessary to evaluate the environmental impacts associated with wastewater collection, treatment, and disposal alternatives considered by WWCG during the review and update of the 1978 Master Plan. The scope of environmental work required for the review and update of the Master Plan is limited. Specific tasks to be performed include:

- o Outline environmental impacts associated with sites considered for conveyance, treatment, and effluent or sludge disposal.
- o Investigate potential for irrigation of other potential lands not considered in the Master Plan.
- o Review existing public health aspects.

According to recent USAID environmental regulations (Federal Register, Vol. 45, No. 207, October 23, 1980) a detailed IAS or EIS would normally be required for a larger-scale sewerage project. However, the scope of work for this review and update was developed prior to the publication of the USAID regulations. Consequently, a detailed environmental review cannot be developed at this time. However, the spirit and forest of the regulations will be followed when possible. For example, this appendix presents descriptions of the areas affected by the alternatives in sufficient detail to allow understanding of the impacts caused by the alternatives.

Discussions of the beneficial and adverse environmental impacts are presented in the specific appendices dealing with conveyance, treatment, or disposal alternatives.

Approach

The approach utilized for updating environmental baseline material consisted of:

- o Review of the 1978 Master Plan and associated documents
- o Brief field visits to affected areas
- o Consultation with local experts to solicit comments on the Master Plan and environmental concerns

Background

The 1978 Master Plan initially recommended wastewater effluent disposal to the Mediterranean Sea via two long outfalls following preliminary treatment. Significant modifications were made to the Master Plan in response to the USAID Final Environmental Impact Statement issued in August 1979. USAID modified the plan to upgrade wastewater treatment from preliminary to primary. The recommended Kait Bey outfall length was decreased, and the proposed diffuser lengths were increased to maximize the dispersion of settleable solids.

Documents Reviewed

The principal documents reviewed to summarize and update the environmental baseline information included:

- o Environmental Impact Statement
  - Vol. I - Final Statement
  - Vol. II - Final Technical Report
  - Vol. III - Final EIS Comments and Responses

- o Alexandria Wastewater Master Plan Study
  - Vol. I - Summary Report
  - Vol. II - Technical Report
  - Vol. III - Appendices
  - Vol. IV - Marine Studies

- Feasibility Study of West (Nubarya) Extension Reclamation and Settlement Project--Final Report, Vol. I, Annexes I and II.

- Classification of Water Bodies, Sewage Treatment Requirements and Sewage Collection Requirements (MISR Laws).

References cited in this appendix are referred to by number, (e.g., Ref. 1), and complete citations are given at the end of the text.

Agencies/Individuals Contacted

USAID officials who were contacted to discuss environmental concerns for the Master Plan Review and Update included:

- Mr. Stephen F. Lintner, USAID, Environmental Coordinator, Bureau for the Near East, Washington D. C.

- Mr. Walter McAleer, USAID Mission Office, Cairo, Egypt

Egyptian special consultants who provided input and review comments for this appendix included:

- Dr. Abdel Hakeem M. Gomah, Soil Sciences
- Dr. H. M. Mitwally, Sanitary Engineering
- Dr. S. G. Saad, Sanitary Chemistry
- Dr. K. E. Hakim, Sanitary Chemistry
- Dr. M. H. Wahdan, Epidemiology
- Dr. Ayda El-Lekani, Bacteriology
- Dr. Nahed Kamel, Public Health
- Mr. Nadia Ibrahim, Tourism

Format

The remaining portions of this appendix are presented in three sections:

- Overview of the Study Area
- Future Environment Without a Project
- Additional Studies Needed

The overview of the study area is a concise summary and update of baseline environmental data presented in the Environmental Impact Statement.

Known inconsistencies or omissions of the 1978 Master Plan and subsequent Environmental Impact Statement are discussed. Generalized environmental data are also presented on areas considered for this study but not in the 1978 Master Plan. The section on the future environment without a project is based on the "no action" alternative presented in the Environmental Impact Statement. The final section entitled "Additional Studies Needed" presents environmental data collections and analyses that will be required prior to final design.

OVERVIEW OF THE STUDY AREA

Alexandria is located on the eastern Mediterranean coast at the northwestern fringe of the Nile River Delta (Figure I-1). Areas to the south and west consist of a mixture of reclaimed agricultural lands and undeveloped desert. Cairo, the largest city in Egypt, is located about 200 km to the southeast. Prominent features of the Alexandria area are the Mediterranean Sea, the lower Nile Delta, Lake Maryout, and the western desert. The east-west development pattern of Alexandria has resulted from the parallel orientation of nearby limestone ridges and Lake Maryout, formerly a shallow marine lagoon.

The study area for the Master Plan Review is specifically confined to the more central and urban areas of Alexandria. However, remote areas within 90 km to the south and west were also considered for the land application of wastewater. Surrounding environs and the central urban study area are shown in Figure I-2.

Environmental Concerns

Environmental concerns that influence wastewater planning for the study area and surrounding environs are listed below in their approximate order of importance:

- Public health
- Assimilative capacity of potential surface water disposal sites
- Public acceptability of land application of wastewater
- Disruption to or relocation of existing land uses due to siting and construction of wastewater facilities.
- Preemption of alternative development due to wastewater facilities planning

- Disruption to potentially significant historic/archaeologic resources during construction

Summarized below are pertinent aspects of the natural and social systems of the study area that influence these environmental concerns. This summary is based primarily on the existing conditions chapter of the Environmental Impact Statement and updated as necessary to reflect new information and to include the remote land application areas not considered in the 1978 Master Plan.

Natural Systems

Important features of the natural systems of the region that influence or will be affected by wastewater facilities planning are discussed below.

Climatic Data Summary. Based on rainfall and temperature characteristics, the climate of the Alexandria region is classified as arid Mediterranean (Ref. 1). The average annual precipitation in Alexandria is approximately 200 mm and occurs mainly during winter storms from November through February (Table I-1). Annual precipitation is on the order of 100 mm, 90 km to the south. Annual evaporation rates are approximately 1900 mm in Alexandria. As shown in Table I-2, the prevailing winds average 4 m/s and originate primarily from the northwest. Winds from the southwest are common in December and January. Air quality in the Alexandria region is apparently worse than in some of the more heavily industrialized regions in the United States.

Geology/Soils/Ground Water Resources. The arid Alexandria region is characterized by sedimentary deposits due to fluvial transport of the Nile and repeated marine incursions. In fact, an ancient branch of the Nile once extended to the west of Lake Maryout. Physiographically, the study area can be divided into three regions:

- Lake Idku-Alexandria-Lake Maryout coastal zone
- Modern Nile Delta lands
- Western desert west of the Noubaria Canal and Desert Road

The coastal zone, 10 km wide, contains three to five prominent limestone ridges formed from receding shorelines of the Mediterranean. Shallow depressions between the ridges are occupied by loamy deposits, silts, and clays. The modern Nile Delta lands are located southeast of the coastal zone and northeast of the Noubaria Canal. This area is characterized by silts overlying outwash sands and gravels. The western

desert area is generally located west of the Noubaria Canal and Desert Road and south of the coastal zone. The eastern fringe of this area near the Noubaria Canal can be characterized as ancient Nile flood plain. The remaining areas of the western desert, including the potential land application site, are characterized by accumulations of windblown sands over sands and gravel and rockland.

Ground water in the coastal zones is of marine origin. Some of the ground water in inland areas is brackish because much of the bedrock and subsoil material is permeated with soluble marine salts and gypsum.

Surface Water Resources. Surface water resources of the Alexandria region consist of the Mediterranean Sea, brackish inland lakes, and an extensive system of canals and drains. Characteristics of the Mediterranean are discussed in Appendix L, Sea Disposal and Outfall Review, and briefly summarized below. Alexandria lies on the eastern basin of the Mediterranean. High salinities of this inland sea (approximately 39 parts per thousand, ppt) are due to a combination of high evaporation rates, low freshwater recharge, and limited mixing with other oceanic water masses. Currents of the Mediterranean in the Alexandria region were reported in the 1978 Master Plan to trend northeast parallel to the coast. Recent surface drifter studies (Ref. 2) indicate differences in offshore and nearshore current patterns. Offshore currents, approximately 10 to 25 km off Alexandria, generally move in a northeasterly direction. However, a reversal to the southwest was noted in the offshore component in May 1977. Coastal water masses within 6 km of Alexandria move in a southwesterly direction throughout most of the year. A strong shoreward component was noted in September 1977. These recent studies indicate a net northeasterly drift of offshore water masses and a net southwesterly movement of nearshore waters. Nearshore waters of the Alexandria region are impacted by municipal wastes, industrial discharges, and large volumes of effluent from drains that convey a mixture of agricultural runoff and municipal/industrial wastes. These discharges have resulted in the formation of a fresh water lens along the coast between Dekheila Bay and Abu Qir. Water quality problems noted in this freshwater lens include:

- Elevated concentrations of indicator bacteria
- Depressions of dissolved oxygen
- Elevated concentrations of hydrogen sulfide
- Elevated concentrations of suspended solids, floatables, debris, and waste oil

The two shallow brackish water lakes in the Alexandria region include Lake Maryout and Idku. Lake Maryout approximates the southern boundary

of the Alexandria urban area and consists of a series of discrete basins. The lake was essentially dry during the eighteenth century because the Canopic branch of the Nile River had silted up. The area was reflooded in the early 1800's when the dikes separating the lake bed and the sea were breached during the Napoleonic war. Agricultural irrigation systems were developed for the southern province of Beheira in the late 1800's and the lake used as a drainage catch basin. Since the lake was below sea level, the Mex pump station was constructed to convey drainage waters to the sea. Approximately 7000 Ml/day are presently pumped from the main basin into Dekheila Bay by the Mex pump station. Reclamation has reduced the total lake area from 20,000 ha in 1900 to approximately 8000 ha at present. The lake exhibits salinities from 3 to 13 ppt and depths do not generally exceed 1 m. Water quality problems are evident in the main basin due to drainage and wastewater discharges. The lake is nutrient-enriched, and extremely low values of dissolved oxygen exist in areas of the main basin that receive drainage and wastewater. Hydrogen sulfide concentrations are extremely high, ranging from 2 to 20 mg/l. Transparency is low and secchi disc readings are generally on the order of 40 cm. The east basin, or fishery lagoon, exhibits similar water quality characteristics except that dissolved oxygen readings are higher. No other water quality data was available in the 1978 Master Plan and Environmental Impact Statement for the remaining basins of Lake Maryout. Lake Idku encompasses an area of approximately 12,600 ha and averages 1 m in depth. It receives drainage water from Idku and Berkik drains but remains freely connected to coastal waters of Abu Qir Bay. Salinities range between 5 and 12 ppt. Nutrient concentrations in Lake Idku are lower than reported for Lake Maryout. No other water quality data on Lake Idku was presented in the 1978 Master Plan.

Aquatic Ecology. Waters of the eastern Mediterranean are generally considered by most authorities to be nutrient-poor, with phosphorus the limiting nutrient. (The 1978 Master Plan concluded that nitrogen was limiting. See Appendix L for a detailed discussion.) Construction of the Aswan High Dam and subsequent reductions of nutrient inputs to the Mediterranean have resulted in a general decline in the fisheries of the eastern Mediterranean. In addition, the fisheries of the Alexandria region have also been impacted by overfishing and pollution. Industrial pollution problems are evident in the Mex Dekheila and Abu Qir regions. A variety of industrial wastes are discharged to Abu Qir Bay via the Tabia pump station. This effluent plume moves easterly along the Abu Qir Bay shoreline and may entrain the tidal exchange from Lake Idku. These wastes have been implicated in the decline of the penaeid shrimp fishery of Abu Qir Bay.

The advanced state of eutrophication of shallow Lake Maryout (particularly the main basin) is evidenced by the increased growth of

aquatic plants, reed grass, and occasional floating mats of blue-green algae. In addition, the commercial landings of Tilapia fisheries have apparently declined, and most of the catches appear to be restricted to the southeast fishery lagoon and Noubaria Canal. Lake Idku differs since it is connected to Abu Qir Bay and it appears to be less severely impacted by nutrient enrichment. However, extensive growths of aquatic weeds and reed grass are evident. Tilapia spp. are the most common fisheries of Lake Idku. Mullet and penaeid shrimp are also present due to the connection with the bay. Fisheries landings for both Lake Maryout and Idku cannot be compared since the figures reported in the 1978 Master Plan and Environment Impact Statement appear unreliable.

Terrestrial Ecology. The study area environs consist of urban development, brackish water lakes and marshes, irrigated agricultural lands, and desert. Domesticated work animals and introduced plant species characterize the urban and agricultural areas. Brackish water lakes and marshes serve as feeding and resting areas for resident and migrating waterfowl. Although remote, the desert areas within 100 km from Alexandria are not entirely pristine. Increased population growth in outlying districts has increased the destruction of native vegetation and wildlife. Gathering of woody bushes for fuel and overgrazing by goats and sheep has altered the native landscape. Areas that once supported extensive steppe-like vegetation may be sparsely vegetated. Windblown sands and even calcareous soils that overlay areas with good water holding capacity can support moderately dense stands of perennial woody bushes such as Thymalea and Salicornia. Animals that are present in this desert environment include fox and various rodents as well as numerous soil- and plant-associated invertebrates. Certain outlying desert areas could support populations of the slender-horned gazelle (Gazella leptoceros). This endangered species occurs in Egypt and is included on the U. S. Fish and Wildlife Service worldwide list of endangered and threatened plants and wildlife (Federal Register, Vol. 45, No. 99, Tuesday May 20, 1980, "Republication of the Lists of Endangered and Threatened Species"). The Tora hartebeest (Alcelaphus buselaphus tora) is another endangered species found in Egypt. This species is restricted in distribution to the southern portion of the country near Sudan.

Social Systems

The various aspects of social systems that influence or are affected by wastewater collection, treatment, and disposal are summarized below. Particular emphasis is given to public health.

Public Health. Public health problems in arid regions are accentuated because consumptive uses of limited water supplies (e.g., for irriga-

tion, drinking, washing etc.), conflict with simultaneous uses for wastewater disposal. Thus, problems associated with the transmission of waterborne infectious diseases can become severe in densely populated arid regions. In addition, the intensive reclamation of undeveloped areas for perennial irrigation can create health problems that may impact the benefits associated with increased food production and improved nutrition. Irrigation systems can serve as the transfer medium and habitat for parasitic disease vectors and intermediate hosts. Discussed below are some of the conditions that contribute to inadequate public health conditions in Alexandria and surrounding areas. These include:

- Overall poor sanitary conditions
- Problems associated with the transmission of water-borne infectious diseases
- Problems associated with parasitic diseases in reclaimed irrigated agricultural areas

As discussed in the 1978 Master Plan and Environmental Impact Statement, a variety of inadequate sanitary provisions for water supply, waste disposal, and food processing and preservation contribute to overall public health problems in urban Alexandria. Some of the most notable deficiencies are listed below:

- Absence of sewers or adequate on-lot disposal systems in approximately 20 percent of the densely populated regions of Alexandria
- Occasional leakage of sewage from existing collector sewers or force mains into drinking water canals upstream from water treatment plants and distribution systems
- A largely uninformed populace that continues to use sewage-contaminated surface waters for washing, cooking, and in some cases for drinking
- Failures and collapse of some existing sewers
- Periodic discharges of raw wastewater to bathing beaches
- Topography and drainage characteristics in some areas of the city that enhance the formation of sewage-filled ditches and puddles
- The presence of large numbers of work and farm animals in residential and commercial areas

- Inadequate solid waste and refuse collection and disposal
- General lack of refrigeration for perishable foods

Improper waste disposal practices and contamination of limited surface water supplies certainly contribute to the health problems of urban residents. Contaminated water supplies and food account for the prevalence of gastrointestinal ailments among the populace. It is estimated that diarrhetic ailments account for 50 percent of the outpatient care in general public hospitals and private clinics (Ref. 3). Other health problems that are prevalent in Alexandria, particularly among children, include upper respiratory infections such as bronchitis and pneumonia. Remaining disease problems include measles, mumps, chicken pox, tetanus, poliomyelitis, infectious hepatitis, cholera, typhoid, and dysentery.

Direct transmission of poliomyelitis, infectious hepatitis, cholera, typhoid, and dysentery can result from contact and ingestion of contaminated water supplies (Ref. 4). Data on the incidence of some of these waterborne diseases in Alexandria from 1969 through 1979 are presented in Table I-3. These data must be viewed with extreme caution. The values shown in Table I-3 underestimate the actual incidence rates, since they are based on the number of cases admitted to the one communicable disease hospital located in the Alexandria governorate. Thus, cases treated by private physicians and at private clinics go unreported. In addition, incidence rates cannot be compared between governorates since each has a slightly different reporting system and quality of care at the hospital. These factors were not sufficiently emphasized in the 1978 Master Plan and Environmental Impact Statement. Consequently, statements that disease incidence rates in Alexandria exceed those in Cairo cannot be substantiated.

The public health impacts of the polluted beaches along Alexandria are also of concern. A survey of the relationship between bathing water quality and ill health effects was conducted in Alexandria between 1976 and 1978 (Ref. 5).

The first year of the study indicated a greater difference in the incidence of gastrointestinal problems (e.g., vomiting, diarrhea) between swimmers and nonswimmers at the more heavily polluted beaches. Symptoms seemed to plateau at E. coli and enterococcus densities of 200-300/100 ml. The study was extended in 1977 and 1978 to include both Alexandria residents and summer vacationers from Cairo. In addition, the followup period was extended approximately 1 month to monitor the potential development of infectious hepatitis and typhoid fever in swimmer and nonswimmer control groups. Cairo visitors showed a much higher incidence of swimming-associated gastrointestinal ailments than the Alexandria residents. Rates of swimming-associated gastrointestinal

ailments for Cairo visitors were 2.5 to 15 per 1000 person-days. Children generally exhibited the higher rates. Rates for Alexandria residents were <1 to 6 per 1000 person-days. Several cases of typhoid fever were found among swimmers at the Sporting Beach, one of the more heavily polluted beaches of Alexandria, which is adjacent to one of the pump station outfalls. Infectious hepatitis was rarely found, and the authors suggested that both residents and vacationers from Cairo have previously been exposed and may exhibit immunity to the hepatitis A virus.

Public health problems in the outlying agricultural regions may be more severe than previously described for the urban area, since access to health care facilities is much poorer. In addition, parasitic disease problems are much more evident in the remote irrigated agricultural areas. As shown in Table I-4, a variety of parasites affect populations in irrigated agricultural areas. This work was based on a 1978 study of a random sample of individuals in Abis II, an agricultural village area about 10 km from central Alexandria. It indicates a high incidence of amoebic infections and schistosomiasis. These parasitic diseases can become endemic in areas where human wastes contaminate surface water supplies and where effective controls of intermediate host propagation in irrigation systems are not instituted. Other hygienic factors which contribute to the presence of these parasitic diseases include:

- Eating unwashed vegetables
- Living in close association with work animals
- Not wearing footwear

Population and Land Use Considerations. The current population of the Alexandria governorate is estimated at approximately 2,537,000 and is expected to reach 4,660,000 by the year 2000. Approximately 20 percent of existing residents are not served by sewers. Approximately 400,000 seasonal residents move to Alexandria in the summer months, and this number is expected to increase to approximately 670,000 by the year 2000.

Land uses of the study area environs consist of urban development, industrial areas, irrigated agricultural lands, and desert. The urban areas of the central and eastern zones of the city are characterized by mixed residential housing interspersed with small markets and businesses. Heavy industrial development is generally confined to two corridors. One extends from Mex-Dekheila westward along the Mahmoudiya Canal to just west of the Hydrodrome. The other extends from the Tabia pump station and east along the shoreline of Abu Qir Bay to Lake Idku. Coastal beach and recreation areas are found along the coastal zone from Agami

to Mamoura. Irrigated agricultural lands are located south of the city along the Noubaria, El Nasser, Maryout, and Bahig Canals. Desert lies south and west of the irrigated lands. A more detailed discussion of population and land use aspects is presented in Appendix C, Planning Considerations.

Economics and Government Structure. These are closely intertied in Egypt. It is therefore appropriate to discuss both economic and government functions in the same context. Alexandria is one of the major components of the Egyptian economy. It is the country's primary maritime port, its second largest city, and is a major Egyptian resort area because of its beaches. The industrial and commercial sectors account for most of the employment. Some of the larger nationalized industries located in and near Alexandria include petroleum refineries, textile and cotton mills, food processing plants, and metal working facilities. Major businesses include manufacturing, transportation, communications, and construction. Tourism is also very important to the City of Alexandria. Areas surrounding Alexandria generally depend on agriculture and related services and markets. However, several industrial developments are located south of Alexandria along the Desert Road. As shown in Figure I-2, the Alexandria governorate boundaries include the city, the Agami beaches, Abu Qir, and an extensive area to the south containing irrigated lands and undeveloped desert. The Alexandria governorate structure serves both as an arm of the national government and as the administrator of necessary services for the city and outlying areas. The Alexandria General Organization for Sanitary Drainage (A/GOSD) is the primary organization for wastewater facilities in Alexandria.

Social/Religious Aspects. Approximately 90 percent of Egyptians are of the Islamic faith and follow the teachings of the Koran. The traditional Islamic concept of pollution forms some of the basis for Egyptian laws, which prohibit the discharge of sanitary wastes to the Nile River or its branches. Less than 10 percent of Egyptians follow the Christian Coptic faith.

Historic/Cultural Resources. The city of Alexandria has a rich and varied history and contains many sites of historical importance. It was founded in the third century B.C. by Alexander the Great. This important seaport of the eastern Mediterranean has been dominated by a varied sequence of cultures and empires that have influenced its development. These include: Phoenician mariners, Pharaonic garrisons, Greek and Roman invasions, Ptolemaic dynasties, Christian influences, the Ottoman Empire, and Islamic rule. Modern influences have included the French and British. Alexandria was once famed for the Pharos Lighthouse, one of the seven wonders of the ancient world. This structure was demolished but stood at the present site of the Kait Bey Fort, on the

Eastern Harbor. Other prominent sites of historical significance are listed in Table I-5. Most of these are confined to the area surrounded by the ancient Greco-Roman walls which enclosed the old city south of the Eastern Harbor. Based on its rich history it may be likely that additional sites of significance could be unearthed in the future.

The desert environment surrounding Alexandria has a less rich and varied history. The thin coastal belt to the west of Alexandria was once an important agricultural area in the Greco-Roman era. Dryland farming in this area was facilitated by the construction of earthen mounds, or karms. These karms collected and concentrated limited runoff for the cultivation of grapes, olives, figs, and barley (Ref. 6). These structures still dot the landscape west of Alexandria. Romans excavated numerous cisterns to hold rainwater in the limestone ridges that parallel the coast west of Alexandria. Numerous tombs of unknown significance are indicated on the 1949 topographic maps (1:100,000) of areas south and west of Alexandria. Old Coptic, or Egyptian Christian, monasteries are located at remote sites such as Wadi Natroun and in the Abu Mena Basin south of the Bahig Canal.

FUTURE ENVIRONMENT WITHOUT A PROJECT

This section presents an updated summary of the descriptions of the future environmental setting in the year 2000 without substantial improvements to wastewater facilities. This section represents a summary of the "no-action" alternative as presented in the Environmental Impact Statement for the 1978 Master Plan. The impacts of no action on natural and social system features of the study area environs cannot be presented without first describing the assumptions about planned projects expected to be ongoing or in place by the year 2000.

Assumptions

Population, land uses, and economic development of the Alexandria region are expected to change dramatically by the year 2000, regardless of any improvements that might be made to the existing wastewater facilities. Current projections indicate that the population will increase approximately 84 percent by the year 2000. A variety of residential, commercial, and industrial developments are expected to accommodate many of the future demands for housing and employment. The major projects anticipated for the year 2000 are summarized in Table I-6. The most significant projects consist of the planned developments at Amria and Sadat City. Each is expected to support a permanent population of 500,000 by the year 2000. Total wastewater flows in the Alexandria region are expected to increase from 480 Ml/day (sewered) in 1980 to about 1476 Ml/day in the year 2000.

Natural Systems

Features of natural systems that will be adversely impacted by increased wastewater flows at existing discharge sites include:

- Increased bacterial pollution of the Alexandria beaches
- Continued adverse impacts from industrial waste discharges to the Mex-Dekheila and Abu Qir shoreline areas
- Continued eutrophication and siltation of Lake Maryout--decreases in the areal extent of the lake with subsequent declines in fishery landings brought on by potential land reclamation projects

Social Systems

The public health consequences of increased population densities in non-sewered areas and increases in the discharges of untreated wastewater flows will be severe. Public health conditions would probably worsen in the year 2000, even though improvements might be made to the water supply system. The risks of contacting contaminated fresh and marine waters will increase. Worsening of the beach pollution could have a significant adverse impact on the tourism economy of the Alexandria region.

Summary

Clearly, the adverse environmental impacts associated with taking little or no action to improve the wastewater facilities of Alexandria are unacceptable. In fact, this course of action would conflict with the future prospects of growth and development of the region.

ADDITIONAL STUDIES NEEDED

This limited review and update of the environmental data base of the Alexandria region reveals that some information on environmental issues is unclear or unreliable, and information on others is completely lacking. The environmental impacts associated with a large-scale wastewater project cannot be completely evaluated unless the affected environment is well understood. Benefits can be maximized and adverse impacts minimized only if the environmental data base is adequate. Some of the key needs are listed below:

- Aerial photos of conveyance routes and potential sites for wastewater facilities
- Complete estimates of the incidence of waterborne diseases of the Alexandria governorate
- A better understanding of the public health problems specific to irrigated agricultural areas
- Extensive planning involvement with local academics, government officials, and United Nations agencies
- A complete evaluation of industrial pollution problems and impacts on water quality, aquatic biota, and man
- Recent and reliable estimates of the significance of fresh water and marine commercial fisheries in the Alexandria region
- Locations of potentially significant archaeologic resources in affected outlying areas

Specific needs in the areas of oceanography and marine biology are presented at the end of Appendix L, Sea Disposal and Outfall Review. In addition, detailed site investigations are recommended in Appendix K, Land Application of Wastewater. The scope of any additional studies will obviously depend on the plan selected.

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Table I-1
ALEXANDRIA CLIMATIC DATA SUMMARY (1942-1960)

Month	Mean Temp (°C) Maximum	Mean Temp (°C) Minimum	Mean Rainfall (mm)	Bright Sunshine (% possible)	Relative Humidity (%)	Mean Evaporation (mm/mo)
Jan	18.3	9.3	48.3	72.1	71	136
Feb	19.2	9.7	28.4	68.5	70	136
Mar	21.0	11.2	14.0	72.5	67	167
Apr	23.6	13.5	2.7	75.2	68	165
May	26.5	16.7	1.5	81.0	70	180
Jun	28.2	20.2	Trace	85.8	72	171
Jul	29.6	22.7	Trace	87.1	73	177
Aug	30.4	22.9	0.5	88.7	73	177
Sep	29.4	21.3	0.4	84.7	69	177
Oct	27.7	17.8	7.9	81.6	68	171
Nov	24.4	14.8	32.2	75.5	72	132
Dec	20.4	11.2	56.2	62.7	74	118
Annual Mean	24.9	15.9	192.1	78.1	70	1,907

Source: Alexandria Wastewater Master Plan Study, Initial Environmental Impact Study, Volume II, Final Technical Report, October 1978, Pages 3-15

Table I-2

MEAN MONTHLY WIND SPEED AND DIRECTION AT ALEXANDRIA (1952-1960)

Month	Mean Wind Speed (m/sec)	Directions (as percent occurrence)								
		N	NE	E	SE	S	SW	W	NW	Calm
Jan	4.4	5.2	8.1	4.6	10.0	4.2	29.3	11.9	19.7	5.9
Feb	4.6	9.4	14.3	8.5	10.1	3.9	14.8	8.0	27.3	2.9
Mar	4.7	9.0	17.2	8.1	9.3	2.5	6.7	8.8	34.8	2.4
Apr	4.3	12.4	18.4	7.2	8.6	1.5	3.4	4.2	40.4	2.7
May	4.1	19.6	19.4	4.1	6.5	1.0	2.1	8.3	39.1	3.8
Jun	4.1	18.6	5.2	0.8	1.6	0.4	1.8	1.8	65.8	2.3
Jul	4.4	16.2	3.7	0.2	0.3	0.2	0.7	3.5	71.6	2.3
Aug	4.0	16.6	2.6	0.3	0.5	0.4	0.9	2.2	70.4	4.2
Sep	3.6	28.2	8.3	0.9	1.8	0.6	1.3	0.6	51.2	4.8
Oct	3.2	26.0	22.9	4.[illegible]	4.5	1.5	4.8	2.1	25.1	5.2
Nov	3.4	12.6	24.7	6.3	6.4	2.9	11.2	4.8	24.2	5.0
Dec	4.1	6.7	18.1	6.7	7.9	4.2	27.1	4.9	16.9	5.5

Source: Alexandria Wastewater Master Plan Study, Initial Environmental Impact Study, Volume II, Final Technical Report, October 1978, pages 3-16

Table I-3

INCIDENCE OF SELECTED WATERBORNE DISEASES
IN THE ALEXANDRIA GOVERNORATE (1969-1979)
(no. of admitted cases/100,000 persons)

Year	Poliomyelitis	Infectious Hepatitis	Typhoid and Paratyphoid
1969	1.0	103.1	159.3
1970	0.3	106.5	107.1
1971	0.7	121.5	88.0
1972	2.1	122.8	131.6
1973	3.4	131.1	131.3
1974	4.4	152.6	134.8
1975	0.6	95.3	107.0
1976	1.2	95.9	61.1
1977	0.7	172.7	67.8
1978	0.8	175.5	55.9
1979	0.7	110.4	47.6
Average 69 - 79	1	126	99

Source: 1980 Annual Statistical Report for the Governorate of Alexandria, 1980, page 52 (in Arabic)

Note: Data only based on admissions at the one public communicable disease hospital in the governorate.

Table I-4
PREVALENCE OF PARASITIC INFECTIONS
IN ABIS II VILLAGE NEAR ALEXANDRIA (1978)

Parasite	Disease	Percent of Population[a] Affected	Reservoir(s)
Entamoeba coli	Amebiasis	48.6	Man, domestic animals
Schistosoma mansoni	Schistosomiasis	41.3	Man, aquatic snails
Entamoeba histolytica	Amebiasis	17.3	Man
Giardia lamblia	Giardiasis	11.2	Man, domestic and wild animals
Ascaris lumbricoides	Ascariasis	10.6	Man, swine?
Hymenolepis nana (dwarf tapeworm)	Taeniasis	9.5	Man, rat
Fasciola spp.	Fasciolliasis	7.3	Man
Schistosoma haematobium	Schistosomiasis	6.6	Man, aquatic snails
Trichuris trichiura	Trichuriasis	2.7	Man
Ancylostoma duodenale	Ancylostomiasis	2.0	Man

Source: Farag, H. F, R. M. R. Barakat, M. Ragab and E. Omar. Sept./Oct. 1979, A Focus of Human Fasciolliasis in the Nile Delta, Egypt. Journal of the High Institute of Public Health, Alexandria University, pages 188 - 190.

[a]Based on a random sample of 547 individuals (except 528 for S. haematobium)

Table I-5
SITES OF HISTORICAL/ARCHAEOLOGICAL SIGNIFICANCE
IN THE CITY OF ALEXANDRIA

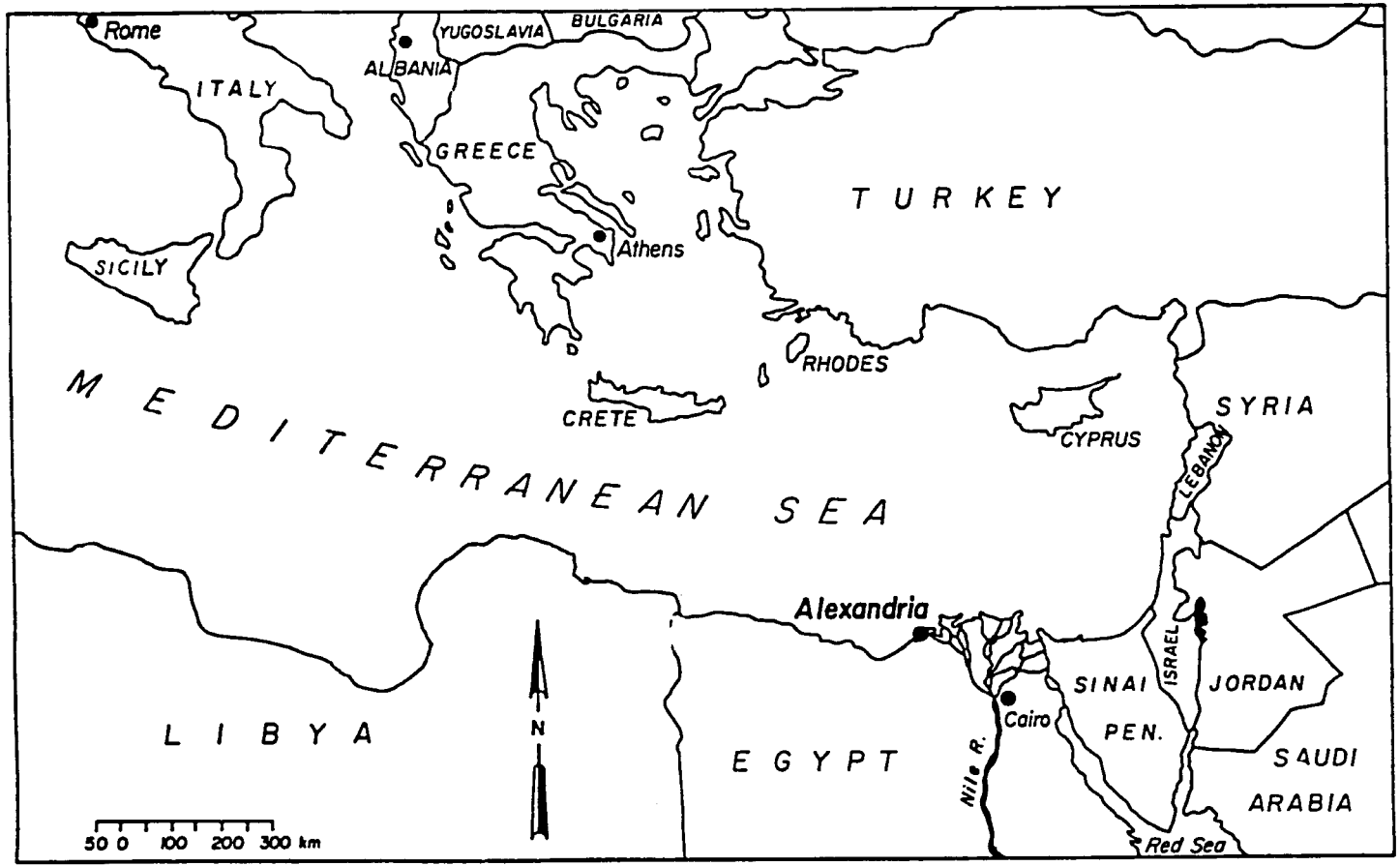
Site	Significance
Kait Bey Fort	Old location of the Pharos Lighthouse; fort now houses a naval museum
Catacombs Kom el-Shougafa	Underground rock burial chambers of Pharaonic style
Serapeum Temple	Site of a 27-m column, misnamed "Pompey's Pillar".
Greco-Roman Museum	Collection of Pharaonic mummies, amulets and Greek and Roman statues, furniture, jewelry and coins
Greco-Roman Theater	A recently restored ancient amphitheater
Necropolis of Mustafa Pasha	A Hellenistic cemetery patterned after a Greek dwelling containing well preserved murals

Source: Initial Environmental Impact Statement, Volume II, Final Technical Report. November 1978; Newcomers Guide to Alexandria, British Airways, 1978.

Table I-6
FUTURE PROJECTS PLANNED FOR
THE ALEXANDRIA REGION BY THE YEAR 2000

Project	Description
Amria	Planned city of 500,000 south of Alexandria
Sadat City	Planned city of 500,000 east in Montazah
Ras El-Soda, Sidi Gaber, and Ramleh	Planned residential developments in urban Alexandria of unknown size
Alexandria Maritime Port	50% expansion of port handling capacity
Dekheila Port	Addition of a refinery and iron and steel complex
Amria Free Zone	Specialized industrial park
Abu Qir Industrial Development	Construction of a large fertilizer plant and possibly other industries

Source: Appendix C, Planning Considerations, and the 1978 Environmental Impact Statement, Volume II, Final Technical Report.



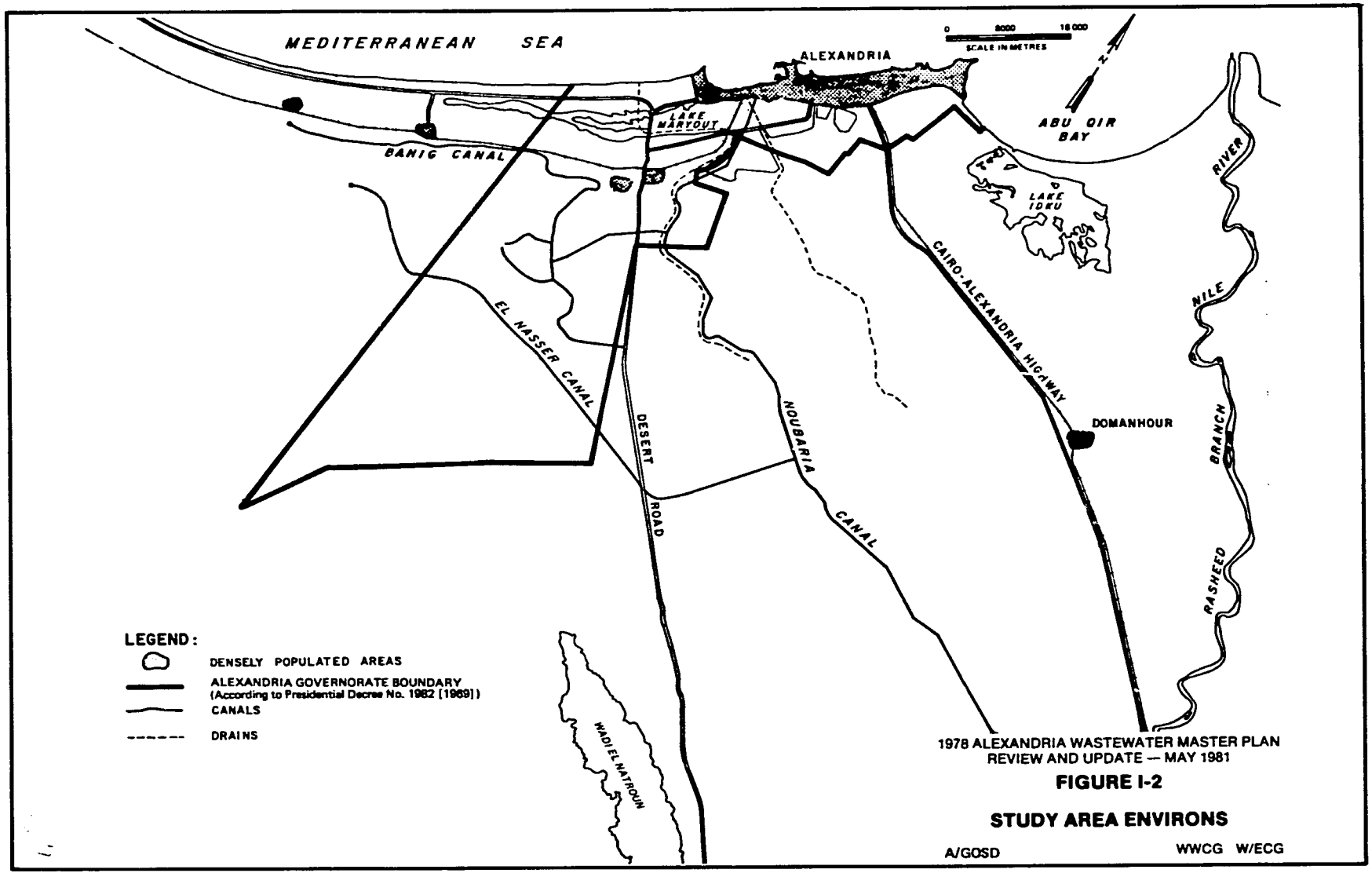
SOURCE: INITIAL ENVIRONMENTAL IMPACT STATEMENT, VOL. 1, 1978

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE I-1

LOCATION MAP

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE I-2

STUDY AREA ENVIRONS

A/GOSD WWCG W/ECG

APPENDIX J

TREATMENT ALTERNATIVES

INTRODUCTION

The 1978 Master Plan recommended sea disposal of treated sewage from the central, western, and eastern sewerage zones of Alexandria. The degree of treatment recommended prior to disposal consisted of coarse screening and grit removal. These unit processes constitute preliminary treatment.

Following publication of the 1978 Master Plan, significant modifications were made to the Master Plan in response to the USAID Final Environmental Impact Statement issued in August 1979. Specifically, USAID modified the original plan to upgrade wastewater treatment from preliminary to primary. This change in pretreatment requirements prior to outfall disposal significantly changed the possible cost effectiveness of previously considered alternatives regarding land disposal.

This appendix reports on the following analyses:

- The re-evaluation of treatment associated with the sea disposal alternative, adding primary sedimentation, chlorination, and primary sludge management processes
- Re-evaluation of plant siting for the sea disposal alternative
- Evaluation of secondary treatment with effluent discharged to drains

Evaluation of treatment needs associated with the land application alternatives are fully discussed in Appendix K, Land Application of Wastewater.

Effluent Requirements

As stated previously, effluent requirements for sea disposal of wastewater changed following publication of the 1978 Master Plan. Sea disposal effluent criteria used herein are shown in Table J-1.

Requirements for discharge of treated sanitary sewage to drains (which convey wastewater to the sea) are stated in Egyption Law No. 93 adopted in 1962. These requirements are also shown in Table J-1.

Land application of wastewater requires different levels of treatment. These are fully discussed in Appendix K.

Basis of Evaluation

The evaluation of the treatment alternatives required that the individual treatment unit processes be sized. This was necessary because adequate historical costs of similar facilities recently constructed are unavailable. In addition, some alternative unit processes had to be sized and evaluated before the overall treatment alternative could be evaluated. Design criteria were established as discussed under the section entitled "Process Design Criteria" in this appendix.

Population projections and industrial flow projections were made as discussed in Appendices C and D. The 1978 Master Plan documents were reviewed and additional data and reports examined to determine appropriate design flows and loadings. The results of the review and analyses (discussed in Chapter 2 and Appendix C) are presented in Tables J-2 and J-3.

Environmental concerns were also considered in evaluating treatment plant sites and unit processes. Specific concerns for siting new facilities or expanding existing ones included:

- Disruption to adjacent land uses
- Preemption of other types of land development
- Requirements for filling wetland or shoreline areas
- Several problems related to the construction of the associated conveyance system
- Anticipated disruption of cultural resources

Environmental aspects considered for the evaluation of specific unit processes include:

- Implication of treatment process on public health
- Potential for odor generation
- Risk assessment of transport and storage of chlorine cylinders

For example, the need for odor control facilities was considered in establishing the plant design criteria.

The environmental impacts of effluent disposal on sea or land are not discussed in this appendix since they are described in Appendices L and

K, respectively. However, the impacts of intermittent effluent disposal into nearby drains or Lake Maryout, due to power failures or bypasses resulting from construction activities, are evaluated in this appendix. The impact of discharging secondary effluent into drains that, in turn, discharge to the sea is also evaluated.

Opportunities for staging construction were investigated and taken where available. Such staging is advantageous, since it prolongs the useful life of the facilities not needed immediately.

Cost Basis

The purpose of the cost comparison used is to estimate the capital costs and the operation and maintenance costs for the treatment portion (and discharge portion for secondary treatment only) of the overall alternative. US cost curves for individual unit processes or facilities were adjusted for local conditions and then used. Staging the facilities (where such staging is feasible) is provided for. Costs for the other components (collection, conveyance, and disposal) of each alternative are estimated in other appendices, and a present worth analysis of the final alternatives is presented in Chapters 4 and 5.

WASTEWATER TREATMENT PROCESS SELECTION

Wastewater treatment processes were evaluated so that appropriate processes could be selected to satisfy the discharge requirements for the options being considered. The preliminary treatment processes evaluated for new plants were as stated in the 1978 Master Plan. These were flow measurement; coarse screening by mechanically cleaned bar screens; grit removal in aerated grit basins followed by removal of oil, grease, and other floatables by air flotation and skimming.

The additional primary treatment process that was evaluated was sedimentation, where about 60 percent of the suspended solids remaining after preliminary treatment are removed. A discussion of the treatment required for the resulting solids is covered in a following section of this appendix.

Biological treatment in addition to primary treatment will be the minimal requirement prior to effluent discharge to drains. Two types of biological processes were considered. Treatment by the activated sludge process is one normally acceptable method. This process is employed at the existing East plant. In the activated sludge process, treatment is effected by a dispersed mass of microbiological organisms (microbes) in the presence of oxygen. The microbes consume the organic pollutants

contained in the wastewater. The oxygen can be supplied to the microbes by a variety of mechanical systems. Active microbes are removed in secondary sedimentation tanks following the aeration step and recycled back to the aeration tanks in order to maintain a sufficient number of active microbes in the tanks to achieve the desired BOD removal. An excess of biological organisms is produced and must be wasted from the system. This solid material is removed in a slurry containing about 0.5-1 percent solids, and must be further concentrated and/or treated prior to discharge.

The second biological treatment process considered is generally termed a trickling filter, fixed biological growth reactor, or biological tower. The term "biological tower" will be used subsequently in this appendix. The specific system considered most appropriate for Alexandria would employ short (8.5 m) media depth towers filled with a plastic media. The media is designed to have a large surface area per unit of volume, while still presenting no threat to plugging by primary treated wastewaters. The microbes necessary for treatment attach themselves to the media surfaces. The hydraulic flushing action provided by wastewater and recycle flows applied to the top of the tower flushes excess microbes from the media. These excess microbes, termed secondary sludge, are largely removed in secondary sedimentation tanks which, in the flow sequence, follow the towers.

The alternative considered for Alexandria would incorporate a system to return a portion of the secondary sedimentation tank underflow, containing the secondary sludge, to the tower influent. With adequate recycle a suspended microbe growth results, giving the process some characteristics of the activated sludge process without the associated operational difficulties. Some additional BOD and suspended solids removals are experienced over that which would be experienced in the standard plastic media biological tower process. Aeration can be provided to the secondary sedimentation tank feed conduits to further increase performance.

As previously indicated, only a part of the secondary sedimentation tank underflow containing secondary sludge is recycled to the tower influent. The remainder is wasted to the primary sedimentation tank influent. The secondary sludge contained in this recycled flow co-settles with and is incorporated into the primary sludge.

The advantages and disadvantages of the above systems are shown in Table J-4. For Alexandria, the biological tower is the unit process chosen for further consideration at new plants or plant expansions. The basic reasons the biological tower was chosen for further evaluation are: (1) it is much easier to operate, (2) it is less costly both in construction and operation, and (3) it requires less land space.

For land application of effluent, only screening of wastewater at the collection system pump stations will occur prior to transport to the land application site. A discussion and evaluation of treatment for the flows transported to the site is covered in Appendix K.

SOLIDS MANAGEMENT

The solids management alternatives considered were those that could most reasonably be incorporated into the wastewater management options developed. The factors used in selecting the alternatives were:

- Simplicity of operation
- Reliability, considering available operational skill
- Land use
- Public health
- Costs

Wastewater solids are of four general types: grit, which is sand and other inorganic constituents; screenings, consisting of rags, sticks, and other organic or inorganic gross solids; scum, composed of oil and grease and other floatable solids; and sludge, which is made up of the settleable organic and inorganic solids removed in primary or secondary treatment. The Master Plan Review assumed that grit, screenings, and scum from all wastewater facilities would be landfilled or managed by other readily available and acceptable methods, such as recycling. The 1978 Master Plan recommendations are endorsed in this regard. This review only concerns the evaluation of options for managing sludge.

The components of a sludge management system can be categorized into treatment, transport, and use/disposal. Treatment includes those processes that alter the physical, chemical, or biological character of sludge to enhance ease and suitability of transport and use/disposal. It includes such processes as conditioning, thickening, dewatering, stabilization, and reduction.

Transport includes those processes that are the interface between treatment and use/disposal or between treatment processes. It is dependent on the treatment processes used and the intended use/disposal. For example, if sludge is dewatered at the treatment plant, then truck transport to the final use/disposal site is possible. If the sludge is not dewatered at the treatment plant then either pipeline or truck transport can be used.

The use/disposal options actually dictate the required sludge treatment. Type and location of sludge treatment facilities establish transport options. As an illustration: if soil amendment is the desired use/disposal method, then the sludge should be stabilized or stored for an extended period of time to reduce the potential for odor and to permit destruction of pathogenic organisms. If it is determined that the sludge should be stabilized and dewatered at the treatment plantsite, then trucking is the only viable option for transport to the point of reuse.

Treatment Options

Sludge treatment can include thickening and/or dewatering to remove water and thus lessen the volume to be managed. Thickening can be by gravity settling methods or flotation using dissolved air. Neither of these alternatives is recommended in this study because readily settleable sludge would be produced under all wastewater treatment options considered. Sludge concentrations of at least four percent solids should be obtainable in the primary sedimentation tank underflow, constituting the total waste sludge in secondary as well as primary treatment facilities.

Dewatering can be by mechanical means using sophisticated frame filter presses, less complex belt presses and centrifuges, and relatively simple vacuum filters. Each of these dewatering devices is preceded by conditioning, using thermal or chemical means. Thermal conditioning systems are complex, require a high degree of skill, are odorous, produce a difficult to process liquid sidestream, and are relatively costly. Chemical conditioning uses lime and ferric chloride or organic polymers to alter the physical character of the sludge, enhancing its dewaterability.

Of the mechanical means, vacuum filtration using polymer conditioning was the dewatering method selected for cost estimating purposes in dewatering primary sludge alone. This system should be capable of producing a dewatered sludge cake of at least 23 percent solids. The selection of this option is based on its proven reliability, simplicity, and relatively low level of operation and maintenance skill required. Sludge dewatered by a vacuum filter must be stabilized or stored for an extended period to permit destruction of pathogens prior to unrestricted use on farm land.

A belt press system, with polymer conditioning, was selected for dewatering combined primary and waste biological tower secondary sludge. A dewatered sludge cake of 25 percent solids is anticipated. This system was selected for application because the coil media vacuum filters

assumed for dewatering primary sludge alone will not operate satisfactorily where secondary sludges are involved. The belt presses are proven dewatering devices and should serve well in this application. Again, additional stabilization of the dewatered sludge is required for pathogen destruction prior to unrestricted land application.

Sludge can also be dewatered by nonmechanical means using sand drying beds or anaerobic lagoons. Drying beds are presently installed at the East and West Treatment Plants. The main disadvantage of these methods of dewatering is the large land area required. Considerable operational manpower is needed but this need not be skilled labor. Solids concentrations as high as 50 percent can be expected. Sludge from anaerobic lagoons is stabilized as a result of the anaerobic conditions during the detention period (as long as 3 years) and is suitable for direct use on farmland. Sludge from drying beds, however, must be stored for 1 to 2 years to permit pathogen destruction.

Sludge can be stabilized by anaerobic or aerobic digestion, chemical or thermal methods, composting, anaerobic lagoons, or long-term storage. Anaerobic and aerobic digestion, and thermal stabilization entail high capital expenditures; and aerobic digestion, chemical and thermal stabilization are associated with relatively high capital, and operation and maintenance costs. All of these systems require relatively sophisticated operational skill as compared to composting, anaerobic lagoons, or long-term storage. The main disadvantages of these latter three stabilization methods are the large land area requirements. However, where land is available these disadvantages are outweighed by the simplicity and reliability of these processes. Consequently, composting was the stabilization method evaluated following mechanical dewatering and prior to land application. Long-term storage was selected to follow drying bed dewatering. Both of these processes can produce a satisfactory product of from 50 to 70 percent solids concentration.

Reduction processes, e.g., incineration, were not considered for Alexandria due to the unreliability of these systems. Also, such systems entail relatively high capital and operation costs compared to other alternatives. Finally, ash from sludge incineration could contain high levels of heavy metals due to industrial influences and thus could require nonconventional landfilling methods to preclude environmental contamination.

Transport Options

Transportation is required between treatment processes and for moving the final sludge product to its point of disposition. If all sludge treatment processes are at the same site as the source of the sludge,

then conventional pumping and conveyor systems are appropriate. However, if sludge treatment processes and final disposition points are remote from the sludge source, then sludge must be transported by sophisticated pumping systems or trucks.

Trucks can be used for transport of either liquid or dewatered sludge. High traffic volumes and narrow streets in Alexandria make truck transport difficult; therefore, truck transport must be kept to an absolute minimum and was considered only for dewatered sludge. This method of transport entails low capital costs relative to pumping systems, but higher operating costs.

Pipeline transport of sludge was also considered for Alexandria's sludge, and would be appropriate where relatively high sludge quantities are to be transported. Pipeline transport eliminates the problem of traffic congestion, but can cause significant local traffic problems during construction. This method of transport is characterized by comparatively high capital costs, but relatively low operating costs.

Use/Disposal Options

Considering environmental impacts, sludge can be disposed of only to the land, either by landfilling or land application in support of agriculture, horticulture, or silviculture. In the case of Alexandria, land application in support of agriculture was the method considered. Landfilling would be inappropriate and wasteful of the nutrients in sludge. This method is even less acceptable considering the agricultural community close to Alexandria and the need for nutrients and organic matter contained in the sludge.

The land-applied sludge would be produced by composting, anaerobic lagoons, or drying beds with long-term storage. This evaluation assumed the user would transport the sludge at least part of the way to his fields and would apply the sludge product at no cost to A/GOSD. This is the present method of sludge disposal employed at the East Treatment Plant and is intended for sludge from the West plant.

PROCESS DESIGN CRITERIA

The process design criteria used in this study are shown in Tables J-5 and J-6. These criteria are general goals, not absolute limits. When unit processes are sized, the size chosen is based on many factors. One important factor is constructibility. Dimensions should be matched to methods of construction and availability of material that may be produced in standard sizes. Therefore, the resulting size of process units

will usually not produce loading values identical to the design criteria.

INDUSTRIAL WASTEWATER IMPACTS ON TREATMENT/DISPOSAL

The projection of industrial flows and general waste strengths were presented in Appendices C and D. The 1978 Master Plan contained recommendations for industrial discharges. These recommendations covered in-plant modifications, such as improved plant operations, recycling of water, segregation of some flows and constituents, and pretreatment. Disposal of toxic wastes to an acceptable facility was recommended and could be considered a part of pretreatment. Other pretreatment measures could include any appropriate combination of the following examples: pH adjustment, screening, oil and grease removal, coagulation for toxic removal, sedimentation, and flow equalization. The 1978 Master Plan further recommended that the industries discharge to the sewers following pretreatment. That recommendation is endorsed.

Primary Treatment

The industrial impacts on primary treatment processes are expected to be minimal with the exception of those from large quantities of oil, grease, and fibers. The fibers of concern are those from the pulp, paper, and textile industries. However, oil and grease and such fibrous material that would be detrimental to proper operation of the collection system, including pump stations, should be removed prior to discharge to the sewer system. The levels of oil and grease that remain can be effectively removed in the flotation section of the primary plant. Remaining fibers can be largely removed in primary sedimentation if low overflow rates are used.

Secondary Treatment

Industrial wastes discharged to secondary treatment plants may require pretreatment to remove toxic substances. Heavy metals and toxic organic pollutants in sufficient concentration can inhibit the biological treatment processes, and some industrial pollutants can inhibit biological activity to a point where the sewage might have to be bypassed while biological treatment units are cleaned and the plant restarted.

The analysis presented in the 1978 Master Plan indicates that lead from some industries must be removed to prevent toxicity in the activated sludge process at the East plant. If discharged to a larger biological plant, such as Ras El-Soda, the level of lead at the plant (because of dilution effects) may not be toxic to the biological process.

Likewise, if the West plant were to provide secondary treatment, lead would need to be removed from some industrial wastewaters in its service area to avoid toxicity problems. This conclusion is based on the data presented in the 1978 Master Plan. Lead is now being discharged in sufficient quantity that it would be toxic to the biological process that would be constructed for any size West plant evaluated in this study (85 Ml/d to 395 Ml/d). The 1978 Master Plan data did not indicate that other heavy metals would pose a problem to the treatment process in a secondary plant.

No data was presented in the 1978 Master Plan on organic pollutants from industry. As such, an evaluation of their impact on secondary treatment cannot be made at this time.

Sea Disposal

The impact of industrial wastewaters on sea disposal was addressed in the 1978 Master Plan Study, Initial Environmental Impact Statement. The study concluded that, with the recommended pretreatment program operating satisfactorily, there should be no problem with industrial wastes. The study also examined the effect of no pretreatment. The conclusion was that, with pretreatment, there would be no harmful effect to the sea or its users. That conclusion may not be warranted, as discussed further in Appendix L.

Land Application

Data presented in the 1978 Master Plan Report on industrial discharges indicate the presence of significant quantities of heavy metals in industrial discharges. There could be a problem if the amount shown for chromium were discharged to a land application system without further pretreatment. The current amount of industrial chromium discharge reported in the 1978 Master Plan is approximately 85 kg/d. If industries with chromium-rich wastewater expand in number and/or capacity, this quantity will increase. The maximum allowable chromium loading to a land application system serving the study area would be about 94 kg/d. This is based on sludge management practices in the United States. Some of the chromium in the wastewater would be removed as sludge in the anaerobic lagoon (perhaps 50-60%). If the sludge is applied to the same land as the effluent, the entire chromium content must be considered as land applied.

Sludge Utilization

Sludge utilization is proposed for all treatment alternatives considered herein. Sludge would either be composted or digested and dewatered in place in anaerobic lagoons. In either case, the data presented in the 1978 Master Plan Report indicate that copper, zinc, and nickel could limit the application of sludge at a normal rate of about 22.5 dry tonnes/ha/yr for a 20-year period. Pretreatment to reduce these heavy metals, together with chromium and lead, would be preferable to the alternative of decreasing the sludge application rate.

CURRENT MASTER PLAN WITH PRIMARY TREATMENT--ALTERNATIVE IA

This alternative is the sea disposal alternative recommended by the 1978 Master Plan with the addition of primary treatment and sludge treatment/disposal. New primary treatment plants would be located at Ras El-Soda (585 Ml/d) and Kait Bey (195 Ml/d). The West Treatment Plant, currently under construction would be expanded to 200 Ml/d capacity. Chlorinated effluent from the West and Kait Bey plants would be discharged to the Mediterranean Sea through an outfall 8 km long extending seaward from Kait Bey. An outfall 10 km long at Sidi Bishr would similarly serve the Ras El-Soda plant. Sludge would be dewatered and composted for distribution to local farmers. The existing East Treatment Plant would be rehabilitated under this alternative (as is the case with all alternatives) and would provide a 45 Ml/d capacity of secondary (activated sludge) treatment. This work would be completed prior to other construction which would help alleviate some of the existing raw sewage discharges.

East Plant

The existing activated sludge secondary treatment plant was planned to be improved and renovated. This plant is located on an approximately 32-ha site near the northwest corner of the Hydrodrome. The upgraded plant was planned to provide secondary treatment for an average design flow of 45 Ml/d and a peak flow of 70 Ml/d. Only minor capital expenditures were anticipated necessary to accomplish the renovation. Effluent was planned to be discharged into the Qala Drain.

The current East plant situation is discussed in Appendix E. The design capacity of the East plant is to be 45 Ml/d (average dry weather flow) with discharge to the drain. Modifications recommended in Appendix E are considerably more extensive than those outlined in the 1978 Master Plan. This is a result of the more detailed evaluation of the plant as

presented in Appendix E. Chlorination of the effluent is needed, as recommended in Appendix E, to meet public health requirements. A site plan of the East plant showing some of the proposed modifications is presented on Figure J-1.

Few adverse environmental impacts are anticipated at the East plant if upgraded as proposed. Improved treatment and disinfection will substantially improve water quality in the Qala Drain and the Lake Maryout main basin. Extensive sludge accumulations in the drain at the site of the existing raw discharge will remain for some time. The potential for adverse odor impacts at this site from treatment units and sludge drying beds is anticipated to be minor since abutting areas consist of low density industrial and transportation land uses.

Chlorine transport and storage does represent a potential hazard to treatment plant workers and the general populace along the truck transport route. Strict safeguards should be instituted to ensure that chlorine cylinders do not fail during operation or storage since worker housing is located within 250 m of the proposed chlorine storage area (Figure J-1). Likewise, the truck transport route from the site of chlorine manufacture to the treatment plant should be selected to consist of major roadways that avoid high density residential or commercial uses. The feasibility of rail transport should be investigated since the East plantsite is within several hundred metres of an existing railway.

West Plant

An 85 Ml/d primary treatment plant is presently under construction on an approximately 10.2-ha site located on the northern shore of the Lake Maryout main basin. The 1978 Master Plan proposed conversion and expansion of this facility into a 220 Ml/d preliminary treatment plant. Treatment processes to be provided included mechanically cleaned screens, aerated grit chambers, and air flotation chambers. This plant was also planned to include two large, mechanically mixed flow equalization basins. The majority of influent to this plant was planned to be pumped by a new influent pump station. A new effluent pumping station having a firm capacity of 325 Ml/d was planned to deliver effluent to the Kait Bey effluent pump station for repumping to the submarine outfall associated with the Kait Bey facility. Grit and screenings were to be landfilled.

Since the Master Plan was published, considerable construction work on the West plant has been completed. The modifications recommended for the plant as an 85 Ml/day primary facility are given in Appendix F.

The flows for the year 2000 have changed so that the revised average dry weather flow to be used for design would be 200 Ml/d. Expansion of the plant to 200 Ml/d would be accomplished by adding the following:

- Mechanically cleaned bar screens--two screen channels plus one bypass channel
- Aerated grit basins--four tanks and a 130-m^2 overall area
- Flotation basins-four tanks and a 215-m^2 overall area
- Flow measurement--two Parshall flumes, each with an 0.9-m throat
- Primary sedimentation--16 tanks in a 4320-m^2 overall area
- Effluent disinfection--four evaporators and four chlorinators (2 tonne/d units)
- Effluent station--pump to the Kait Bey outfall pump station
- Chlorine storage--30 1-tonne cylinders in a 320-m^2 area
- Sludge handling facilities--drying beds and long-term storage before agricultural reuse.

Grit would be removed by pumps and then separated and dewatered by grit cyclones and washer/classifiers. Screenings and dewatered grit would be hauled by truck to a landfill site. Oil and grease would be removed from the flotation basins and the primary sedimentation tanks and then hauled to a landfill. A revised site plan is shown on Figure J-2.

The adverse environmental impacts of improving and expanding the West plant include:

- Hazards due to transport and storage of chlorine cylinders
- Filling of approximately 3 ha of Lake Maryout
- Potential odor impacts on nearby residences

Worker housing at the West plant is located within 250 m of the proposed chlorine storage facilities and thus represents a major hazard if chlorine cylinders fail during storage or operation. Truck transport access to the site is very limited since roadways leading to the plant in the congested residential areas are narrow and unpaved. In addition, trucks entering the plantsite would have to cross an active railway.

These chlorine transport hazards can be minimized if the selected truck transport roadway is paved and a controlled railway signal crossing is constructed near the plantsite. Impacts of filling approximately 3 ha of reed grass (Phragimites Communis) marsh on the shores of the main basin of Lake Maryout (1880 ha total area) are expected to be minor.

Full evaluation of the cumulative impacts of filling is not possible due to the lack of reliable information on sedimentation processes in the shallow lake and filling proposed for other projects.

Odor impacts due to treatment units and sludge drying beds are expected to be minor since the prevailing winds (from the northwest) should ensure that the congested residential areas to the north and northwest are rarely impacted.

Kait Bey Plant

A new primary treatment plant located on a 0.9-ha site at the location of the existing Kait Bey pumping station was proposed in the 1978 Master Plan. Treatment and disposal processes to be provided were the same as previously discussed for the West Plant expansion. The design capacity was to be 175 Ml/d, average dry weather flow, with a peak capacity of 255 Ml/d. All flow was planned to be conveyed to the plant by gravity. The associated submarine outfall and effluent pumping station were planned to include capacity for flows from the West Treatment Plant in addition to the flow from this plant. Accordingly, the effluent pumping station was planned to have a firm capacity of 577 Ml/d.

The preliminary treatment plantsite development shown in the 1978 Master Plan included a 0.9-ha area fill in the sea. Additional fill area required to accommodate primary treatment was estimated at 1.3 ha. These fill areas appear adequate for these unit processes; however, locating mechanical dewatering at this site would require even more filling.

The 1978 Master Plan did not show the need for an influent pump station. One would be needed, however. The influent sewers are approximately 2.4 m below sea level, and therefore the bottoms of the treatment basins would vary from about 6 to 9 m below sea level unless influent pumping were provided. Thus the basins would be subjected to uplift forces of about 7,260 kg/m^2.

The population and flows for the central zone have been updated as part of this study. The average dry weather flow for the year 2000 has been changed to 195 Ml/d with a peak of 295 Ml/d. Facilities to provide primary level treatment to these flows would be accomplished by constructing:

- An influent pumping station
- Mechanically cleaned bar screens--three screen channels and one bypass channel
- Aerated grit tanks--four tanks in a 202-m^2 overall area
- Flotation tanks--four tanks in a 336-m^2 overall area
- Flow measurement--two Parshall flumes, each with a 1.2-m throat
- Primary sedimentation--24 tanks in a 6100-m^2 overall area
- Effluent disinfection--four evaporators and four chlorinators (2 tonne/d units)
- Effluent pump station--pump through the Kait Bey outfall
- Chlorine storage--30 1-tonne cylinders in a 320-m^2 building
- Sludge handling facilities--belt press dewatering and truck hauling to composting site in agricultural area

A revised site plan is shown on Figure J-3.

Siting the primary treatment facilities at the Kait Bey site as shown in Figure J-3 would result in a major adverse impact. Specific impacts associated with this site include:

- Incompatible land use in a densely populated coastal area
- Filling more than 2.2 ha of shoreline
- Hazards associated with chlorine transport and storage in a densely populated area
- Odor impacts on nearby businesses, residents, and tourists
- Relocation of small businesses
- Adverse impacts due to transporting dewatered sludge by truck

Incompatibility of treatment facilities at this site is evidenced by the surrounding land uses. Small businesses would be relocated and the proposed site would be within 50 m of a school and large office building. A night club and a yacht club are located within 100 m and a mosque

within 250 m of the site. In addition, the historically significant Kait Bey Fort is located within 500 m.

Filling over 2.2 ha of shoreline would detract from natural features and also require the construction of a sea wall to prevent wave damage.

The hazards associated with chlorine storage and transport are anticipated to be severe due to the high population density near the site. Potential chlorine leaks at the site could impact the densely populated regions to the southeast, including the downtown shopping zone 1 km away, since prevailing winds are from the northeast. Chlorine transport via truck would also be hazardous since roadways to the site are narrow and congested.

Odors from treatment processes could affect prime areas of the Eastern and Western Harbors, the adjacent Kait Bey Fort, and prime beach areas east to Montazah. Covering the basins would partially alleviate the odor potential but would add to the cost of treatment facilities.

Location of the facilities at Kait Bey would result in the relocation of many small businesses and fishermen. It is also possible that the adverse odor impacts discussed above could preclude future development or expansion of existing commercial, public, or tourist facilities.

Adverse traffic impacts from hauling dewatered sludge would result due to the heavy congestion on roadways. If dewatered sludge is hauled by truck, one truckload would leave every half hour over a 14-hour period. The impacts of truck traffic and likely odors would be extremely undesirable. If pipeline transport of liquid sludge were to be used instead of trucks hauling dewatered sludge, the short-term impacts of constructing a dual sludge pipeline through an extremely congested part of the city would be high. The long-term impact would be the danger to public health in the event of a pipeline break in the downtown area or along the harbor, depending on the route selected.

Ras El-Soda Plant

A new preliminary treatment plant located on a 5.25-ha site in the neighborhood of Ras El-Soda was planned in the 1978 Master Plan. Treatment and disposal processes to be provided were the same as previously discussed for the West Plant expansion. Effluent pumping was planned at the plant, and effluent disposal was proposed through a submarine outfall to the Mediterranean Sea. Removed grit and screenings were to be transported by truck to a landfill. All influent to this location was planned to be pumped. A design average dry weather flow capacity of 560 Ml/d and a peak capacity of 850 Ml/d were planned. Firm capacity of the effluent pump station was planned at 875 Ml/d.

A site plan was presented which showed the land required if primary treatment was added. However, insufficient space was shown for primary sedimentation even if sludge dewatering facilities were to be offsite. Effluent disinfection by chlorination was not included as a part of preliminary treatment but would be added with the addition of primary sedimentation.

The Ras El-Soda site presented in the 1978 Master Plan was small (5.25 ha) and located in a densely populated neighborhood. The adverse environmental impacts associated with this site include:

- Relocation of adjacent multifamily residences
- Odor impacts
- Hazards due to chlorine transport and storage
- Added noise, dust, and debris due to construction activities

Since this particular site was not considered large enough and not studied, additional discussion of environmental concerns is unnecessary.

The population of and estimated flows from the Ras El-Soda plant service area have been updated as part of this study. The estimated average dry weather flow for the year 2000 has been changed to 585 Ml/d with a peak of 895 Ml/d. Proposed process facilities to provide these flows with a primary level of treatment would include the following:

- Mechanically cleaned bar screens--six screen channels and two bypass channels
- Aerated grit tanks--six tanks in a 560-m^2 overall area
- Flotation tanks--six tanks in a 930-m^2 overall area
- Flow measurement--four Parshall flumes, each with a 1.8-m throat
- Primary sedimentation--64 tanks in a 18,400-m^2 overall area
- Effluent disinfection--five evaporators and five chlorinators (4 tonne/d units)
- Effluent pump station--four constant-speed and three variable-speed pumps (two redundant pumps), 1000 hp each; 7460 m^3/hr at 30 m TDH, 585 rpm; 750 mm diameter suction, 600 mm diameter discharge

- Chlorine storage--96 1-tonne cylinders in a 345-m^2 building
- Sludge handling facilities--vacuum filter dewatering followed by composting

A revised site plan is shown on Figure J-4.

The environmental impacts associated with a revised site location for this plant are the same as for other alternatives discussed hereinafter and, accordingly, are presented in a following section.

Outer Areas

The following treatment facilities are to be evaluated in more detail in subsequent work but are mentioned here to provide a complete summary of the treatment facilities recommended in the 1978 Master Plan. The method of treatment and size of process units for Amria, Mex-Dekheila, and Abu Qir industrial flows will not affect the comparison of the sea disposal, secondary treatment, and land application alternatives.

Amria. A new waste stabilization facility was planned for a 400-ha site adjacent to the Noubaria Canal and south of Desert Road. Treatment was to be by series anaerobic-aerobic ponds. Effluent recirculation from the aerobic ponds to the anaerobic ponds was to be provided. Effluent discharge to the existing Noubaria Canal was planned. The average daily dry weather flow design capacity was set at 137 Ml/d.

A well designed and operated anaerobic-aerobic pond system should achieve wastewater influent BOD and suspended solids removals similar to those achieved in mechanical secondary treatment plants. However, algae produced in the treatment process will raise the effluent suspended solids and BOD levels above that allowed by current law, as indicated in Table J-1. These solids are typically not detrimental to health. In fact, US practice allows a higher suspended solids level in effluent from small lagoons (less than 3.78 Ml/d in capacity) than it allows in mechanical secondary treatment plant effluent. Because of the anticipated higher effluent BOD and suspended solids concentrations (equal to or greater than 55 mg/l), either solids removal facilities must be added to the treatment method or the law changed to allow the discharge of algae to drains.

The treatment ponds proposed in the 1978 Master Plan for Amria and Mex-Dekheila were inadequately sized, particularly when considering the influent wastewater strength anticipated. Design criteria proposed in the 1978 Master Plan and alternative criteria proposed in this review are as follows:

- Anaerobic ponds--proposed detention time with all ponds in service
  - 1978 Master Plan = 3 days
  - Recommended by this review = 8 days
- Anticipated BOD Removal
  - 1978 Master Plan = 35 percent
  - Recommended by this review = 45 percent
- Facultative ponds following anaerobic ponds or primary treatment--proposed BOD loading, all ponds in service
  - 1978 Master Plan = 300 kg/ha/day
  - Recommended by this review = 200 kg/ha/day
- Facultative ponds receiving raw wastewater--proposed BOD loading, all ponds in service
  - Master Plan = not considered
  - Recommended by this review = 150 kg/ha/day

The recommended design criteria changes have basis in current state-of-the-art practice and experience with pond systems in warm climates. Organic loadings significantly higher than those recommended in this review would be expected to yield severe sludge management and odor problems in anaerobic lagoons, up to 50 percent of which must be out of service at any one time for sludge drying and removal. Overloading of facultative ponds, particularly where the wastewater has not been pretreated for settleable solids removal, can lead to severe pond overloading near inlets, production of unmanageable sludge deposits, and odor problems.

The sites for anaerobic ponds must be chosen with caution as even a well-designed system will yield some foul odors, and upset conditions can result in odors that extend well beyond the pond system. If such pond systems cannot be placed well away from human habitation (3 to 5 km minimum), then they should not be used.

In the event the selected treatment/disposal alternative for Alexandria is land treatment, then wastewater from Amria should likely be included in the land treatment plan. Incorporation of wastewater flows from other outer areas may also be appropriate.

Mex-Dekheila. A new waste stabiliz[illegible]n pond facility similar to that proposed for Amria was planned. These ponds were planned to occupy a 400-ha site located in the western basin of Lake Maryout immediately south of the northern shoreline. Concerns about anaerobic pond siting and the pond design criteria changes recommended in discussion of the Amria area also pertain here.

Lake Idku. A new evaporation pond facility located on a 4000-ha site in Lake Idku was planned. Influent flow would be pumped to this facility for disposal through evaporation. A design capacity for an average daily flow of 150 Ml/d was planned. Concern over taking Lake Idku area from fisheries and wetland uses may preclude implementation of this plan.

Solids Management

The addition of primary sedimentation to the treatment plants recommended in the 1978 Master Plan significantly increases the scope and cost of treatment. Approximately 317,000 kg/d of primary solids must be removed, dewatered, and treated under the sea disposal alternative.

Insufficient land is available at Kait Bey for mechanical sludge dewatering without placement of additional (beyond the 2.2 ha area) fill in the sea. Because of this and adverse environmental impacts from trucking sludge, liquid sludge pumping to the West plant for mechanical dewatering would be preferred if the Kait Bey plant were to be constructed.

Alternative IA with its associated need for a large primary treatment plant is considered less feasible than other possible alternatives. The proposed plantsite at Kait Bey is inappropriate for several reasons. The adverse environmental impacts were delineated earlier. Engineering and associated cost considerations yield the following additional conclusions:

- A sea wall would have to be constructed to protect structures. Possible breaching of the sea wall by heavy seas could inflict damage to structures and equipment.
- Costs for demolishing existing structures and businesses would be incurred.
- Decreased life to structures and equipment and/or use of costlier materials would result from exposure to the corrosive salt spray atmosphere.

- The need for an influent pump station increa: ıe cost over that indicated in the 1978 Master Plan.

- If pipeline transport of liquid sludge (to West plant for dewatering) were to be used instead of truck transport, capital costs would be significantly increased.

- The capital cost of a plant at Kait Bey, with the considerations listed above and others to mitigate adverse environmental impacts described before, would be higher than those associated with providing the same treatment capacity at the West plant. Not only are many of the above items not needed at the West plant, but economy of scale and fewer units needed to satisfy redundancy requirements reduce the capital cost for a single plant serving the western and central zones at the West plant site. The West plant flows require pumping to Kait Bey in any event, and an effluent pump station for the combined flow is required. Therefore, the capital cost of pump stations is similar for both a West plant location and Kait Bey location. Pipeline and operating costs to pump the central zone flows to the West plant and back are the only cost increases. This is offset by the decrease in capital and operational cost for combining the two plants at the West plant.

A better location for treating wastewater flows from the central zone is at the West plant, in conjunction with the western zone wastewater flows. This is the configuration discussed hereinafter. Therefore Alternative IA is not considered further in this appendix.

The remainder of this appendix proposes an alternative primary treatment siting configuration appropriate to sea disposal; it develops and discusses an alternative for secondary treatment with the secondary effluent discharged to drains leading to the sea; and it provides a site evaluation and re-examination of mechanical plants (Fast and West) remaining in service under the land application alternative. Solids management for the alternatives considered to be most viable is discussed and evaluated in the succeeding sections.

SEA DISPOSAL ALTERNATIVE IB

This alternative is similar to Alternative IA except that only two primary treatment plants are required. The largest, at Ras El-Soda, would be the same as for Alternative IA. The second, a larger (395 Ml/d) expansion of the West plant, would serve the balance of the sewered area of Alexandria. Each plant would have its own outfall, similar to

Alternative IA. Sludge treatment and disposal as well as the rehabilitation of the East plant would be as previously described.

East Plant

The East plant would be as described in Appendix E and as shown on Figure J-1. It would be a secondary plant sized for an average flow of 45 Ml/d with solids treatment by drying beds and long-term storage onsite. It will initially discharge to the Qala Drain.

The disinfected and nitrified secondary effluent anticipated to be discharged from the East plant after its upgrading should be of high quality, containing only approximately 20 and 30 mg/l BOD and suspended solids, respectively. As such, it has value for irrigation. The plant is near both industry and agriculture. Once the plant is upgraded and its ability to produce a high quality effluent demonstrated, the effluent should be marketed to those who will reuse it.

Environmental impacts at the East plant under this sea disposal alternative are as previously described under the current Master Plan with primary treatment (Alternative IA).

West Plant

The West plant would take flows from the central and western zones amounting to 395 Ml/d. Therefore, a plant expansion of 310 Ml/d over that which is now under construction would be required. Recommended modifications to the 85 Ml/d plant under construction are presented in Appendix F. In the event the sea disposal alternative is selected for implementation, all recommendations for plant upgrading should be put in place except for those pertaining to addition of sludge drying bed capacity. All sludge from the expanded plant would be mechanically dewatered and the existing drying beds would be removed. The expansion to 395 Ml/d capacity would include addition of the following process units:

- Influent pump station--310 Ml/d capacity with three constant-speed and, two variable-speed pumps, 150 kW each
- Mechanically cleaned bar screen--four screen channels plus a bypass channel
- Aerated grit--six tanks in a 300-m^2 overall area
- Flotation--six tanks in a 500-m^2 overall area

- Flow measurement--two Parshall flumes, each with a 1.2-m throat
- Primary sedimentation--six tanks in a 9800-m^2 overall area
- Effluent disinfection--six evaporators and six chlorinators (2 tonne/d units)
- Effluent pump station-- three constant-speed and three variable-speed pumps (two redundant pumps), 375 kW each; 8403 m^3/hr at 12 m TDH, 585 rpm; 900 mm diameter suction, 800 mm diameter discharge
- Chlorine storage--60 1-tonne cylinders in a 595-m^2 area
- Sludge handling facilities as discussed later under solids management

A revised site plan is shown on Figure J-5 and a flow schematic on Figure J-6.

The environmental impacts associated with the West plant under this alternative are generally as previously described under the current Master Plan with primary treatment (Alternative IA). The major exception is that filling will include approximately 6.5 ha of the Lake Maryout marsh.

The effluent pump station would pump to the Central pump station. The Central pump station would be a dual collection and outfall pump station. That is, it would collect flow from the central zone and pump to the West plant, and it would take West plant effluent and pump it through the Kait Bey outfall.

Functional design criteria for the onsite effluent pump stations will be essentially the same as outlined in the 1978 Master Plan. Physical layouts will be similar to those shown on Figure I-6 of Appendix I of the 1978 Master Plan Report except as differences in flow rate and heads may dictate. Standby power will be provided by connection to two separate power authority substations. If dual power connections are not available, diesel driven standby power generation will be provided.

Ras El-Soda Plant

The treatment plant at Ras El-Soda for Alternative IB is as discussed in the section on the 1978 Master Plan with primary treatment (Alternative IA). However, a site for the Ras El-Soda pump station and treatment

plant has been chosen that is different from that shown in the 1978 Master Plan Report. Significantly more land (developed only as farmland at present) is potentially available. In addition, it appears that the site would have less problems with ground water than the 1978 Master Plan site. A revised site vicinity map is shown on Figure J-7 and a flow schematic is provided on Figure J-8.

Environmental impacts associated with primary treatment facilities at the undeveloped Ras El-Soda site (Figure J-7) include:

- Permanent loss of agricultural land (41 ha)
- Preemption of alternative development for residential, institutional, or commercial use
- Hazards associated with chlorine storage and shipment
- Odor impacts on nearby residential and institutional areas
- Added noise, dust, and debris due to construction activities

Location of primary treatment facilities at this Ras El-Soda site would result in the permanent loss of 41 ha of agricultural land. However, the Ministry of Housing and Finance has planned to develop housing on the Ras El-Soda site so loss of agricultural use appears probable in any event.

Adverse odor impacts could result at the site due to the proximity of adjacent residential and institutional uses. Designs must provide for odor control in preliminary treatment and sludge dewatering to help mitigate odor problems.

The potential hazards associated with chlorine storage at the site are severe due to the proximity of multifamily residences. Potential problems associated with chlorine transport are expected to be minor since the main access roadway (Mustafa Kamel Street) is paved and relatively uncongested.

Solids Management

Under the sea disposal alternative, the following average annual sludge quantities in the year 2000 would require management:

- West Plant--121,500 kg dry sludge/day
- Ras El-Soda Plant--168,000 kg dry sludge/day

- East Plant--27,300 kg dry sludge/day

All sludge is anticipated to be at least 4 percent solids in the primary sedimentation tank underflow.

Various combinations of vacuum filter dewatering plus composting, drying beds with storage, and anaerobic drying lagoons were considered. For the East plant, expansion of the present drying beds and storage facilities was found to be appropriate. Sufficient land exists at the East plant to support this reliable, simple sludge management method. Details of the system are explained in Appendix E, East Plant Evaluation. Approximately 14 ha of land will be needed at the East plant for the drying beds and storage area. One and one-half years of storage are provided for pathogen decay prior to application to farmland. The users would transport and apply the sludge.

Drying beds and anaerobic drying lagoons were eventually eliminated from consideration for the West and Ras El-Soda plants because of the large land area requirements. For drying lagoons, 141 and 176 ha of land (West and Ras El-Soda plants, respectively) would be needed. The drying bed/storage alternative 66 and 82 ha of land for the West and Ras El-Soda plants, respectively. This amount of land is not available adjacent to either plant, and land acquisition within a reasonable distance (up to 15 km) would be difficult and conflict with other intended land uses.

Sufficient land is available at the two plantsites to accommodate the third sludge management alternative, that of vacuum filter dewatering plus composting. Only 9.4 ha would be needed for the West plant and 12 ha for the Ras El-Soda plant. These amounts of land could be made available at either of the plantsites. However, the future development of land around the Ras El-Soda site dictates offsite composting. A site should be chosen away from future housing development but in close proximity to the plant to minimize sludge transportation costs. A site south of the plant near the Cairo-Alexandria Delta Highway is proposed. Appropriate sites for the drying bed or anaerobic lagoon alternative would likely be more distant than the 10 to 15 km anticipated from the plantsite to the compost site.

Two options for transporting sludge between the Ras El-Soda plant and its associated offsite compost operation were studied. Vacuum filter dewatering can take place either at the treatment plantsite or at the compost site. If the dewatering occurs at the plantsite, the sludge must be hauled 10 to 15 km by truck to the compost site. If the dewatering occurs at the compost site, the sludge must be hauled as a liquid or pumped via pipeline. The preferred method would be pipeline because of the huge volume of liquid which would otherwise require truck transport. The filtrate must be pumped back to the plantsite for treat-

ment. A redundant sludge pipeline would ordinarily be used for filtrate return.

Using trucks to transport dewatered sludge to an offsite compost facility serving the Ras El-Soda Plant was selected as the sludge transport method because:

- It is less prone to operational problems, such as pipe plugging or breaking.
- It is more flexible. If the compost site changes location or if several small sites are desired, the transport system can be readily adjusted.
- It lends itself to staging better than a pipeline (which must initially be installed at full capacity plus 100% redundancy).
- The present worths of the two alternatives are essentially the same.

An advantage of onsite vacuum filter/composting at the West plant is the elimination of sludge transportation, prior to treatment, and its associated costs.

In summary, vacuum filter dewatering plus composting was selected for managing sludge from the West and Ras El-Soda treatment plants. The system entails dewatering by vacuum filters onsite, composting for 40 days (onsite at the West plant and offsite for Ras El-Soda) and curing (storage) for 30 days. Users of the sludge will pick up, transport, and apply the compost to farmlands.

The selected solids management system, that of vacuum filter dewatering and composting, would require the construction or purchase of the following facilities for the West plant:

- Liquid sludge holding tanks--two tanks, each 17 m in diameter and 8 m deep
- Vacuum filters--seven filters, each in a 56-m^2 area
- Compost site--81,000 m^2
- Rotoshredders--three at 2700 m^3/hr
- Front-end loaders--three loaders, each with an 0.5-m^3 bucket

The solids management system for the Ras El-Soda plant would require construction or purchase of the following:

- Liquid sludge holding tanks--two tanks, each 19.5 m in diameter and 9 m deep
- Vacuum filters--11 56-m^2 units including three redundant units
- Transport trucks--six trucks, each 10 m^3
- Compost site--155,000 m^2
- Rotoshredders--three at 2700 m^3/hr

SECONDARY TREATMENT ALTERNATIVE II

Alternative II differs from Alternative IB in that secondary treatment by biological towers would be provided in addition to primary treatment. Effluent would be discharged to the Qala and Weqf drains from two plants sized and located as in Alternative IB. Long sea outfalls would therefore not be required. In other respects, Alternative II is similar to Alternative IB. No secondary treatment plant siting is appropriate at Kait Bey because of the small site area available in comparison to the extensive site area requirements for secondary treatment. The activated sludge secondary treatment facilities at the East plant would remain as in all other options.

The treatment process recommended at both the Ras El-Soda and West plants is depicted on Figure J-9. The process was selected for its reliability, ease of operation, and treatment potential. Two modifications from the traditional biological tower process are employed. The first is recycle of secondary sludge to the tower influent, resulting in the formation of a suspended biomass or mixed liquor that increases the BOD and suspended solids removal. This helps in two ways. It adds to the system biomass, resulting in more conversion of wastewater organic matter into settleable biomass. Secondly, it results in raising the secondary sedimentation biomass concentration to a level (1000 to 2000 mg/l) where flocculation can occur, yielding a cleaner secondary effluent, free of more BOD and suspended solids. Gentle aeration of the secondary sedimentation tank distribution/feed channels aids in this flocculation.

The second variation is the addition of chlorine contact tanks. In conventional biological tower treatment, effluent disinfection is provided in the secondary sedimentation tanks. This is not practical in the proposed system where high sedimentation tank solids concentrations are

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maintained. Provision of separate chlorine contact tanks will result not only in less chlorine used to obtain even better disinfection; it will result in less BOD and suspended solids in the final effluent. This occurs because the chlorine contact tanks provide an opportunity to catch some of the solids which have escaped the secondary sedimentation units. Addition of chlorine makes these solids more settleable. The chlorine contact tanks would be designed for easy cleaning.

The foregoing proposed modifications to the biological tower process should permit 85 to 90 percent BOD and suspended solids removals without using excessively conservative BOD loadings on the biological towers. For planning purposes, effluent BOD and suspended solids concentrations should be anticipated at approximately 55 mg/l. While the 55 mg/l concentration is above that currently permitted by law for discharges to drains, attainment of this effluent quality would certainly greatly improve the current situation. If this alternative were selected for implementation, its selection should be provisioned upon a modification in the law permitting implementation. Construction and initial operation of the treatment facilities could be followed in subsequent years by partial or complete use of the treated effluent in agriculture south of Alexandria, as the need arises.

The discussion which follows does not include preliminary and primary treatment units or chlorine storage and feed systems, as these would be the same as those proposed under Alternative IB. The discussion does mention the effluent discharge facilities, since only one disposal option was evaluated in each case.

Environmental impacts anticipated at the East plant under Alternative II are identical to those outlined for the East plant under Alternative IA.

West Plant

The West plant would take average dry weather flows of 395 Ml/day in the year 2000 from the central and western zones. As previously indicated, primary treatment would generally be as described under Alternative IB. Secondary treatment and solids management would require the following process facilities:

- Biological tower influent pump station--one with firm capacity of 508 m^3/min; eight wet pit mixed flow pumps at 84.7 m^3/min each
- Biological towers--three units 60 m in diameter and a media depth of 8.5 m; total media volume of 72,100 m^3
- Secondary sedimentation tanks--12 units 41 m in diameter with

a 3.5-m side wall depth; total surface area = 15,800 m^2

- Secondary sludge pump station--recycle to tower influent, firm capacity = 84 m^3/min (three screw pumps at 42 Ml/day each); sludge wasting to primary influent, firm capacity of 9 m^3/min (5 centrifugal pumps at 3 m^3/min. each).

- Chlorine contact tanks--eight tanks, each 3 m wide x 120 m long x 3 m avg. depth; total volume of 8640 m^3.

- Effluent pump station--one with a firm capacity of 420 m^3/min; six wet pit mixed flow pumps of 105 m^3/min. capacity each

- Effluent force main--one concrete cylinder force main 2 m in diameter and approximately 6 km in total length discharging to the Umum Drain immediately downstream from the Mex pump station (see Figure J-10)

- Liquid sludge holding tanks--two units 19 m in diameter and 10 m deep; total volume = 5680 m^3

- Belt presses--60 2-m units, with eight redundant units

- Trucking to offsite composting--27 trucks, 9.2 m^3 each; seven redundant trucks

- Composting--one 18.6-ha site south of Lake Maryout within 20 km of the West plantsite; composting as discussed before under primary treatment (Alternative IB) solids management

Figure J-10 shows the plantsite and effluent force main location. An alternative force main location is indicated for further study should the secondary treatment alternative be selected for implementation.

Figure J-11 depicts a preliminary site plan. Note the additional infringment on the Lake Maryout wetlands area. Fill would be provided to reclaim an additional 6.8 ha of wetlands area. The additional fill will not provide nearly sufficient onsite area for composting; thus offsite composting in the Amria area south of Lake Maryout is proposed. No specific site has been selected; however, with the land area available south of the lake, a suitable site should be available within a 20-km distance of the West plant. Compost would be marketed to farmers working the newly reclaimed desert lands. This land is low in organic matter and nutrients.

Environmental impacts from expanding the West plant to secondary treatment under Alternative II are similar to those discussed for the West

plant under Alternative IA. These impacts involve the hazards associated with transport and storage of chlorine and odors from treatment processes and sludge handling systems. Because the West plant would be enlarged and a different effluent disposal point utilized, additional environmental impacts are anticipated. These include:

- Filling of approximately 13 ha of marsh and open water areas of Lake Maryout (as opposed to approximately 1.5 ha of marsh under Alternative IA)
- Discharge of 395 Ml/day of secondary effluent to the Umum Drain
- Impacts associated with constructing 6 km of effluent force main

Filling will result in the loss of less than 1 percent of the shoreline habitat of the Lake Maryout main basin. This area represents a marginal habitat for fisheries and wildlife due to existing water quality and sedimentation problems in the shallow lake. It is likely that reed grass (Phrasmites communis) will become naturally revegetated along the periphery of the proposed fill site. This species rapidly recolonizes disturbed areas. Construction activities will result in localized discoloration and siltation. This can be minimized if sheeting is used to enclose the affected area before filling begins.

Discharge of 395 Ml/day of secondary effluent (55 mg/l BOD and suspended solids) should represent only a minor adverse impact. The effluent discharge would only comprise about 5.6 percent of the total flow out of the Umum Drain. Thus, the contribution of this discharge to the freshwater lens along Dekheila Bay would be minimal. In addition, the discharge point below the Mex pump station is ideal, since flow in the drain is rapid and turbulent. This would permit rapid effluent dispersion and minimize the potential for solids accumulation. The drain discharges to Dekheila Bay west of the Western Harbor breakwater. Thus, the effluent discharge is unlikely to become trapped in the Western Harbor. Land uses along the drain below the Mex pump station are predominantly industrial. Thus, the effluent discharge should not adversely impact any beneficial uses of the drain.

Impacts of constructing the 6-km force main are expected to be substantial since the route traverses congested industrial, transportation-related, and residential areas. The force main crosses the Noubaria Canal and a major railroad. Disturbing major archaeological resources is unlikely. However, the appropriate government agency should be contacted prior to final design for a determination on the presence of archaeological resources.

Ras El-Soda Plant

The Ras El-Soda plant would take average dry weather flows of 585 Ml/day from the eastern zones. As previously indicated, primary treatment would generally be as described under Alternative IB. Secondary treatment and solids management would require the following process facilities:

- Biological tower influent pump station--one with a firm capacity of 719 m^3/min; 11 wet pit mixed flow pumps at 80 m^3/min. each

- Biological towers--five units 58 m in diameter with an 8.5-m media depth; total media volume = 112,000 m^3

- Secondary sedimentation tanks--20 tanks 39 m diameter and 3.5-m side wall depth; total surface area = 23,900 m^2

- Secondary sludge pump station--recycle to tower influent, firm capacity of 118 m^3/min. (four screw pumps at 39 m^3/min. each); sludge wasting to primary influent, firm capacity of 13 m^3/min (five centrifugal pumps at 4.33 m^3/min. each)

- Chlorine contact tanks--nine tanks, each 3.4 m wide, 136 m long with a 3.0 m average depth; total volume = 12,500 m^3

- Liquid sludge holding tanks--three tanks 19 m in diameter and 10 m deep; total volume = 8,500 m^3

- Belt presses--87 2-m units, with 11 of these units being redundant

- Trucking to offsite composting--35 trucks at 9.2 m^3 each, 19 of these units being redundant

- Composting--one 27-ha site near the Delta Highway, within a 15-km haul distance south of the Ras El-Soda plant site; composting as described before under primary treatment plant (Alternative IB) solids management.

Figure J-12 is a location plan for the Ras El-Soda secondary plant and shows the plant discharge to the Weqf collector. This collector and all downstream receiving drains are assumed increased in cross section to convey the peak year 2000 plant effluent flows to Abu Qir Bay via the Tabia pump station. The plant cost estimate, presented subsequently in this appendix, provides for enlargement of the drains and expansion of

the Tabia pump station capacity by 10 m^3/sec. Indicated operations and maintenance costs provide for additional annual pumping costs to lift the plant effluent into Abu Qir Bay.

Figure J-13 depicts a preliminary site plan. The entire site area is required for treatment and sludge dewatering facilities. As such, off-site composting is required. A secondary plant on the Ras El-Soda site will interfere with the proposed Ras El-Soda sewage collector design. Should the secondary treatment alternative be selected, the design of the collectors will need to be revised.

The compost site would be located near the Delta Road to Cairo within a 15-km haul distance south of the plantsite. The compost from this operation would be marketed to the Delta farmers.

Impacts of expanding the Ras El-Soda plant to secondary treatment differ from those described for the Ras El-Soda primary plant under Alternative IB. However, the impacts associated with the actual treatment plant site such as chlorine storage, odors etc., are almost identical. The most important difference is effluent discharge to the drainage system that is pumped to Abu Qir Bay. Discharge of 585 Ml/day to the Weqf, Gharbi, Wastani, and Abu Qir drains will impact a portion of this drainage system. Solids accumulations, anaerobic conditions, and odor problems (primarily hydrogen sulfide) could be possible for a distance of up to 2 to 3 km downstream from the Ras El-Soda plant. However, this should not impact adjacent agricultural land uses. Downstream from this degraded drain section, the water quality in the drain should improve over the next 8 to 9 km in the drainage system. Thus, by the time the effluent reaches the Tabia pump station and Abu Qir Bay there should be little or no adverse impact attributable to the Ras El-Soda plant on Abu Qir Bay.

LAND APPLICATION ALTERNATIVES IIIA and IIIB

These alternatives would reuse the treated Alexandria effluent to irrigate up to 70,000 feddans of new farmland reclaimed from the western desert. Raw sewage would be pumped via a long, multiple barreled force main system to a site 60 km south of Alexandria. The East Treatment Plant would be rehabilitated as with previous alternatives to provide 45 Ml/day capacity. All other flows (945 M/day annual average flow) would be used for irrigation. The West Treatment Plant (now under construction) would be completed to its design 85 Ml/d capacity prior to completion of the irrigation system. Together with the East plant, it would provide early relief from raw sewage discharges now occurring. Upon completion of the land application system, the West plant would be converted to provide industrial pretreatment. It should be noted that

utilizing the West plant to its present design capacity of 85 Ml/day will not provide sufficient capacity for its entire service area during the interim period. Therefore, some pump stations will be required to continue discharging to Lake Maryout until the land application system is completed.

Two differing possibilities for the treatment and irrigation portions of Alternative III were considered. Alternative IIIA would provide wastewater treatment by anaerobic lagoons followed by rapid infiltration into a ground-water storage system. The highly treated water would be withdrawn by a series of wells as needed for irrigation by center pivot mechanisms. Crops grown would be unrestricted by water quality parameters other than dissolved solids. Alternative IIIB would provide wastewater treatment by anaerobic lagoons only. Lagoon effluent after screening would be used for irrigation by center pivot mechanisms. Crops grown would be generally restricted to feed and forage.

The treatment needs for the land application systems are given in Appendix K. However, the alternatives assume continued use of the two existing treatment plants (East and West). The following is a discussion of the East and West Treatment Plants as they would exist under land application alternatives.

East Plant

The East plant would be the same as that proposed for the sea disposal alternative and is fully described in Appendix E. That is, it would be an activated sludge secondary treatment plant with an average dry weather capacity of 45 Ml/day. It would discharge to the Qala Drain after chlorination. A site plan is shown on Figure J-1.

Environmental impacts at the East Plant are identical to those described under Alternative IA.

West Plant

The treatment to be provided at the West plant under these alternatives could be either primary or secondary. Selection of the treatment method depends upon the plant's role and its discharge location. Three options are discussed in the following paragraphs.

Primary Treatment, Separate Land Application Site. One alternative role considered for the West plant is for it to treat the flow from the Industries pump station (42 Ml/d average) plus domestic flow added to bring the total flow to the present plant design capacity of 85 Ml/day.

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The point of effluent discharge from this primary plant would be to a separate land application site south of Lake Maryout.

The possible separate land application site considered, the Nahda extension (see Appendix K), would have to be acquired and reclaimed since it is now a part of Lake Maryout. The rationale for such reclamation is that land is needed for ultimate food production, treatment of sewage is required at a reasonable cost, and wastewater is a fresh water resource that is in short supply and will be even more so in the future. In addition, removal of the discharge otherwise intended for the lake will benefit the lake such that removal of a portion for land reclamation should result in at least an even trade, if not a net benefit.

A vicinity map of this option is shown on Figure J-14. However, since the land is not now scheduled for reclamation by the Ministry of Irrigation, this alternative will be dropped from further consideration at this time. It may be worthy of consideration in the future, however.

Secondary Treatment, Discharge Into Lake Maryout. Another use considered for the West plant is its expansion to a 85 Ml/day secondary plant with effluent discharge to Lake Maryout. This approach would achieve treatment equal to that described under Alternative II. This alternative is not favored because including the West plant effluent in the main Alexandria land treatment plan appears justifiable on the basis of cost benefits plus the environmental benefit of eliminating this indirect sea discharge.

Phased Conversion from Primary Treatment to Industrial Pretreatment. The remaining option, considered to be most viable for further consideration, is to take a phased approach to treatment of sewage from the western zone. That is, complete the construction of the West primary plant and place it in operation. Since the land application option (if ultimately selected) will take some time to design and construct, the operating West plant will provide some interim relief to Lake Maryout. Upon completion of the land application system, the West plant could logically serve as an industrial pretreatment plant, with discharge to the land treatment system.

Not all of the specific industrial users of the pretreatment facility have been identified at this time. Most of the industries within the western zone and the Mex Dekheila area whose wastes require some form of pretreatment could probably utilize the facility. From information presented in the 1978 Master Plan, 22 existing industries have been identified as potential users of the pretreatment facility based on their proximity to the West plant and the limited information on their industrial wastes. These industries are listed in Table J-7 along with estimates of the characteristics of their 1977 wastewater flows.

Preliminary information on toxic substances contained in their wastewater is summarized in Table J-8.

As previously discussed in Appendix D, an industrial wastewater sampling and analysis program is urgently needed to obtain additional information on the quantity and characteristics of the industrial wastes generated by the industries located in Alexandria. Until that information is obtained, projecting the industrial wastewater flows and loads and determining pretreatment needs will be extremely difficult. However, based on the limited data presented in the 1978 Master Plan and the work done during this review, preliminary projections were made of the industrial wastewater flows and loads from the western zone and Mex Dekheila area requiring pretreatment. These preliminary projections are presented in Table J-9. An inspection of this table shows that the anticipated pollutants of concern for pretreatment include solids (total, volatile, and suspended), BOD, and several of the heavy metals or other potentially toxic constituents (e.g., phenols, iron, etc.). The projected grease and oil levels shown in Table J-9 reflect the fact that some grease and oil removal will be done at most of the industrial plants to reduce or eliminate the potential for plugging the conveyance systems transporting the wastes to the West plant.

We recommend that pretreatment at the West plant consist of a chemical precipitation process. The wastewater treatment facilities that would be included in the overall treatment scheme would include mechanically cleaned bar screens, equalization tanks, neutralization facilities, chemical addition and rapid mix facilities, flocculation reactors, and sedimentation tanks. The pretreated effluent from the plant would be conveyed to the proposed West pump station for transmission to the land application site. The chemical sludges generated by the pretreatment process would be dewatered on sand drying beds and disposed of in a sanitary landfill (due to the potential contamination of the sludge by heavy metals and other toxic constituents found in the industrial wastewater). A schematic flow diagram for the prepared pretreatment facility is presented in Figure J-15.

Several modifications of the West plant would be required to implement the industrial pretreatment scheme:

- Replace existing comminutors with mechanically cleaned bar screens (may be done as part of proposed West plant improvements).

- If adequate wastewater flow equalization is not provided at the industrial sources, construct new flow equalization tanks as required.

- If required, construct new neutralization facilities including chemical storage and feed systems and neutralization tanks. (Depending on the characteristics of the waste, the neutralization process might be combined with the chemical precipitation process).

- Construct new precipitant chemical storage and feed facilities and rapid mix tanks. Treatability tests should be conducted to determine the most effective chemical to use on the waste (e.g., lime, ferric chloride, ferric sulfate, etc.) and the dosage required.

- Convert the existing pre-aeration basins into flocculation reactors utilizing turbine type flocculators.

- Construct additional sand drying beds to handle the increased quantity of sludge generated by the pretreatment system.

The West plant, at the time of conversion to a pretreatment plant (July 1988), is estimated to have a salvage value of LE 16.7 million. This salvage value should be charged against the GOFI industrial pretreatment program at the time the plant's role changes.

Environmental Impacts. Environmental impacts associated with the West Treatment Plant under Alternative II depend on whether municipal or industrial flows are treated. Use of the West plant for primary treatment of combined wastes with interim discharge to Lake Maryout would entail the following minor adverse impacts:

- Hazards associated with chlorine storage and transport if effluent disinfection is practiced

- Potential odors from treatment processes and sludge handling systems

- Water quality impacts due to interim effluent discharge to Lake Maryout

The hazards due to chlorine use at the West plant can be mitigated as described for Alternative IB.

Odor impacts are anticipated to be minor since the flows are relatively small (85 Ml/day), and areas downwind from the plant during the prominent north and west winds consist of undeveloped Lake Maryout.

Because the flow input to Lake Maryout is not thoroughly understood at this time, it is difficult to assess the effect of the interim effluent

discharge to the lake. However, with the implementation of the master plan, much of the untreated wastewater now reaching the lake will be given primary or secondary treatment. Therefore, the quality of the lake's water should improve. The secondary impact of nutrient loads on the lake cannot be accurately determined because the nutrient removal capacity of the lake is unknown.

Treatment at both the East and West plants would reduce future pollutant loadings to the lake. However, the response time of the lake will be slow and eutrophication and associated sedimentation problems will likely continue, only at a slow

Few adverse environmental impacts are anticipated at the West plant if it is used for industrial pretreatment. This use would eliminate the potential chlorine hazards and the adverse water quality impacts. Potential odor problems would be no greater when operating the plant for industrial waste pretreatment than when operating it as a combined domestic plus industrial waste treatment facility.

Solids Management

Management of sludge under the land application alternative is required at three locations: the East Treatment Plant, the West Treatment Plant, and the treatment ponds preceding land application. Management of the East and West plant sludges would be by drying bed dewatering followed by storage and ultimately farmland application. Details of this system as they relate to the East and West plants are in Appendices E and F.

Management of sludge from the treatment ponds at the land application site would be by distribution to area farmers. Treatment pond sludge management details are explained in Appendix K.

ESTIMATED COSTS

Capital Costs

Estimated capital costs were computed by using procedures described in Appendix B and are on an end-of-1980 cost basis.

Sea Disposal Alternative IB. As discussed previously, the sea disposal alternative presented in the Master Plan has been changed by (1) updating the flow projections and revising treatment facility sizing accordingly, (2) treating all the central zone flow at the West Treatment Plant, (3) adding primary sedimentation and chlorination to the preliminary treatment units (adjusted for the revised flows), and

(4) dewatering, trucking, and composting the primary solids removed in primary treatment. The capital costs for treatment including the onsite effluent pump stations and the onsite West plant influent pump station, are shown in Table J-10. The cost of the Ras El-Soda pump station at the plantsite is covered under Appendix M, Conveyance System Alternatives.

Secondary Treatment Alternative II. The secondary treatment alternative was added to the comparison between sea disposal and land application to provide a high level of treatment without the enormous expenditure of capital needed for the permanent transport and disposal systems associated with the other two options. This alternative has the disadvantage of expending capital for facilities not needed for either the land application or sea disposal alternatives should they be implemented at a later date. The capital costs for this alternative are shown in Table J-10. Costs shown for the Ras El-Soda plant include capital needed to convey the Ras El-Soda effluent to Abu Qir Bay, including enlargement of the Weqf, Gharbi, Wastans and Abu Qir drains and supplementing the Tabia pumping station. Costs for the onsite Ras El-Soda pump station are provided for in Appendix M, Conveyance Systems.

Costs shown for the West plant provide for effluent pumping to the Umum Drain downstream from the Mex pumping station. Also included are costs for an onsite influent pumping station serving the western zone.

Land Application Alternatives IIIA and IIIB. The estimated capital costs for upgrading the East and West plants under this option are shown in Table J-10. To these costs must be added the land application system costs presented in Appendix K. The total cost of the West plant is not appropriately charged against this alternative because it is intended that the plant be phased out as a domestic sewage treatment plant when the land application system is constructed. The cost of the West plant is therefore shown, together with a salvage value in July 1988 at the time the land application system is estimated to be operable and the West Plant is converted to an industrial pretreatment facility.

Operation and Maintenance Requirements and Costs

Table J-11 through J-15 present the estimated operation and maintenance costs for facilities costed in this appendix.

Sea Disposal Alternative IB. Estimated operation and maintenance costs for the West and Ras El-Soda primary treatment plants are detailed in Tables J-11 and J-12, respectively. The estimated cost of operating the East plant is detailed in Appendix E. Table J-15 summarizes the operation and maintenance cost estimates associated with this alternative.

Estimates for both the Ras El-Soda and West plants provide for effluent pumping. At the Ras El-Soda plant, the effluent pump station pumps directly to the sea, whereas the West plant effluent pump station pumps to the central zone collection pump station which in turn pumps to the sea. The cost for operating and maintaining the central zone collection pump station are presented in Appendix M.

Secondary Treatment Alternative II. The costs for the operation and maintenance of the West and Ras El-Soda secondary treatment plants and their disposal facilities are shown on Tables J-13 and 14, respectively. The costs of operating the East plant are detailed in Appendix E. Table J-15 summarizes operation and maintenance costs.

Land Application Alternatives IIIA and IIIB. The costs for operating and maintaining the land application system are estimated in Appendix K. The land application system would receive, treat, and reuse most of Alexandria's wastewater flows under this alternative. The estimated costs for operating and maintaining the East Treatment Plant (common to all options) are detailed in Appendix E. The estimated costs for operating and maintaining the West plant for 3 years and 6 months (during construction of the land application alternative) are presented in Appendix F. Table J-15 summarizes the estimated cost of operating and maintaining these plants under this alternative.

RECOMMENDED OPTIONS FOR FURTHER CONSIDERATION

The evaluation presented in this appendix shows the capital cost of the secondary treatment to be higher by approximately LE 45 million, or 9.5 percent, than the cost of sea disposal inclusive of effluent conveyance and disposal to the sea. (See Appendices L and M.) The cost for operation and maintenance of the secondary treatment alternative is only approximately 4 percent greater than that for the sea disposal alternative when including effluent conveyance and disposal costs. For these reasons, the secondary treatment alternative with discharge to drains does not carry a significant cost disadvantage when compared to sea disposal. Unfortunately, secondary treatment does have the following disadvantages: (1) it is most sensitive to operation and maintenance difficulties, (2) it is most sensitive to industrial flows resulting from accidental spills of toxic substance, (3) it requires the most expensive pretreatment facilities for industry, and (4) significant additional capital will be required if a later decision is made to implement either a sea disposal system or a land application system. For these reasons, the secondary treatment alternative is not recommended for further consideration.

The three alternatives recommended for further consideration are sea disposal (Alternative IB) and land application (Alternatives IIIA and

IIIB). These alternatives are further evaluated and compared in Chapters 4, 5, and 6.

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Table J-1
EFFLUENT DISCHARGE REQUIREMENTS
(concentrations in effluent, mg/l)[a]

Constituent	Sea[b]	Drain
BOD	390	≤ 40
COD		≤ 30[c]
Suspended Solids	200	≤ 50
Settleable Solids[d]		
Dissolved Solids		
Oil & Grease		
pH		
SO_4		
Chlorine Residual		≥ 0.5

[a]Unless otherwise stated

[b]Estimated from primary treatment removals

[c]The same wastewater sample COD must be higher than the BOD. The values in the law likely result from COD measurements based on the permanganate method of determining COD. This method was in use when the law was drafted. This procedure does not account for sulfite, thiosulfate, polythionate, or the organic matter in wastewater. The method now in use is the dichromate reflux method (Standard Methods, 14th edition, 1975). The law should be updated to reflect the change.

[d]Amount of solids that will settle out of 1000 ml of effluent within a 1-hour period.

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Table J-2
PROJECTED FLOWS
(Ml/d)

	1980[a]				1990				2000			
	Avg. Annual	Avg. Dry Weather	Max. Day Dry Weather	Max. Hour	Avg. Annual	Avg. Dry Weather	Max. Day Dry Weather	Max. Hour	Avg. Annual	Avg. Dry Weather	Max. Day Dry Weather	Max. Hour
Central	135	140	185	220	160	170	220	250	185	195	255	295
Western	130	130	170	205	165	165	215	260	200	200	260	310
Subtotal	265	270	355	425	325	335	435	510	385	395	515	605
Ras El-Soda	185	210	270	310	405	430	560	670	560	585	760	895
Subtotal	450	480	625	735	730	765	995	1,180	945	980	1,275	1,500
Nouzha	0	0	0	0	30	30	40	45	45	45	55	70
Total	450	480	625	735	760	795	1,035	1,225	990	1,025	1,330	1,570

[a]Estimated flows used to interpolate between 1980 and 1990

Table J-3
ASSUMED SEWAGE CHARACTERISTICS

BOD	560 mg/l
COD	1,140 mg/l
Suspended Solids	500 mg/l
Grit	0.112 m^3/Ml/d
Screenings	75 l/1000m^3

Note: Industrial wastewater characterisitcs are discussed in the section titled "Industrial Wastewater Impacts on Treatment/Disposal."

Table J-4
COMPARISON OF ACTIVATED SLUDGE AND
BIOLOGICAL FILTER PROCESSES

	Activated Sludge	Biological Filter
BOD and suspenued solids removal	B	
Ability to take shock loads		
Hydraulic		B
Biological		SB
Sludge quantities		SB
Ease of operation		MB
Capital cost		SB
Operation & maintenance cost		MB
Ability to control odors	SB	
Land consumption		SB

Legend:

SB Slightly better
B Better
MB Markedly better

Table J-5

DESIGN CRITERIA FOR NEW PRIMARY TREATMENT FACILITIES

Item	Design Criteria
General Criteria	
Hydraulic capacity	Total of firm influent pump station capacities plus peak dry weather gravity flows.
Redundancy at peak flows	Operable without bypass with largest unit of a kind out of service except for the influent and effluent pump stations where two pumps are redundant.
Standby power	Sufficient for pumping, screening, grit removal, flotation and disinfection at peak flows.
Buffer zone	Minimum of 300 m, or provide odor control means
Flow metering	Parshall flumes on primary sedimentation tank inlets
New Primary Plants	
BOD removal	25 to 30% assumed
Susp. solids removal	55 to 60% assumed
Bar screens	
Clear space	2 cm or less
Velocity max.	1.2 m/s
Depth, max.	1.5 m
Width of screen	2.0 m
Screening discharge	Wheeled, hydraulic dump trailer
Drainage	Returned to plant flow
Screenings disposal	Landfill
Grit removal	
Process	Aerated grit chambers
Design particle size	0.25 mm at 2.65 specific gravity
Removal equipment	Screw or chain and flight conveyor
Grit washing	With primary effluent
Chamber depth	4 m
Length:width ratio	3:1 to 4:1
Detention at peak flow	3 min.
Air supply	6 l/s per m of tank length
Oil and grease flotation	
Process	Provide as extension to grit chambers
Detention at peak flow	5 min.
Chamber depth	4 m
Chamber width	Same as grit chamber
Chamber length	As required to achieve 5 min. of detention
Air supply	0.04 m^3/m^3 sewage
Skimmings discharge	Trailer-mounted tanks
Skimmings disposal	Landfill

Table J-5 (continued)

DESIGN CRITERIA FOR NEW PRIMARY TREATMENT FACILITIES

Item	Design Criteria
Primary sedimentation	
Process	Rectangular, double basins
Length:width ratio	7:1 to 8.5:1
Length	45 m to 55m
Width	6 m
Surface loading-average	32 m/day to 34 m/day
Detention--avg.	2.25 hrs
Depth	3 m
Weir loading	200 m^3/m/day
Sludge/scum removal	Plastic chain & flight, electric motor driven
Sludge pumping	Recessed impeller, centrifugal pumps
Scum discharge	Positive displacement piston pumps to trailer-mounted tank
Scum disposal	Landfill
Primary Effluent Disinfection	
Process	Chlorination
Dose, max. at peak flow	20 mg/l
Dose, average	15 mg/l
Feed system	Evaporators and chlorinators manually set, automatically paced to flow
Onsite chlorine storage	10 days at average flow, cylinder storage
Chlorine contact	Sea outfall
Primary sludge dewatering	
Process	Mechnical dewatering and windrow composting agricultural use
Dewatering	Coil media vacuum filter
Filter loading (dry basis)	30 kg/m^2/hr
Filter size	56 m^2
Filter operation	16 hr/day
Feed concentration	4% minimum
Cake solids	23% assumed
Conditioning chemical	Polymer, 3 kg/tonne assumed
Dewatered sludge composting	
Process	Aerobic composting, windrow method
Windrow temp. max.	60°C
Compost time	30-40 days (weather dependent)
Windrow turnings	50
Bulking agent	Finished compost
Finished moisture content	30% assumed
Compost storage	30 days

Table J-6

NEW DESIGN CRITERIA FOR SECONDARY TREATMENT FACILITIES

Item	Design Criteria
General Criteria	
Hydraulic capacity	Same as primary treatment, Table J-5
Redundancy at peak flow	Same as primary treatment, Table J-5
Standby power	Same as primary tretement, Table J-5
Buffer zone	Minimum of 300 m or provide odor control around preliminary, primary, and sludge treatment units, no buffer required secondary treatment units.
Flow metering	Parshall flumes on primary sedimentation tank inlets, weirs on chlorine contact tank outfall.
Removal Efficiency	
BOD removal	85 to 90% assumed in primary plus secondary treatment
Susp. solids removal	" " " " " " " "
Preliminary & Primary Treatment	
Unit operation	Bar screening, grit removal, oil and grease flotation and primary sedimentation provided as for primary treatment, Table J-5.
Secondary Treatment System	Biological towers with plastic media packing. Recycle tower and secondary sedimentation tank underflow to provide a suspended biomass, supplementing the fixed biomass on the plastic media.
Biological Tower	
Configuration	Circular, with wastewater applied by rotary distributor
Media depth	8.5 m
BOD loading, avg.	2.3 $kg/m^3/day$
Hydraulic application	55 $l/m^2/min$
Solids concentration	1000 to 2,000 mg/l in tower discharge, obtained by recycling secondary sedimentation tank underflow.
Secondary sedimentation	
Process	Circular, center feed units
Diameter	45 m maximum
Surface loading, avg.	25 m/day average
Depth	3.5 m
Weir loading	260 $m^3/m/day$
Sludge/scum removal	Rapid sludge removal mechanisms equipped with surface scum arms; electric motor drives
Sludge pumping	Centrifugal pumps for wasting sludge to primary influent; screw pumps for returning sludge to biological tower influent
Scum discharge	Pump to primary influent

Table J-6 (continued)
NEW DESIGN CRITERIA FOR SECONDARY TREATMENT FACILITIES

Item	Design Criteria
Secondary Effluent Disinfection	
Process	Chlorination
Dose, max. at peak flow	20 mg/l
Dose, avg.	10 mg/l
Feed system and storage	Same as for primary effluent, see Table J-5
Chlorine contact	Minimum of 20 minutes at peak hourly flow rate in separate concrete basins, each with minimum length-to-width of 40:1; basins to be easily cleaned. Provides for additional BOD and suspended solids removal.
Sludge Dewatering	
Process	Mechanical dewatering and windrow compost, agricultural use
Dewatering	Belt press
Press loading (flow basis)	3.4 m^3/m belt width/hr.
Press loading (dry basis)	137 kg/m/hr.
Press size	2 m
Press operation	16 hr./day
Feed concentration	4%, approximate
Cake solids	25% assumed
Conditioning chemical	Polymer, 4 kg/tonne assumed
Dewatered Sludge Composting	Same as for primary sludge, see Table J-5

TABLE J-7

POTENTIAL INDUSTRIAL USERS OF WEST PLANT PRETREATMENT FACILITY

INDUSTRY LOCATION NUMBER[a]	NAME OF INDUSTRY	LOCATION OF INDUSTRY	TOTAL FLOW (m^3/d)	TOTAL SOLIDS (kg/d)	VOLATILE SOLIDS (kg/d)	SUSPENDED SOLIDS (kg/d)	BOD (kg/d)	COD (kg/d)	TKN (kg/d)	GREASE AND OIL[b] (kg/d)
8-1	Nile Match Co	Manshiya	550	277	53	31	66	103	11	—
8-2	Nile Match Co	Moharrem Bey	400	200	38	22	48	75	8	—
25-1	National Paper Co	Moharrem Bey	3,000	4,170	2,200	—	1,800	6,000	—	—
27	Alexandria Petroleum Co	Mex	800	5,200	750	77	420	840	27	80
29	Labbon Power Station and Gas Works	Karmouz	2,075	—	—	200	—	200	—	208
37-1	Alexandria Oil and Soap Co	Karmouz	130	16,200	2,330	2,580	1,360	3,200	124	13
37-2	Alexandria Oil and Soap Co	Karmouz	290	72,000	19,300	5,500	8,400	9,000	58	29
37-3	Alexandria Oil and Soap Co	Moharrem Bey	85	—	—	—	332	664	—	9
38-1	Egyptian Salt and Soda Co	Karmouz	380	—	—	—	438	875	—	38
38-2	Egyptian Salt and Soda Co	Moharrem Bey	850	28,400	16,500	—	8,680	26,400	—	85
39-1	Extracted Oils Co	Moharrem Bey	800	—	—	—	3,900	7,800	—	80
39-2	Extracted Oils Co	Moharrem Bey	242	—	—	—	340	680	—	24
39-3	Extracted Oils Co	Moharrem Bey	70	—	—	—	24	48	—	12
39-4	Extracted Oils Co	Gabbary	110	—	—	—	110	220	—	11
39-5	Extracted Oils Co	Gabbary	75	—	—	—	60	120	—	8
40-2	Egyptian Starch and Yeast Co	Moharrem Bey	832	6,500	8,000	8,400	3,250	800	26	—
40-3	Egyptian Starch and Yeast Co	Moharrem Bey	2,970	7,830	3,132	1,250	1,100	403	655	—
41-1	El Nasr Leather Tanning Co	Mex	880	9,530	2,220	1,215	2,200	3,290	144	—
41-2	El Nasr Tanning Co	Mex	200	—	—	—	20	40	—	—
42	Private Tanneries	Mex	210	—	—	—	422	630	—	—
45	Slaughterhouse	Mex	570	2,550	1,983	850	1,700	2,270	3	57
54-2	National Spinning and Weaving Co	Karmouz	9,050	—	—	—	3,400	4,400	—	—

Source: 1978 Master Plan.

[a]See Figures C-1, C-2, and C-4 of 1978 Master Plan, Appendix C, for location of the industries.

[b]Values shown reflect some in-plant grease and oil removal by the industries prior to discharge to the West plant.

TABLE J-8

CURRENT (1977) TOXIC SUBSTANCES INDUSTRIAL LOAD (kg/d)

INDUSTRY LOCATION NUMBER[a]	NAME OF INDUSTRY	IRON	CHROMIUM	MANGANESE	ZINC	LEAD	NICKEL	COPPER	PHENOL
8-1	Nile Match Co., Manshiya	0.40	0.66	0.50	0.77	•	•	•	•
8-2	Nile Match Co., Moharrem Bey	0.27	0.50	0.36	0.56	•	•	•	•
25-1	National Paper Co.	9.00	•	1.00	3.00	0.30	•	0.20	•
27	Alexandria Petroleum Co.	•	•	•	•	•	•	•	8.36
37-1	Alexandria Oil and Soap Co.	16.00	•	1.50	9.00	2.60	5.00	•	•
37-2	Alexandria Oil and Soap Co.	8.00	•	•	8.00	1.50	6.00	•	•
38-2	Egyptian Salt and Soda Co.	16.30	•	6.50	6.20	2.20	•	•	•
41-1	El Nasr Leather Tanning Co.	3.40	70.00	•	1.50	•	•	•	•
42	Private Tanners	•	8.96	•	•	•	•	•	•
	TOTAL	50.37	80.08	9.66	29.03	6.60	11.00	0.20	8.36

Source: 1978 Master Plan

[a]See Figures C-1, C-2 and C-4 of 1978 Master Plan, Appendix C, for location of the industries.

Table J-9
PRELIMINARY PROJECTIONS OF THE WEST PLANT PRETREATMENT FACILITY
INDUSTRIAL FLOWS AND LOADS

Parameter	Average Concentration (mg/d)	1990 Value	2000 Value
Average Flow	--	41.0 Ml/d	53.0 Ml/d
Peak Flow[a]	--	61.5 Ml/d	80.0 Ml/d
Total Solids	6,000	246,000 kg/d	318,000 kg/d
Volatile Solids	2,000	82,000 kg/d	106,000 kg/d
Suspended Solids	800	32,800 kg/d	42,400 kg/d
BOD_5	1,500	61,500 kg/d	79,500 kg/d
COD	3,000	123,000 kg/d	159,000 kg/d
TKN	40	1,640 kg/d	2,120 kg/d
Grease/Oil	30	1,230 kg/d	1,590 kg/d
Iron	2.0	80 kg/d	110 kg/d
Chromium	3.0	120 kg/d	160 kg/d
Manganese	0.4	16 kg/d	20 kg/d
Zinc	1.0	40 kg/d	50 kg/d
Lead	0.3	12 kg/d	16 kg/d
Nickel	0.4	16 kg/d	20 kg/d
Phenol	0.4	16 kg/d	20 kg/d

[a]Peak to average ratio assumed to be 1.5:1 to reflect flow equalization at the industrial source or West plant.

Table J-10
ESTIMATED CAPITAL COSTS FOR TREATMENT
(10^6 LE, end-of-1980 prices)

Alternative	Initial Construction	1990 Construction
Sea Disposal (Alt. IB)		
Exist. 85 Ml/d West Plant Upgrade	1.1	0
West Plant Expansion	42.3	0
Ras El-Soda Plant	53.4	7.6
East Plant Upgrade	11.2	0
Subtotal	108.0	7.6
Contingencies @ 25%	27.0	1.9
Subtotal	135.0	9.5
Engineering, Legal, and Administrative costs @ 15%	20.0	1.4
Land	0.6	0
Total	155.6	10.9
Secondary Treatment (Alt. II)		
Exist. 85 Ml/day West Plant Upgrade[a]	1.1	0
West Plant Expansion & Secondary Treatment	103.3	4.4
Ras El-Soda Secondary Plant	129.5	19.7
East Plant Upgrade	11.2	0
Subtotal	245.1	24.1
Contingencies @ 25%	61.3	6.0
Subtotal	306.4	30.1
Engineering, Legal, and Administrative costs @ 15%	46.0	4.5
Land	1.8	0
Total	354.2	34.6

[a] Includes facilities for plant upgrade recommended in Appendix F, West Treatment Plant Evaluation, except that costs for upgrade and expansion of the sludge drying beds are not included. Sludge will be mechanically dewatered and composted. Costs for these facilities are included under West Plant Expansion (Alt. II).

Table J-10 (continued)
ESTIMATED CAPITAL COSTS FOR TREATMENT
(10^6 LE, end-of-1980 prices)

Alternative	Initial Construction	1990 Construction
Land Application (Alt. III)[b]		
Exist. 85 Ml/d West Plant Upgrade[c]	10.0	0
East Plant Upgrade	11.2	0
Subtotal	21.2	0
Contingencies @ 25%	5.3	0
Subtotal	26.5	0
Engineering, Legal, and Administrative costs @ 15%	4.0	0
Land	0	0
Total	30.5	0

[b]Cost of West and East plants only. Does not include cost of land application facilities or pretreatment at land application site. Cost of land application facilities, including onsite pretreatment, is set forth in Appendix K, Land Application of Wastewater.

[c]Includes all facilities for plant upgrade recommended in Appendix F, West Treatment Plant Evaluation. After construction of the land application system (completion estimated by July 1988), the West plant will become an industrial pretreatment facility. Its salvage value at that time is estimated at LE 16.7 million (end-of-1980 prices). This amount will be charged against industrial pretreatment and credited to Alternative III, Land Application System, in subsequent present worth analyses.

Table J-11
SUMMARY OF ESTIMATED WEST PLANT ANNUAL O&M COSTS--
SEA DISPOSAL OPTION
(end-of-1980 prices)

Item	1980[a] Quantity	1980[a] Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	28,850		33,265		33,365	
Cost		188		217		230
Power						
1,000,000 kWh	11		14		16	
Cost		98		125		146
Fuel						
1000 litres	193		236		280	
Cost		7		8		10
Chemicals						
Tonnes	1,690		2,074		2,458	
Cost		1,584		1,942		2,285
Materials						
Cost		246		285		320
Total						
Cost		2,123		2,577		2,991

[a]Annual O&M costs indicated for 1980 as basis for estimating subsequent O&M costs by interpolation between 1980 and 1990.

Table J-12
SUMMARY OF ESTIMATED RAS EL-SODA PLANT ANNUAL O&M COSTS--
SEA DISPOSAL OPTION
(end-of-1980 prices)

Item	1980[a] Quantity	1980[a] Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	21,600		42,190		50,355	
Cost		140		275		327
Power						
1,000,000 kWh	7		15		21	
Cost		60		126		178
Fuel						
1000 Litres	303		663		917	
Cost		11		24		33
Chemicals						
Tonnes	1,208		2,597		3,527	
Cost		1,083		2,353		3,230
Materials						
Cost		246		383		467
Total						
Cost		1,540		3,161		4,235

[a]Annual O&M cost indicated for 1980 as basis for estimating subsequent years' O&M costs by interpolation between 1980 and 1990 estimated annual cost.

Table J-13

SUMMARY OF ESTIMATED ANNUAL O&M COSTS AT THE WEST SECONDARY TREATMENT PLANT--
ALTERNATIVE II
(end-of-1980 prices)

Item	1980[a] Quantity	1980[a] Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	46,500		53,600		60,000	
Cost		302		348		390
Power						
1,000,000 kWh	17.2		21.6		26.1	
Cost		152		191		231
Fuel						
1000 litres	583		715		833	
Cost		20		25		29
Chemicals						
Tonnes	1,310		1,603		1,882	
Cost		1,436		1,762		7,067
Materials						
Cost		503		564		625
Total						
Cost		2,413		2,890		3,342

[a]Annual O&M cost indicated for 1980 as basis for estimating subsequent years' O&M costs by interpolation between 1980 and 1990 estimated annual costs.

Table J-14

SUMMARY OF ESTIMATED ANNUAL O&M COSTS AT THE RAS EL-SODA SECONDARY TREATMENT PLANT--ALTERNATIVE II

(end-of-1980 prices)

Item	1980[a] Quantity	1980[a] Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	37,200		62,400		82,800	
Cost		242		406		538
Power						
1,000,000 kWh	13.1		26.3		35.2	
Cost		116		232		311
Fuel						
1000 Litres	418		860		1,182	
Cost		15		30		41
Chemicals						
Tonnes	972		2,010		2,743	
Cost		1,259		2,254		3,028
Materials						
Cost		439		715		905
Total						
Cost		2,071		3,637		4,823

[a]Annual O&M cost indicted for 1980 as basis for estimating subsequent years' O&M costs by interpolation between 1980 and 1990 estimated annual costs.

Table J-15
SUMMARY OF OPERATION AND MAINTENANCE COSTS
FOR TREATMENT

Option	1980	1990	2000
Sea Disposal--Alt.IB			
West Plant	2,123	2,577	2,991
Ras El-Soda Plant	1,540	3,161	4,235
East Plant	870	870	870
Total	4,533	6,608	8,096
Secondary Treatment--Alt.II			
West Plant	2,413	2,890	3,342
Ras El-Soda Plant	2,071	3,637	4,823
East Plant	870	870	870
Total	5,354	7,397	9,035
Land Application--Alt.III[a]			
East Plant	870	870	870
West Plant[b]	540	540	540

Note: Estimated end-of-1980 prices (1000 LE) using cost curves adjusted for local conditions.

[a]Costs for treatment of the land application site are in Appendix K.

[b]Costs for operating the West plant at 85 Ml/d for 3.5 years are included in the present worth analysis.

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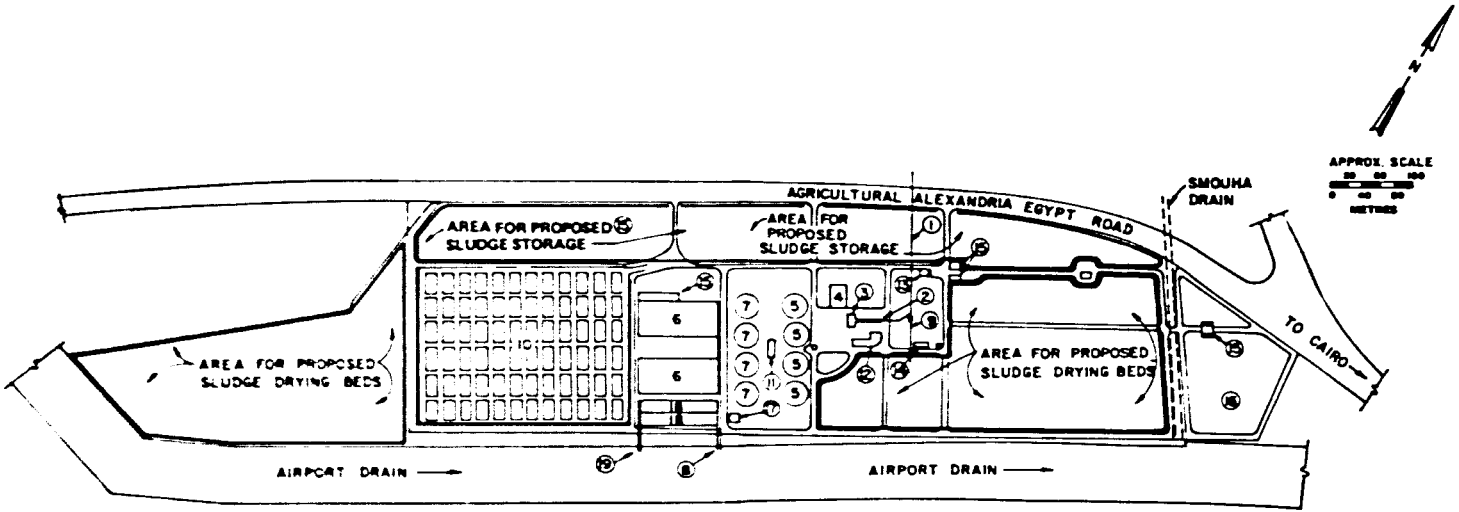


LEGEND:

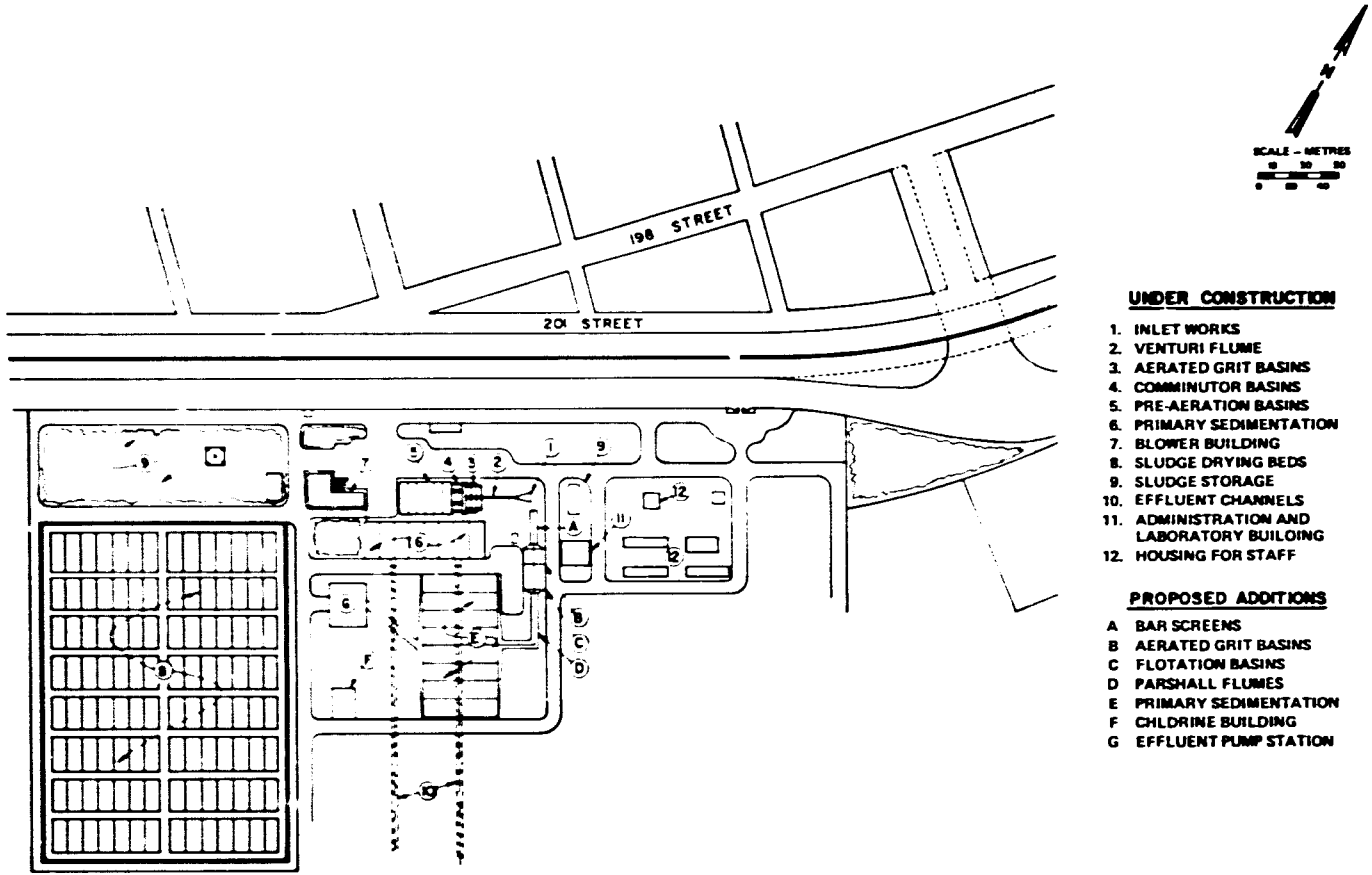
1. INFLUENT FORCE MAIN
2. INLET WORKS
3. AERATED GRIT BASIN
4. PRE AERATION BASINS
5. PRIMARY SEDIMENTATION
6. AERATION BASINS
7. SECONDARY SEDIMENTATION
8. EFFLUENT LINE, EXISTING
9. BYPASS TRAIN
10. SLUDGE DRYING BEDS
11. PUMP HOUSE
12. MACHINERY BUILDING
13. LABORATORY AND ADMINISTRATION BUILDING
14. INDUSTRIAL LAB
15. STORAGE BUILDING
16. HOUSING
17. CHLORINE BUILDING, PROPOSED
18. CHLORINE CONTACT BASIN, PROPOSED
19. EFFLUENT LINE, PROPOSED

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-1

EAST TREATMENT PLANT SITE PLAN (ALL ALTERNATIVES)

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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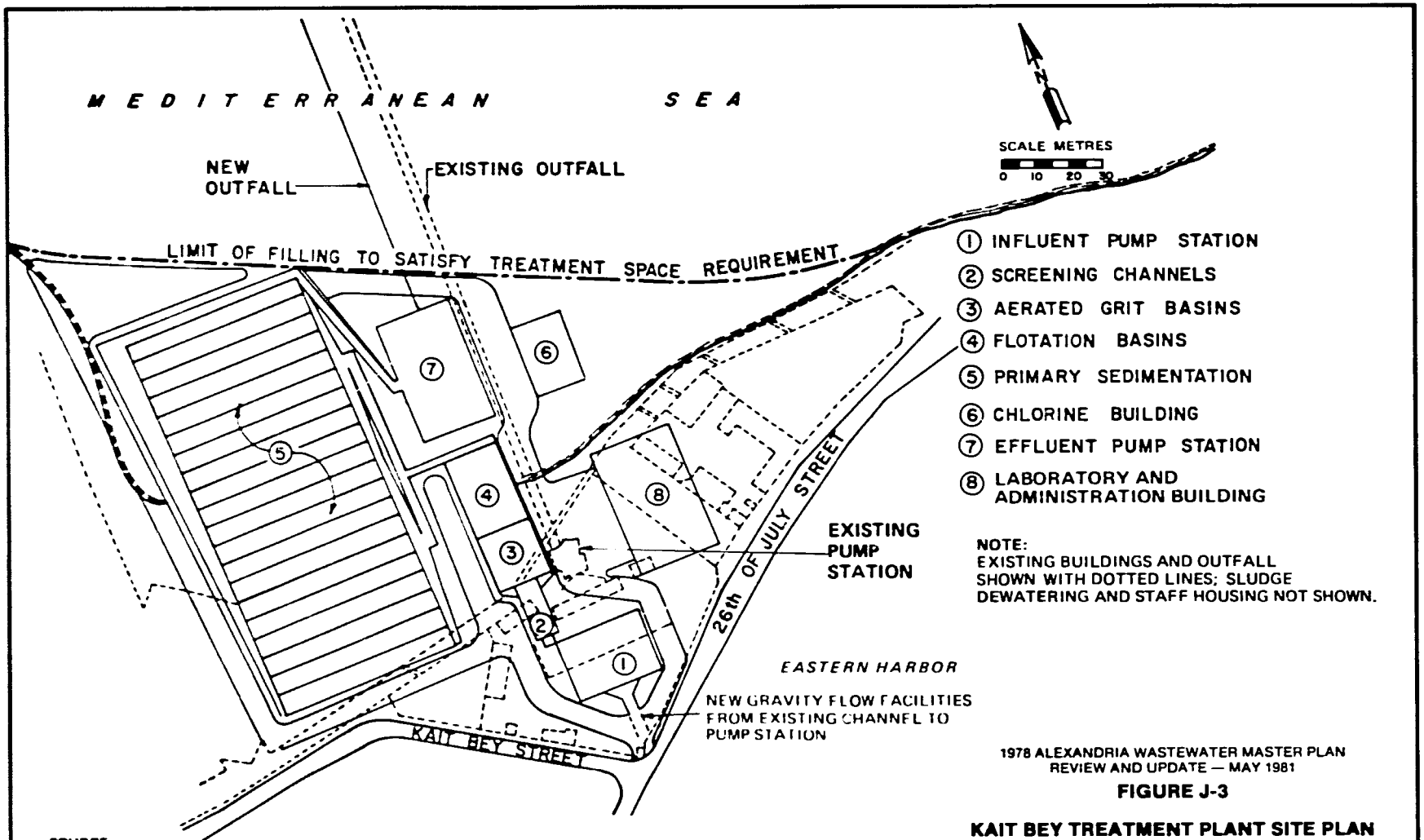
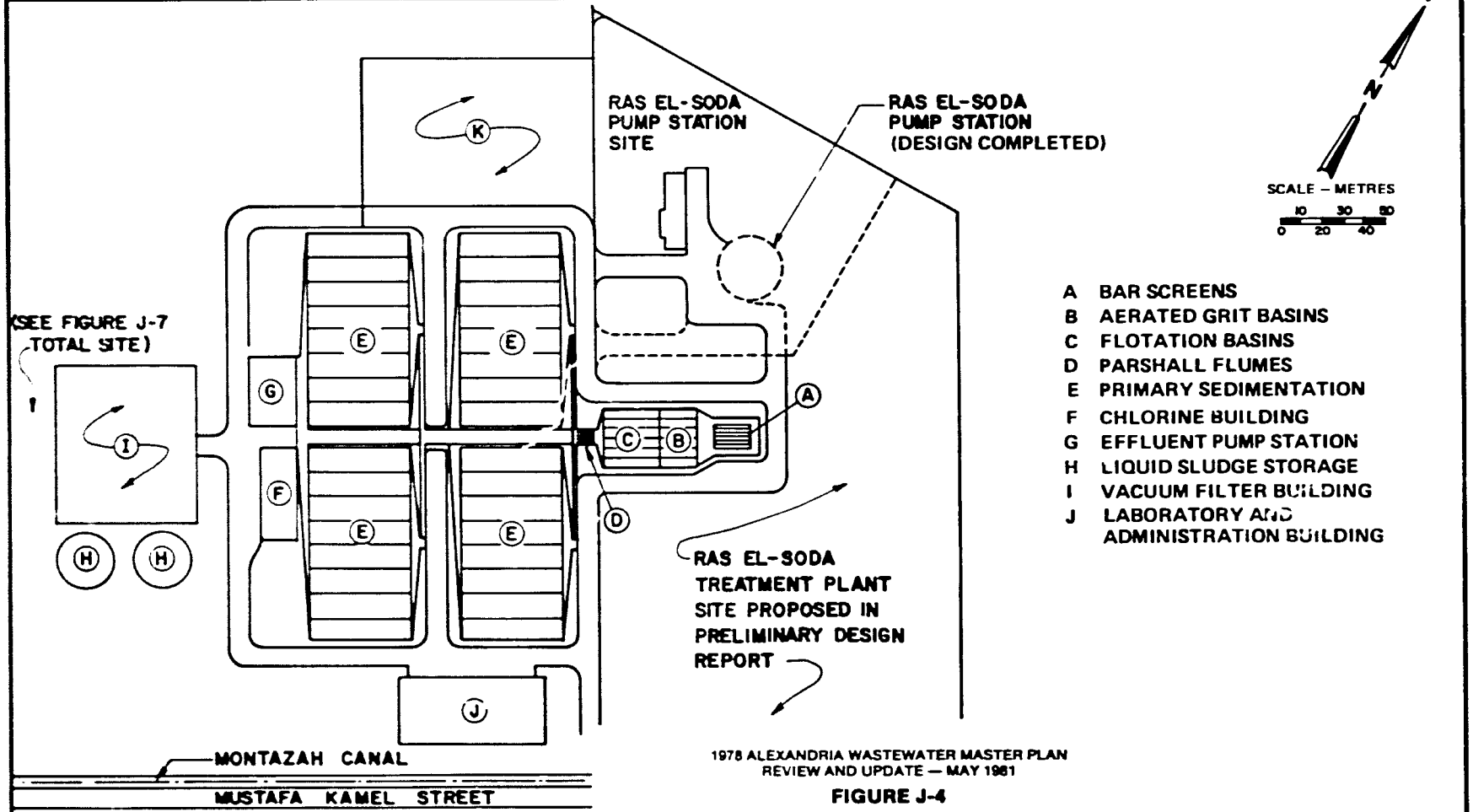
FIGURE J-2

WEST TREATMENT PLANT SITE PLAN (ALTERNATIVE IA)

A/GOSD WWCG W/ECG


MEDITERRANEAN SEA
NEW OUTFALL
EXISTING OUTFALL
LIMIT OF FILLING TO SATISFY TREATMENT SPACE REQUIREMENT
SCALE METRES
0 10 20 30
① INFLUENT PUMP STATION
② SCREENING CHANNELS
③ AERATED GRIT BASINS
④ FLOTATION BASINS
⑤ PRIMARY SEDIMENTATION
⑥ CHLORINE BUILDING
⑦ EFFLUENT PUMP STATION
⑧ LABORATORY AND ADMINISTRATION BUILDING
EXISTING PUMP STATION
26th OF JULY STREET
EASTERN HARBOR
NEW GRAVITY FLOW FACILITIES FROM EXISTING CHANNEL TO PUMP STATION
KAIT BEY STREET
NOTE:
EXISTING BUILDINGS AND OUTFALL SHOWN WITH DOTTED LINES; SLUDGE DEWATERING AND STAFF HOUSING NOT SHOWN.
SOURCE:
1978 MASTER PLAN
(FOR EXISTING STRUCTURES)
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981
FIGURE J-3
KAIT BEY TREATMENT PLANT SITE PLAN
(ALTERNATIVE IA)
A/GOSD
WWCG W/ECG

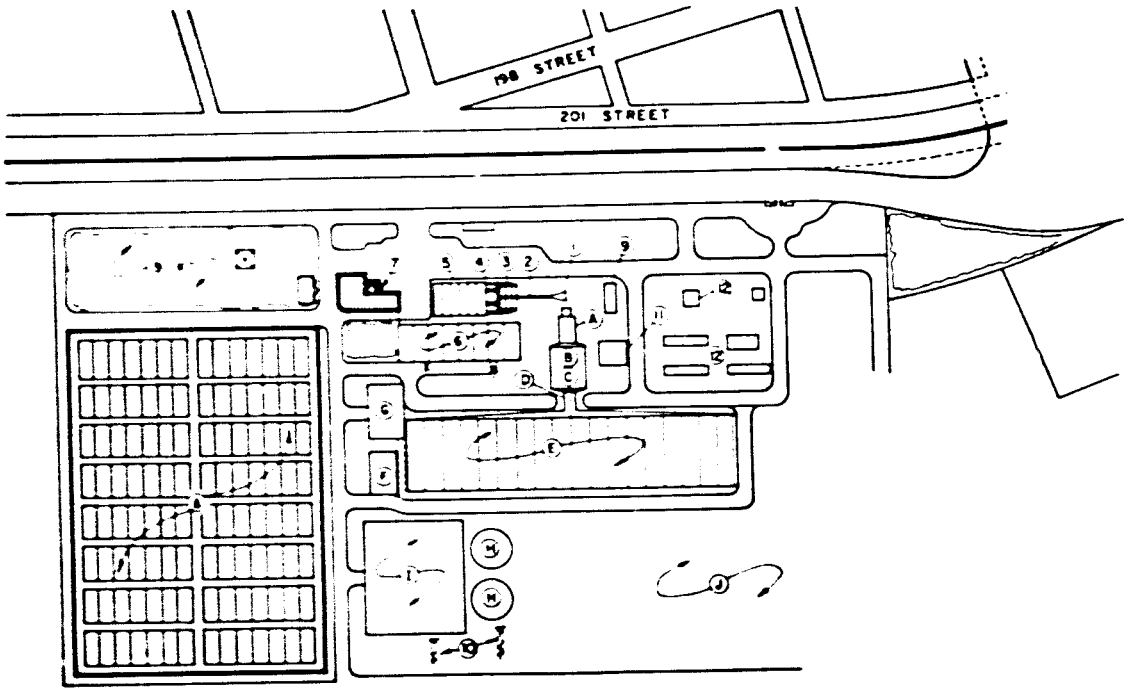


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-4

RAS EL-SODA TREATMENT PLANT SITE PLAN (ALTERNATIVES IA AND IB)

A/GOSD WWCG W/ECG



UNDER CONSTRUCTION

1. INLET WORKS
2. VENTURI FLUME
3. AERATED GRIT BASINS
4. COMMINUTOR BASINS
5. PRE-AERATION BASINS
6. PRIMARY SEDIMENTATION
7. BLOWER BUILDING
8. SLUDGE DRYING BEDS
9. SLUDGE STORAGE
10. EFFLUENT CHANNELS
11. ADMINISTRATION AND LABORATORY BUILDING
12. HOUSING FOR STAFF

PROPOSED ADDITIONS

A BAR SCREENS
B AERATED GRIT BASINS
C FLOTATION BASINS
D PARSHALL FLUMES
E PRIMARY SEDIMENTATION
F CHLORINE BUILDING
G EFFLUENT PUMP STATION
H LIQUID SLUDGE STORAGE
I VACUUM FILTERS
J COMPOST AREA
K COMPOST STORAGE

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-5

WEST TREATMENT PLANT SITE PLAN (ALTERNATIVE IB)

A/GOSD WWCG W/ECG

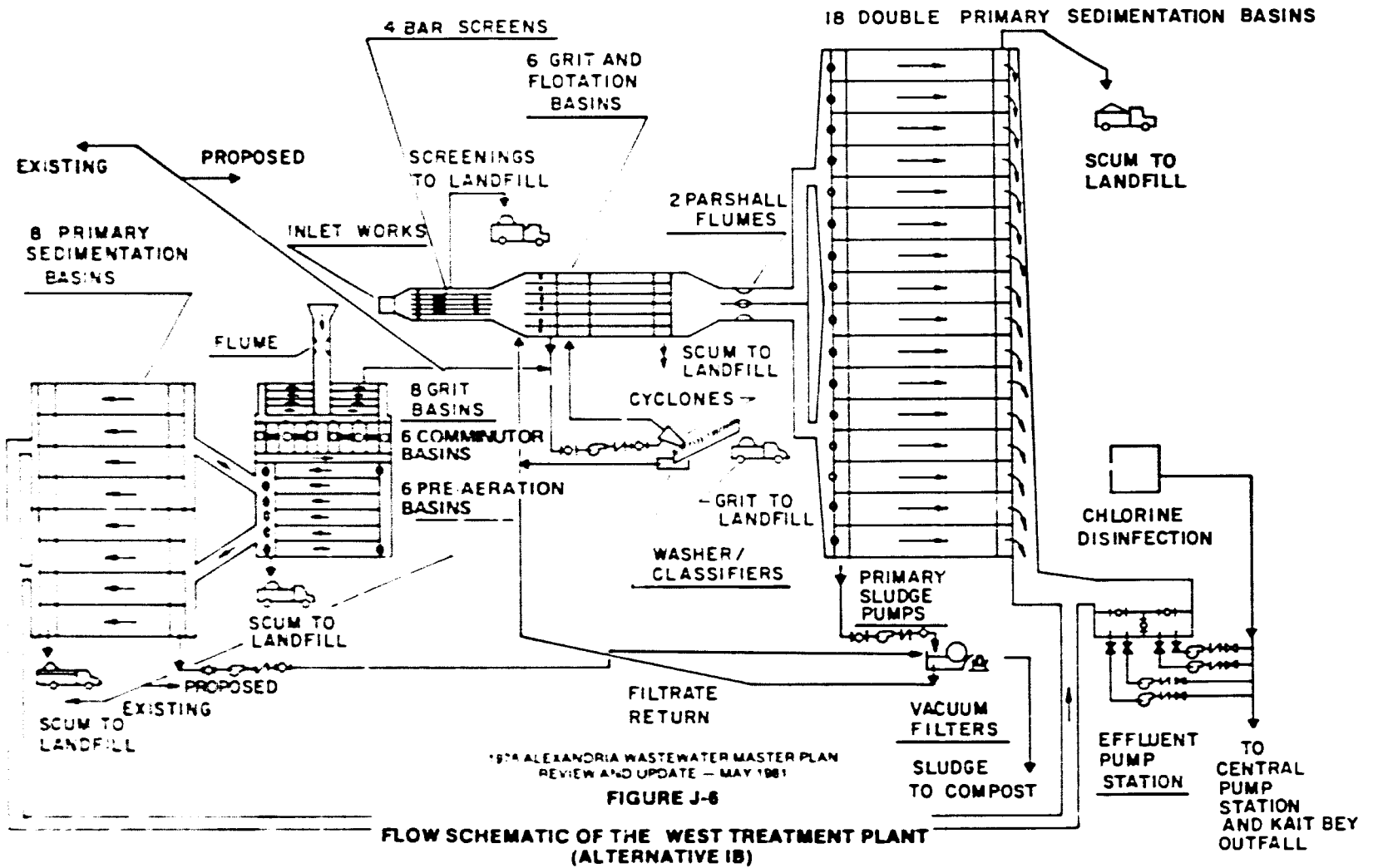
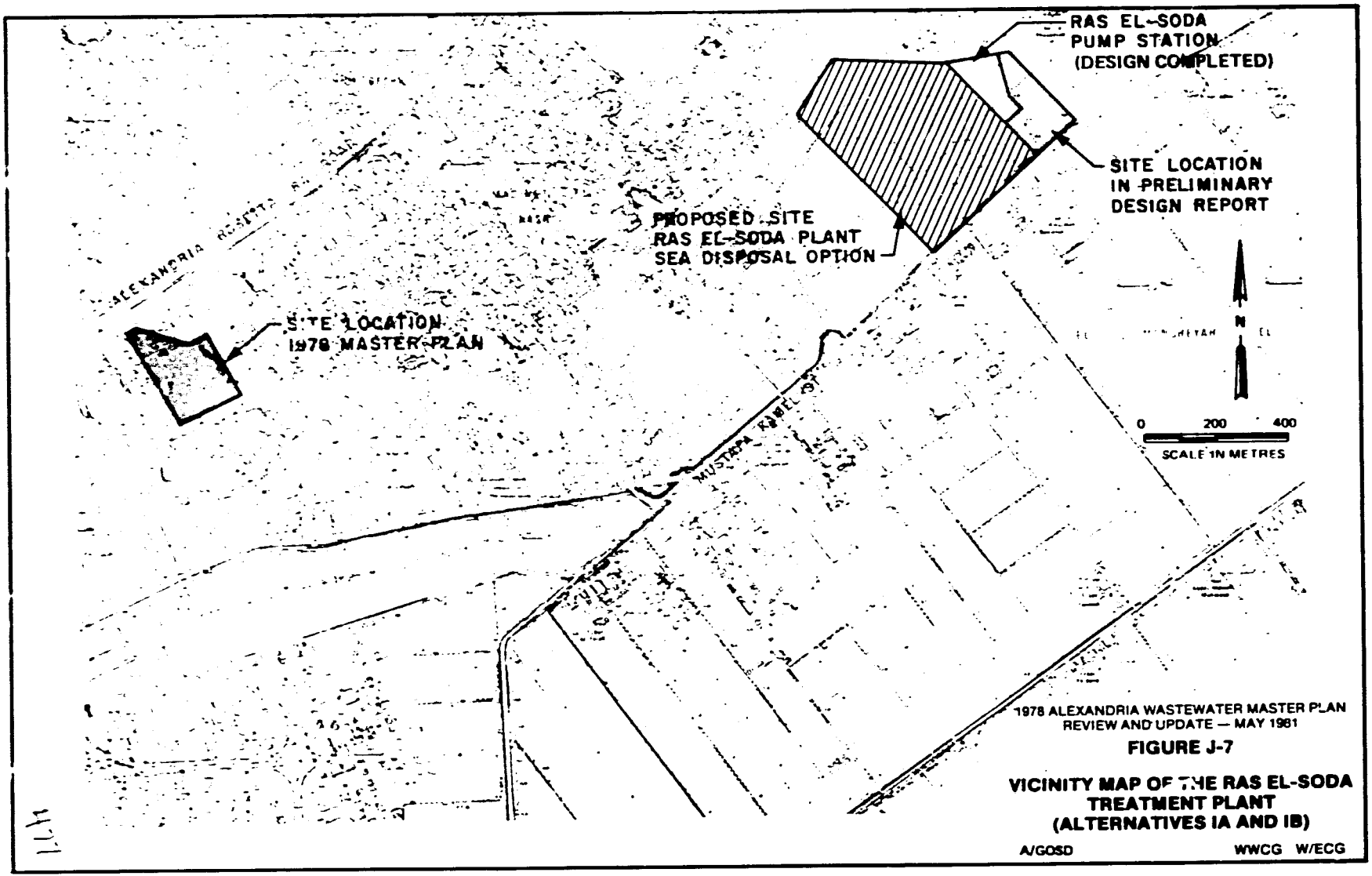
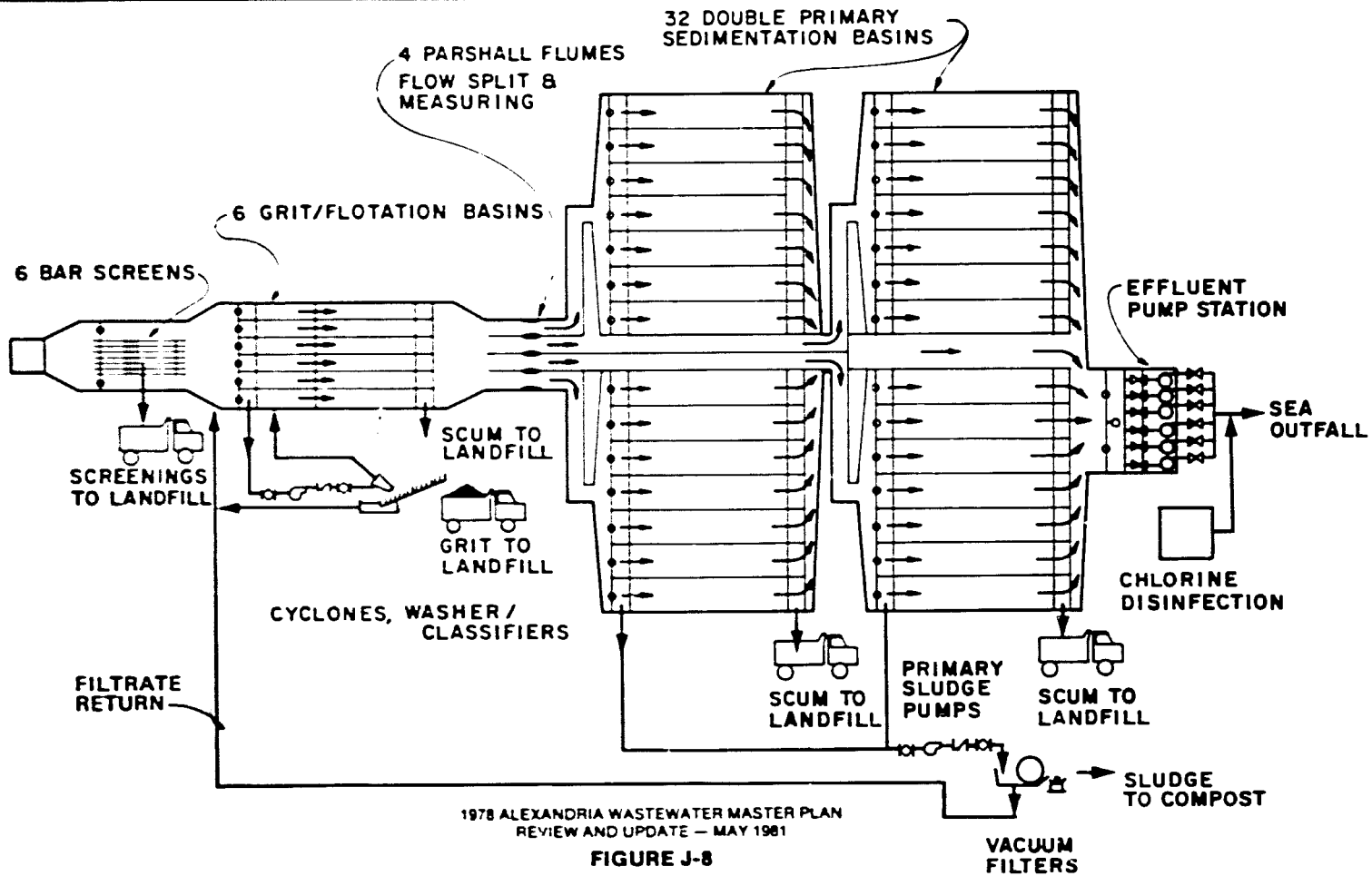


FIGURE J-6

FLOW SCHEMATIC OF THE WEST TREATMENT PLANT (ALTERNATIVE IB)


RAS EL-SODA
PUMP STATION
(DESIGN COMPLETED)
SITE LOCATION
IN PRELIMINARY
DESIGN REPORT
PROPOSED SITE
RAS EL-SODA PLANT
SEA DISPOSAL OPTION
SITE LOCATION
1978 MASTER PLAN
ALEXANDRIA
MUSTAFA KAMEL ST
N
0
200
400
SCALE IN METRES
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981
FIGURE J-7
VICINITY MAP OF THE RAS EL-SODA
TREATMENT PLANT
(ALTERNATIVES IA AND IB)
A/GOSD
WWCG W/ECG

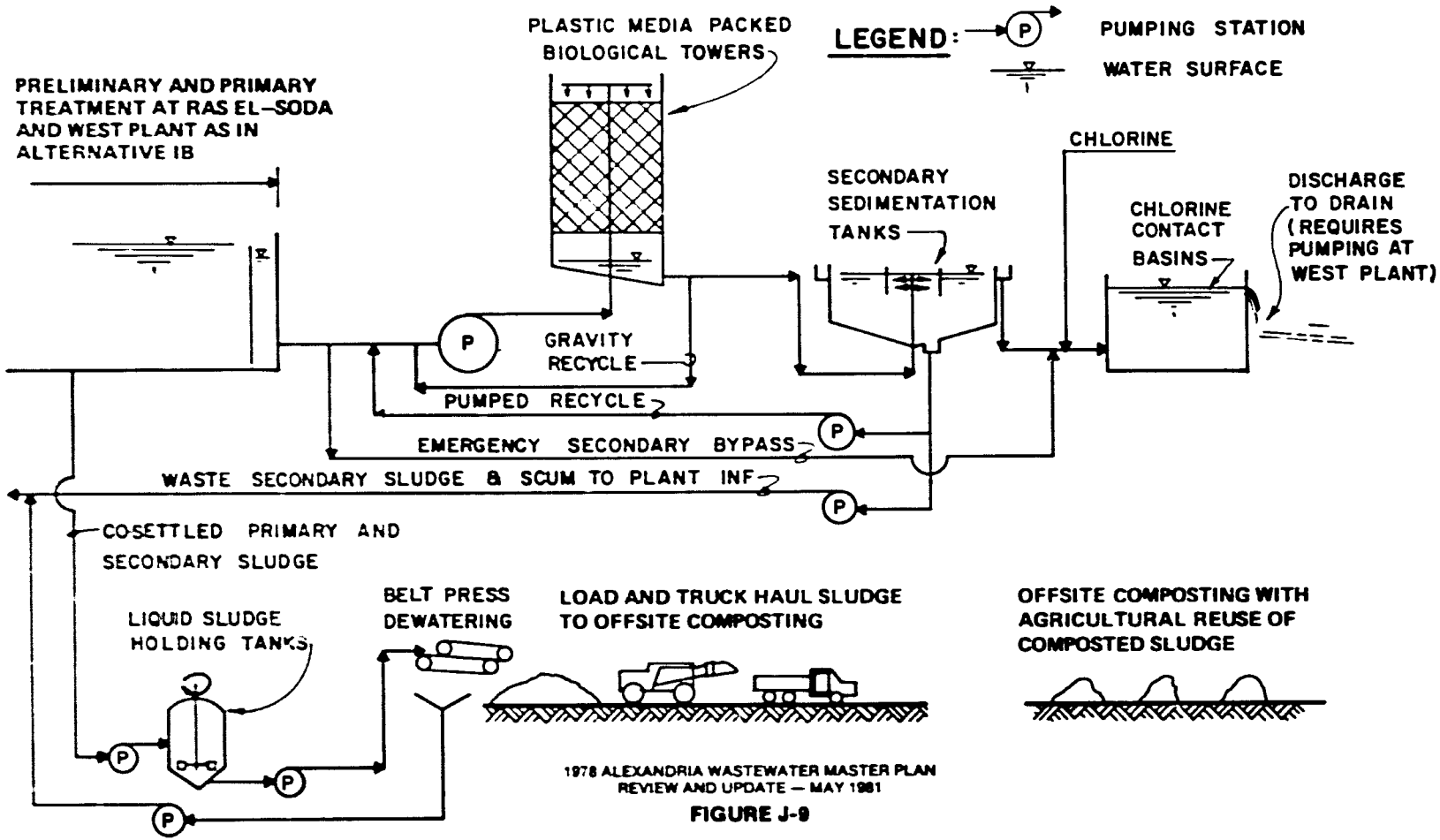


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-8

FLOW SCHEMATIC OF RAS EL-SODA TREATMENT PLANT WITH SEA DISPOSAL (ALTERNATIVE IB)

A/GOSD WWCG W/ECG

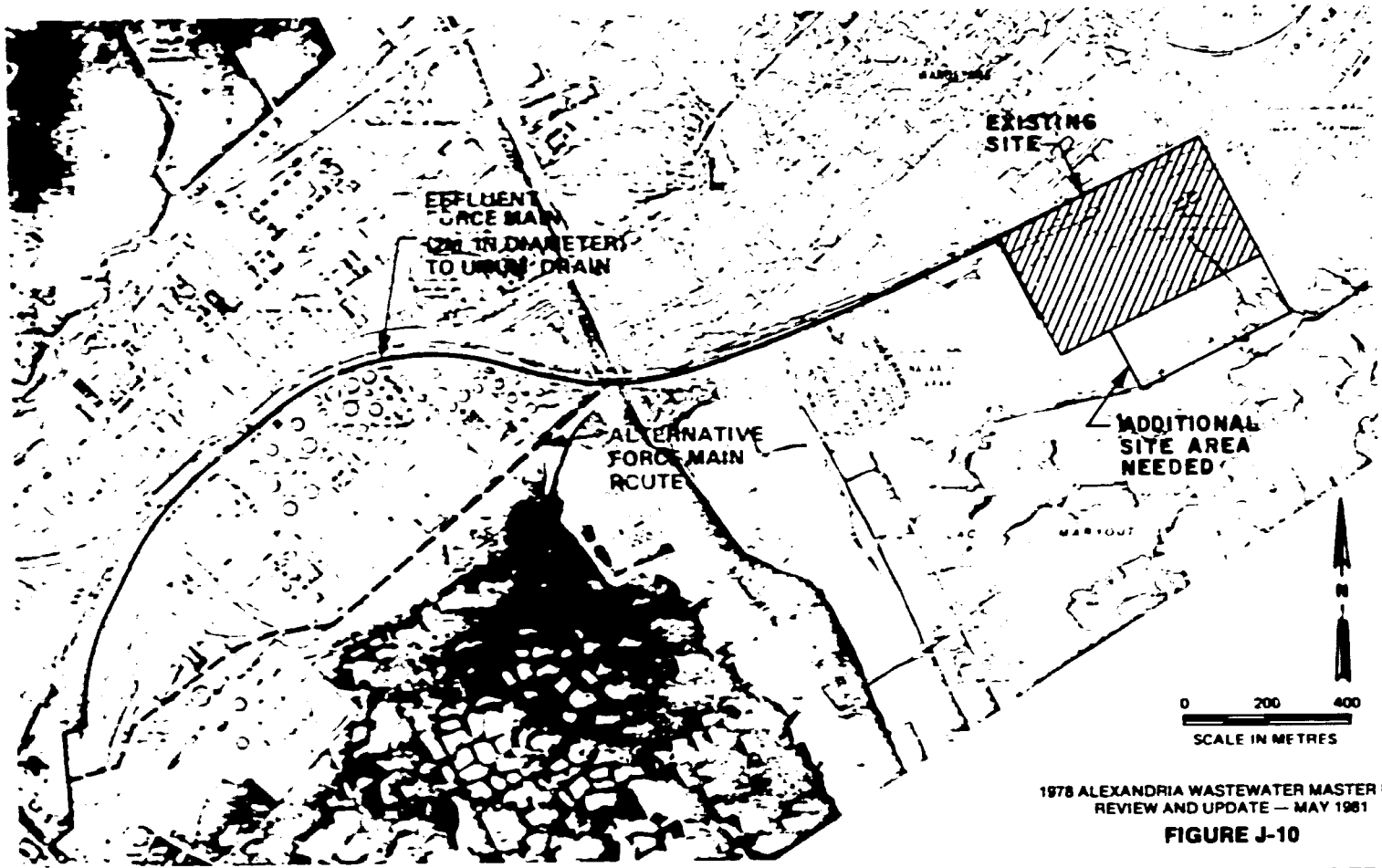


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-9

TREATMENT PROCESS SCHEMATIC OF SECONDARY TREATMENT AND DISCHARGE TO DRAINS

A/GOSD WWCG W/ECG

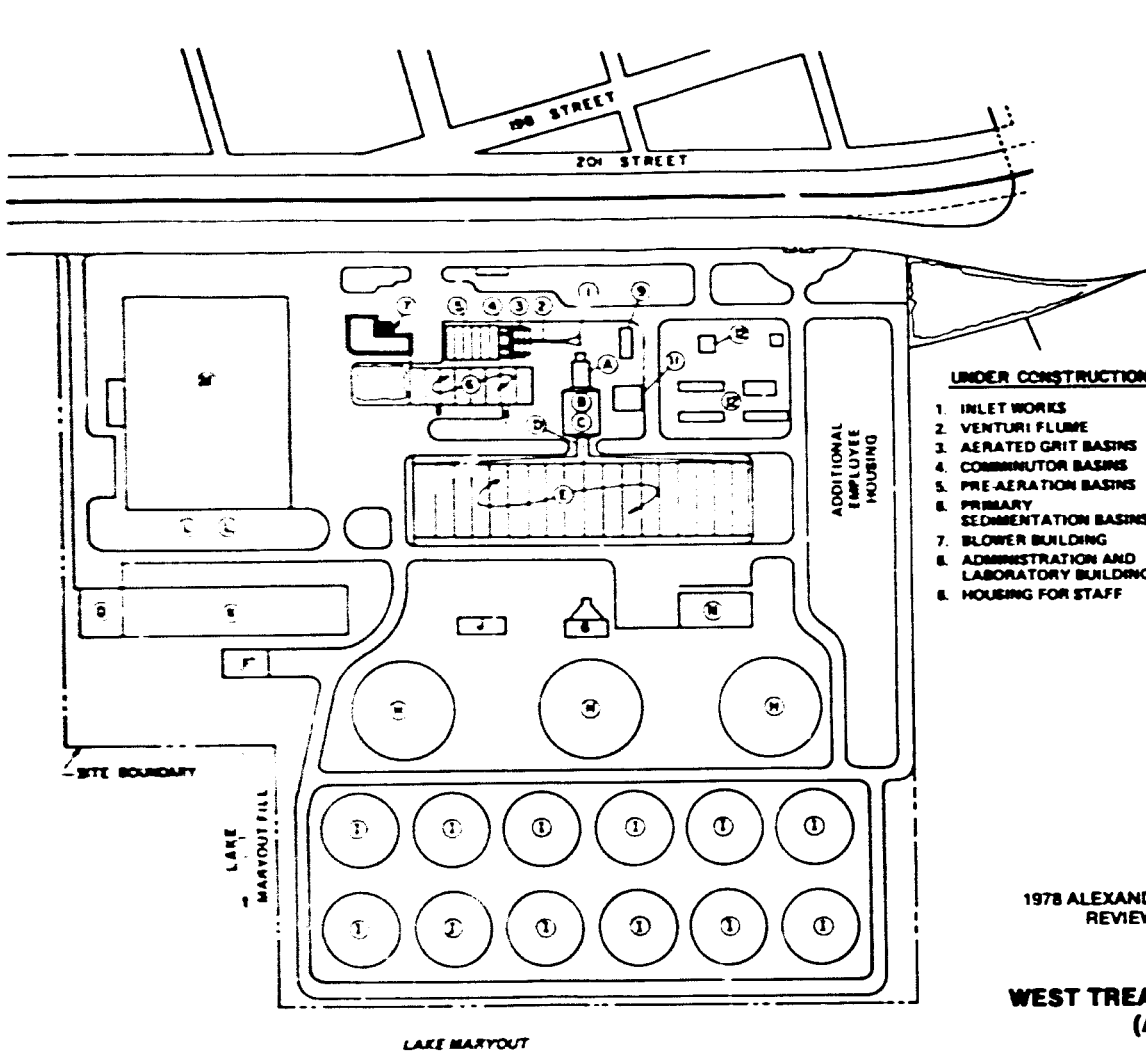


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-10

WEST TREATMENT PLANT SITE AND EFFLUENT DISCHARGE LOCATION (ALTERNATIVE II)

A/GOSD WWCG W/ECG



UNDER CONSTRUCTION

1. INLET WORKS
2. VENTURI FLUME
3. AERATED GRIT BASINS
4. COMMINUTOR BASINS
5. PRE-AERATION BASINS
6. PRIMARY SEDIMENTATION BASINS
7. BLOWER BUILDING
8. ADMINISTRATION AND LABORATORY BUILDING
9. HOUSING FOR STAFF

ADDITIONS

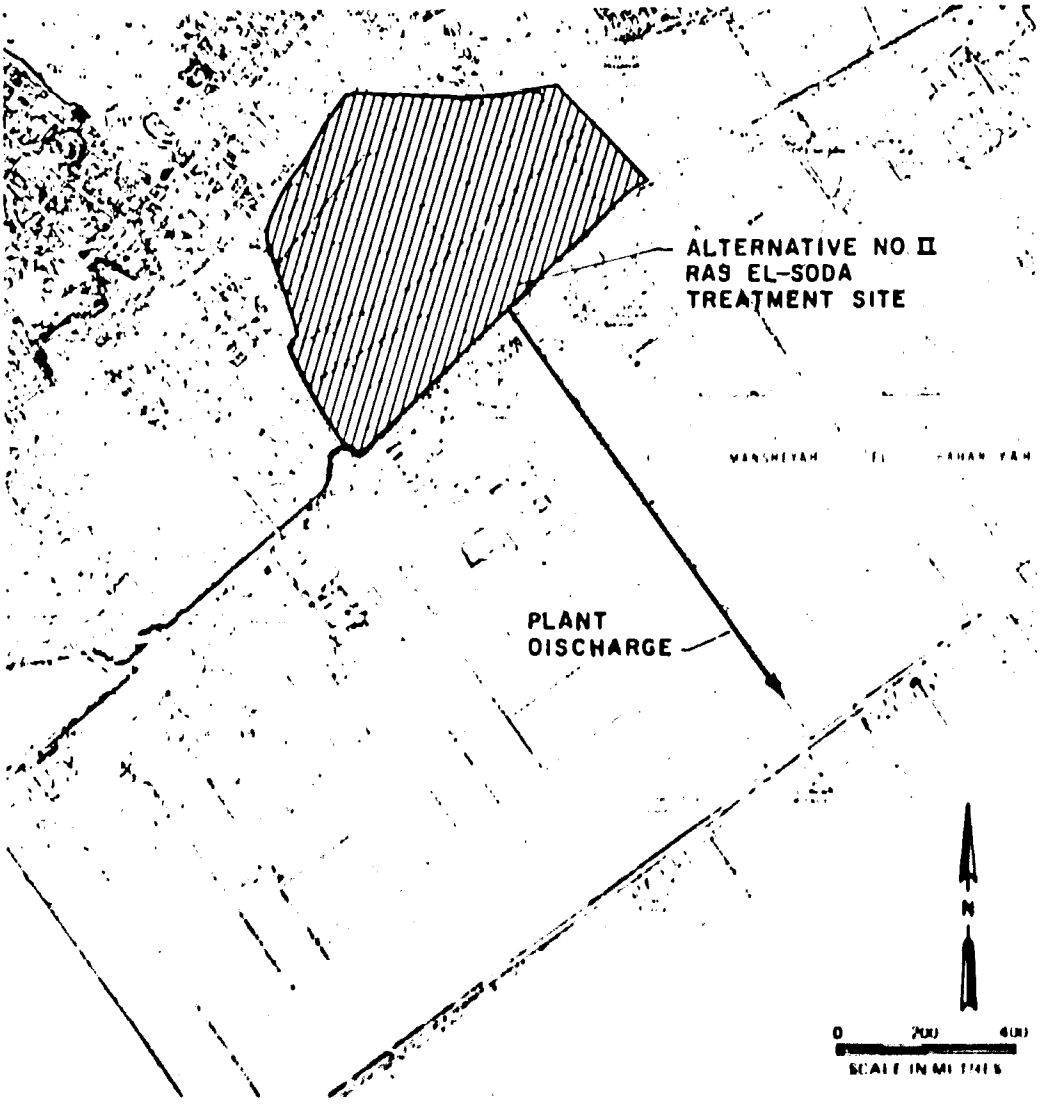
A BAR SCREENS
B AERATED GRIT BASINS
C FLOTATION BASINS
D PARSHALL FLUMES
E PRIMARY SEDIMENTATION
F CHLORINE BUILDING
G BIOLOGICAL TOWER PUMP STATION
H BIOLOGICAL TOWERS
I SECONDARY SEDIMENTATION
J SECONDARY SLUDGE PUMP STATION
K CHLORINE CONTACT BASINS
L LIQUID SLUDGE HOLDING TANKS
M SLUDGE DEWATERING BUILDING
N MAINTENANCE BUILDING
O EFFLUENT PUMP STATION

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE J-11

WEST TREATMENT PLANT SITE PLAN (ALTERNATIVE II)

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE J-12

RAS EL-SODA TREATMENT PLANT SITE AND DISCHARGE LOCATION (ALTERNATIVE II)

A/0080 WRCO WECO

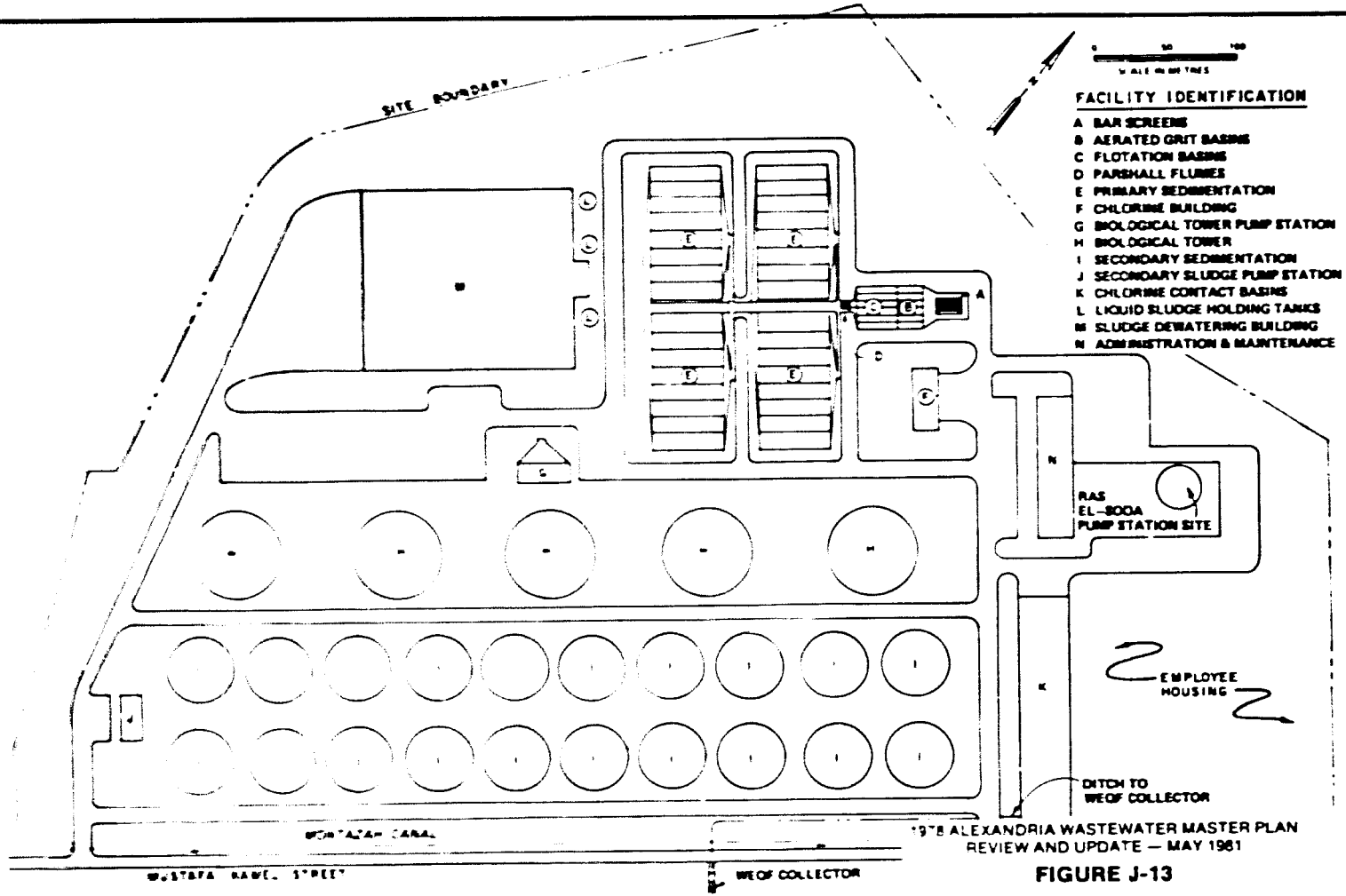


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FIGURE J-13

RAS EL-SODA TREATMENT PLANT SITE PLAN (ALTERNATIVE II)

A/GOSD WWCG W/ECG

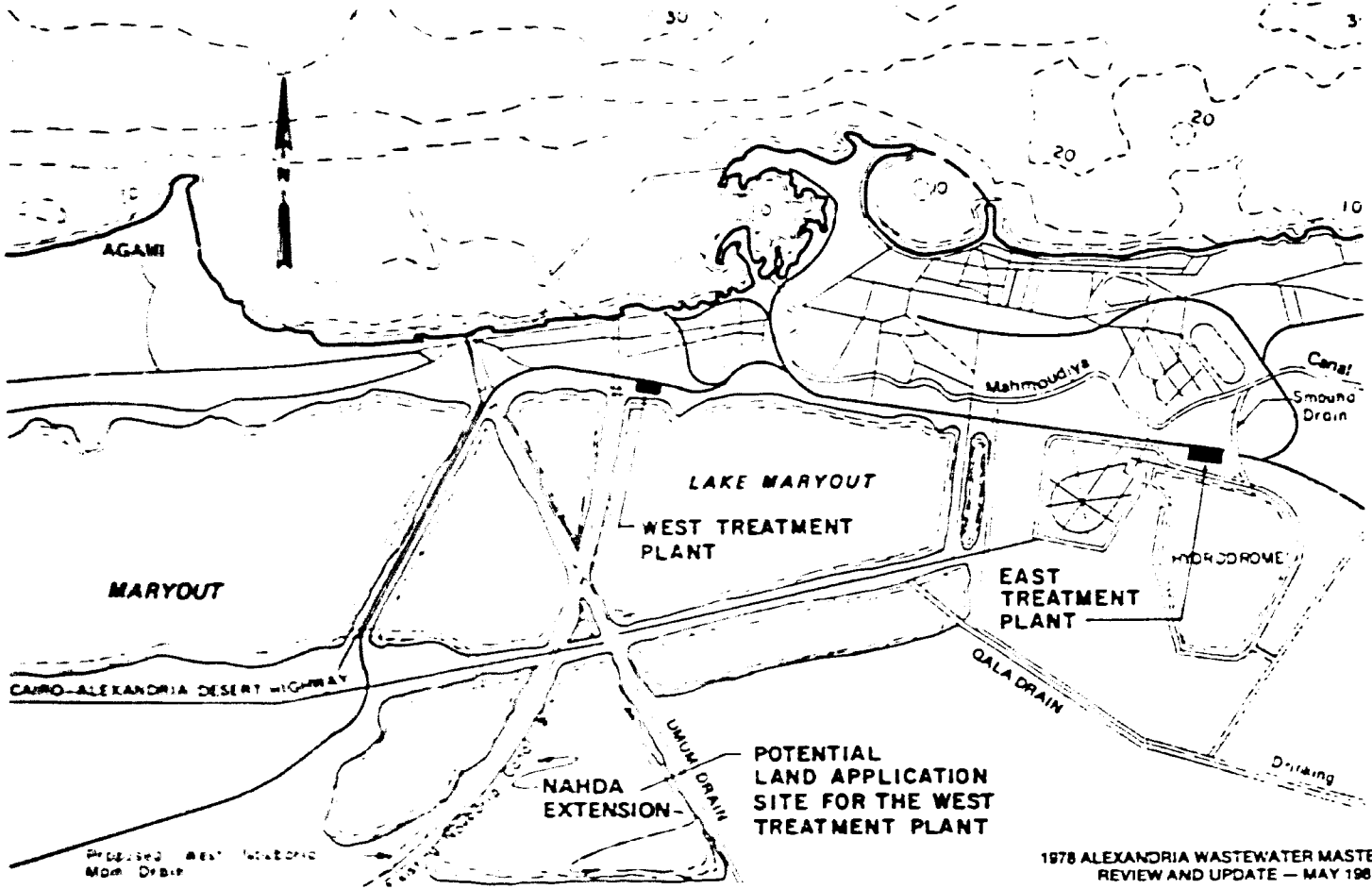

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FIGURE J-14

VICINITY MAP OF THE WEST TREATMENT PLANT LAND APPLICATION SITE

A/GOSD WWCG W/ECG

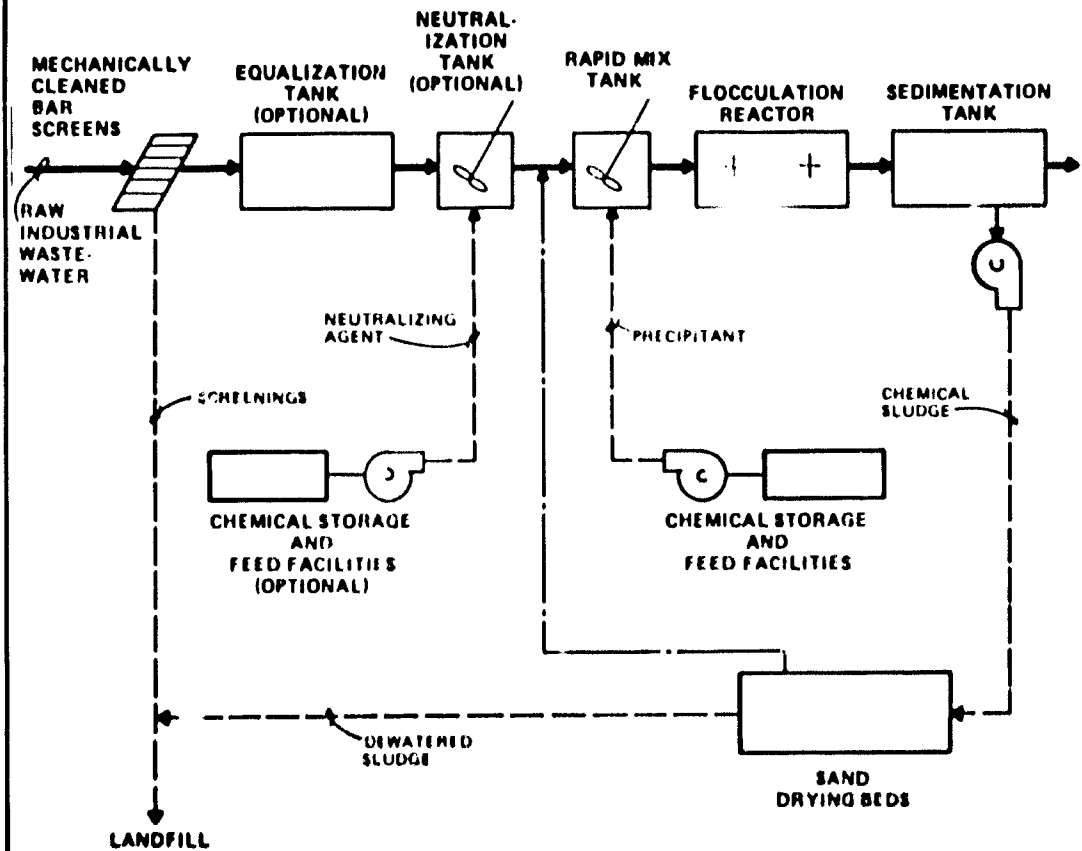

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FIGURE J-15

FLOW SCHEMATIC OF THE
WEST TREATMENT PLANT PRETREATMENT FACILITY

A/GOSD WWCG WECO

APPENDIX K

LAND APPLICATION OF WASTEWATER

INTRODUCTION

Land application offers the City of Alexandria a means to treat and dispose of its wastewater effectively while at the same time developing an additional water resource and using the nutrients it contains. Availability of water is one of the limiting factors in Egypt's ambitious program of desert reclamation. With careful planning, wastewater can be utilized to supplement existing water supplies and create new possibilities for continued growth and development.

In this appendix, the feasibility of a large-scale land application system is reexamined. The previous analysis in the 1978 Master Plan concluded that land application was unattractive when compared with the sea outfall alternative. Since that time, however, some of the basic assumptions and interpretations used in that analysis have changed. As a consequence, the current scope of services requires WWCG to "reevaluate the advisability of sea disposal versus other alternatives, including reuse and evaporation. Review will concentrate mainly on disposal means other than outfalls." Specific areas of investigation include:

- Identification and screening of additional treatment or effluent disposal options
- Investigation of alternatives for irrigation using primary treatment effluent
- Review of existing public health aspects of effluent discharge to canals
- Investigation of using primary effluent to irrigate salt-tolerant and high water- and nutrient-using forage crops
- Investigation of the management and leaching rates, cropping practices, and irrigation methods for effective land application
- Reevaluation and updating of costs presented in the Master Plan for irrigation with secondary effluent mixed with canal water
- Investigation of potential for irrigating other lands not considered in the Master Plan

- Investigation of other land application processes such as overland flow and infiltration-percolation

All of these issues were addressed in this reconnaissance-level study and are discussed in this appendix. Background considerations important to the study are presented first. These include a synopsis of the previous land application study, applicable laws and regulations, and the assumed wastewater quality and quantity data. Next, the basic concepts and requirements for land application relative to the Alexandria area are described. These are followed by a description of the site investigations with the details of the two most feasible sites. Using these basic concepts and site conditions, two alternative land application systems are developed. These two alternatives, identified as IIIA and IIIB, were selected as the final alternatives to be compared with the sea disposal alternative in the main body of this report. In this appendix, Alternatives IIIA and IIIB are evaluated on the basis of costs, public health, environmental, and social effects, implementability, and reliability. Finally, the results of the study are summarized and recommendations presented. A list of the references cited in the study follows the text.

BACKGROUND CONDITIONS

The current feasibility study of land application is part of a larger evaluation of wastewater alternatives and follows a considerable amount of previous work and discussion on the subject. Several important considerations were used as a starting point in this reevaluation. These include reviews of the 1978 Master Plan and applicable laws and regulations pertaining to wastewater reuse in Egypt, and a description of the assumed wastewater quantity and quality available for land application.

Review of the 1978 Master Plan

The Alexandria Wastewater Master Plan Study (Ref. 1), completed in 1978, developed a number of alternatives for treatment and disposal of the major part of Alexandria's wastewater flow. These included one alternative involving large-scale land application and another involving evaporation. The land application alternative consisted of conventional secondary treatment with discharge of the effluent into existing canals for blending with fresh water. The water would then be made available for unrestricted irrigation of 173,800 feddans of existing cropland. The evaporation alternative entailed the conveyance of wastewater to approximately 43,000 feddans of evaporation basins in a remote desert location. In the evaluation phase of the study, both of these alternatives were rejected in favor of a sea disposal alternative.

Two minor alternatives involving land application and evaporation were also recommended in the report. The first of these consisted of discharging 45 Ml/day from the East plant into the Qala Drain for potential irrigation reuse. The other entailed evaporation of the predominantly industrial flow (150 Ml/day) from Abu Qir in a 10,000-feddan pond at Lake Idku.

Review of the 1978 study revealed a number of debatable assumptions and conclusions regarding land application. These must be reevaluated in light of current thinking. Major [illegible]s that need to be addressed include:

- o Level of treatment requi[illegible] or to land application
- o Acceptability of effluent discharge to canals
- o Types of land application methods considered
- o Salts and nitrogen dilution before irrigation
- o Storage requirements
- o General feasibility of agricultural reuse

A separate study of the outer areas will reevaluate the evaporation plan for the outside area flows.

Level Of Treatment Before Land Application. One of the basic assumptions of the 1978 study was that secondary treatment or higher was necessary in all cases prior to crop irrigation. The reasons given were that this level of treatment was required to:

- o Protect public health
- o Reduce land area requirements
- o Maintain soil porosity
- o Minimize odor generation

This reasoning is partially valid if the effluent is to be used on crops such as vegetables, where public contact with the effluent is not controlled and where sites are located near population centers.

However, land application of primary effluent has been successfully practiced at many other locations where appropriate crop and site

selection techniques have been employed and controlled management practices have been adopted. The impact of wastewater irrigation on public health depends upon several factors. These include the form of wastewater distribution, type of crop grown, and the degree of public access to the site.

Guidelines have been developed by the World Health Organization and other regulatory agencies in the United States and other countries (Ref. 2), which relate these factors to the level of pretreatment required. For example, irrigation with primary effluent is permitted for forage, fiber, and seed crops grown where public access to the site is restricted. Increasing levels of pretreatment, including disinfection, are required in cases where the risks of exposure to pathogenic organisms are increased by sprinkling, unregulated public contact, or irrigation of food crops. The risk of public exposure can be reduced by providing potable water supplies for workers and nearby residents, erecting fences around the site, maintaining buffer zones, and planting windbreaks.

The reduction in suspended solids and soluble organics resulting from secondary treatment will generally make the handling of wastewater much easier, but it will seldom have any effect on the land area requirement for irrigation. Other factors such as hydraulic and nutrient loadings will more often control the irrigation rate and, therefore, the land area particularly in arid climates such as in Egypt.

Experience with high organic strength industrial wastewater indicates that most irrigation systems can operate effectively, with no soils clogging, with organic loading rates of 100 kg/ha/day as BOD. Considerably higher loadings are possible with good drainage and proper agricultural management. Assuming a primary effluent with a BOD concentration of 300 mg/l and a typical annual irrigation rate of 2 m/year, only 16 kg/ha/day would be applied. Consequently, soil clogging, with resulting infiltration rate reductions, would not be expected.

Offensive odors can develop if anaerobic conditions are allowed to occur within the irrigation system or on the field. This is generally more of a concern with primary effluent than with secondary effluent, due to higher concentrations of organic matter and settleable solids. However, experience with most wastewater irrigation systems indicates that sources of odors are usually associated with distribution systems rather than the actual application site. Also, most of these odors are usually local in nature and dissipate in relatively short distances. With careful design, odor problems can be minimized, or isolated in locations away from populated areas.

Acceptability of Effluent Discharge to Canals. The irrigation alternative developed in the 1978 Master Plan included discharging treated and

disinfected effluent into existing irrigation canals containing Nile River water. Although this may be technically feasible if proper levels of pretreatment are provided, it would still be unacceptable in most cases for public health, religious, and legal reasons.

The branch canals of the Nile are a primary source of domestic water throughout Egypt, particularly in rural areas. Water quality in these canals is generally poor because the canals already receive some drainage and human wastes, and contain organisms responsible for schistosomiasis. Discharge of even high-level treated wastewater would only create greater water quality problems within the canals. This is particularly true if the reliability of treatment plant performance could not be ensured.

Religious practices also limit the acceptability of canal discharges because canal water is often used by Moslems for ablutions prior to prayers. Addition of partially treated wastewater would make the canal water unsuitable for this purpose.

Egyptian Law 93 (Ref. 3), which is discussed later in this section, also legally constrains implementation of this alternative. The law prohibits discharges of wastewater containing human wastes into the Nile or its branches, including irrigation canals. Industrial discharges are permitted within specified limits.

Types of Land Application Methods Considered. A wide variety of design options is available for land application, making it possible to adapt to many different site conditions and objectives. In the 1978 Master Plan, a number of options were not fully explored that are deserving of consideration. These include two other methods of land application known as rapid infiltration and overland flow, as well as other types of irrigation system.

Both rapid infiltration (also known as infiltration-percolation) and overland flow are essentially methods of wastewater treatment. Given appropriate site conditions, tertiary-level treatment performance can be achieved (Ref. 4). In many cases, this opens up new possibilities for subsequent reuse of the effluent, since the effluent can be used to recharge the ground water or can be discharged to surface waters.

Other types of irrigation that merit further attention are primarily the so-called "high-rate irrigation" systems. These often produce less than optimum crop results, but can achieve significant savings by overirrigating appropriate types of crops using primary level effluent.

Salt and Nitrogen Dilution Before Irrigation. The 1978 Master Plan concluded that the secondary effluent would have to be mixed with canal

water to dilute both the salt and nitrogen content before irrigation. This added considerably to the amount of water that would have been handled in the alternative and consequently increased the total land use.

Electrical conductivity (a measure of salt concentration) is expected to range from about 1.7 to 2.5 millimhos per cm (mmhos/cm), which can cause problems with salt-sensitive crops. Even at this level, however, a number of management techniques can be applied. Certain selected crops will suffer little or no yield reduction (Ref. 5). Where the soils are well drained, the salts can be leached out of the root zone using excess irrigation water. This is a common and necessary practice for most irrigated areas in the Nile Delta of Egypt.

The reported mean total concentration of nitrogen is 33 mg/l as N, which is within the normal range for municipal wastewaters. This can be well managed by most land application systems. Using normal local agricultural practices, a concentration such as this will supply all of the nitrogen fertilizer requirements for most crops, with a small percentage of the nitrogen being lost to volatilization and denitrification, and the rest percolating to the ground water in acceptable concentrations.

Storage Requirements. In order to account for fluctuations in normal irrigation requirements over the year, it was assumed in the 1978 study that 25 percent of the annual wastewater flow would need to be stored. While storage is often advantageous for irrigation systems, it is expensive and not necessary in every case. Several management techniques that can be used in lieu of storage include:

- Selecting crops which can tolerate the stress of overirrigation and underirrigation
- Taking some land out of production during the midsummer peak consumptive use periods
- Accepting less than maximum crop returns as a compromise to reduce wastewater disposal costs

General Feasibility of Agricultural Reuse. One of the conclusions of the Initial Environmental Impact Statement (Ref. 6) was that agricultural reuse was generally infeasible as a result of the combination of the large volumes of water, high direct and indirect costs, high effluent salinity, poor social acceptability, and lack of an organizational unit to receive and utilize the wastewater. Each of these potential adverse impacts is addressed in the current study. The alternative land application systems are developed in such a way that these impacts are minimized or become nonapplicable.

Applicable Laws and Regulations

The current Egyptian policy toward disposal and land application of liquid wastes is set forth in Law 93 of the Ministry of Housing and Public Utilities (Ref. 3). Applicable water quality requirements specified in the law are presented in Table K-1. Discharge of sewage of any form into the Nile and its branches is prohibited. Irrigation canals which derive their water supply from the Nile are also included in this category.

In addition to the water quality requirements for irrigation shown in Table K-1, the law also specifies allowable crop types and uses. Sewage irrigation of vegetables, fruits, and plants that are eaten raw is not permitted. Grazing of dairy animals on sewage-irrigated lands is also not allowed. Irrigation rates must be low enough so that surface ponding does not occur.

Although the law is fairly specific with regard to many aspects of land application of wastewater, it does not entirely cover all situations. In these cases, current standards and practices from the United States have been applied.

Wastewater Quantity and Quality

The wastewater flow rates used as the basis of design for the land application Alternatives IIIA and IIIB are presented in Table K-2. These values were developed in Appendix J, and include only the wastewater from the eastern zone, western zone, central zone, Ras El-Soda, Siouf Keblia, Sadat City, and the domestic flows from Abu Qir.

The assumed raw wastewater quality in this study is presented in Table K-3. These values are based on analyses reported in the 1978 Master Plan.

BASIC CONCEPTS OF LAND APPLICATION

In this section, general concepts and important considerations are introduced for each of the three basic land application processes. These processes, which are shown schematically in Figure K-1, are commonly referred to as irrigation, rapid infiltration, and overland flow. Each process utilizes different physical, biological, and chemical means to treat, dispose of, and/or utilize the wastewater. Typical design features for the three systems are compared in Table K-4 and are described in the

following subsections. The discussion is based on local conditions and highlights considerations that are particularly important for Alexandria.

Irrigation

Wastewater irrigation is a process used to treat and dispose of wastewater while utilizing the water and nutrients for the production of crops. In many respects it is operationally similar to standard irrigated agriculture, with wastewater being applied by either surface or sprinkler distribution methods. Extremely high levels of treatment can be achieved as the wastewater interacts with soils and crops. Most irrigable soils are suitable for wastewater systems; however, poor drainage and steep topography limit site-specific suitability and increase system cost.

Water and nutrients can be managed to either optimize crop production or maximize wastewater irrigation rates. Systems designed to optimize crop production store wastewater to match seasonally variable crop irrigation requirements, apply wastewater at relatively low hydraulic loading rates, and pretreat wastewater to allow use on high income crops. Systems designed to maximize wastewater irrigation rates apply water in excess of crop consumptive water demands and use crops that have high water and high nutrient requirements. Revenues from crop production are generally lower where the irrigation rate is maximized; however, initial capital costs are generally decreased.

Development of a successful wastewater irrigation system requires careful consideration of number of interrelated design factors. These can be varied to suit particular requirements or conditions at a specific site. Some key factors are highlighted in the following discussion including treatment effectiveness, pretreatment requirements, cropping pattern, irrigation rate, storage, land requirement, and system management.

Treatment Effectiveness. The irrigation process is capable of providing excellent levels of pretreatment which greatly exceed the capabilities of most conventional and advanced waste treatment processes. Typically, subsurface drainage from wastewater irrigation systems contains less than 2 mg/l BOD, less than 1 mg/l suspended solids, 3 mg/l total nitrogen, less than 0.1 mg/l phosphorus, and about 100 MPN per 100 ml fecal coliform (Ref. 4). The only water quality limitation that would normally restrict any subsequent reuse is the concentration of total dissolved solids, or electrical conductivity, which will increase significantly. The quality varies with pretreatment level, soil characteristics, irrigation rates, and cropping pattern; however, in almost all cases it would be generally suitable for discharge to surface drains.

Pretreatment Requirements. The level of pretreatment for wastewater irrigation systems will generally be dictated by any or all of the following factors:

- Requirements of the distribution system
- Nuisance conditions from odors
- Public health impacts

For most surface distribution systems of the type commonly used in Egypt there should be no significant problems with primary level effluent. However, with sprinkler systems, careful attention must be paid to the removal of coarse and settleable solids. To avoid plugging of nozzles, it is generally recommended that the size of the largest particle in the applied wastewater be less than one-third the diameter of nozzle orifices. For most systems, other than the traveling gun, which utilizes a large orifice and high pressures, some solids removal is required. If aerobic or facultative ponds are used prior to sprinkling, it is necessary to remove agglomerated algae using relatively fine screens.

Reducing odors is particularly advisable if the wastewater irrigation system is close to residential communities. If they occur, odors are most likely generated in treatment ponds rather than in subsequent storage facilities. Odors from storage ponds can be minimized if the equivalent of primary treatment is provided or if storage ponds are operated as facultative lagoons.

The level of pretreatment required to protect public health is dependent upon the type of crop grown and control of public access to the site. Primary level treatment is generally acceptable if crops are restricted to feed, seed, and fiber crops. Public access can be controlled by limiting the labor requirement, locating residential communities off the site, and by providing easily accessible potable water for workers and residents. Increasing levels of biological treatment and disinfection are advisable for irrigating food crops and where public access cannot be adequately regulated. In areas where irrigating food crops that may be eaten raw is permitted, a minimum of secondary treatment with good disinfection is generally required.

Crop Selection. The cropping pattern used at a wastewater irrigation site affects the application rate, storage requirement, and revenue. These factors influence the cost and feasibility of the operation.

The following considerations must be included in the evaluation of possible crop types:

- High water- and nutrient-using crops require less land area.
- Water-tolerant crops can accept more irrigation water during the winter months, which could reduce storage requirements.

- The crops must be moderately salt-tolerant.
- Crops must be able to tolerate continuous nitrogen applications unless nitrogen concentrations can be controlled.
- The crops must be compatible with expected soil and drainage conditions.
- Forage, fiber, and seed crops are most suitable from the public health standpoint for irrigation wih primary level effluent.
- Vegetables and other high-value crops require much higher levels of pretreatment.
- Crops with minimum cultivation requirements will reduce system operation complexities.
- Crops with high net income can reduce annual operation costs.
- Irrigation methods are available for certain crops that minimize farmer/irrigation contact with the wastewater.
- Selected crops must be marketable.

Irrigation Rate. The wastewater irrigation rate affects storage and land requirements. It is dependent upon crop consumptive use (also referred to as evapotranspiration), precipitation, the salt leaching requirement, and application efficiency. The nutrient loading rate can also limit the wastewater irrigation rate or affect crop selection.

The monthly consumptive use (CU) for selected potential crops is listed in Table K-5. These data were derived, for the most part, from the 1978 Master Plan, with some additions and changes. A modified Blaney Criddle method was used with long-term Alexandria temperature data to estimate CU. The CU for alfalfa was developed using a similar procedure, as it was not included in the previous study. Also, the CU for wheat was modified from that presented in the 1978 Master Plan to reflect the growing season correctly. Table K-5 does not include pre-irrigation for planting, which should be considered for specific site and crop conditions during final design.

The effect of rainfall was considered in the irrigation rate development as it can supply part of the crop CU requirements. However, not all of the rainfall is effective in helping supply water. Some rain may fall during periods of limited crop use and some can be lost to evaporation or runoff before it infiltrates the ground surface. Long-term precipitation records do not exist for the inland desert areas southwest of Alexandria,

which are the most probable land application sites. Therefore, an average rainfall of 100 mm per year was used, based on review of several sources (Refs. 1, 7, and 8).

The wettest year in 10 is commonly used in wastewater irrigation system design to ensure that enough land is available during abnormally rainy years. At Alexandria the highest "1 year in 10" annual precipitation is about 42 percent greater than the normal (Ref. 7). Assuming an equivalent percentage increase for inland desert areas, total precipitation is about 142 mm in a wet year. This total was prorated monthly by using relative monthly precipitation at Alexandria. These values were then used to compute the effective precipitation (e.p.) using standard U.S. Soil Conservation Service methods (Ref. 9). Monthly effective precipitation is listed in Table K-5.

The salt leaching requirement is also an important factor in the wastewater irrigation rate. It is defined as that fraction (expressed as a percent) of the irrigation water that must be leached through the active root zone to maintain a steady state salt balance. It is dependent on several factors, including:

- Irrigation water quality
- Crop type and stage of growth
- Concentration and depth of salts in the soil profile
- Depth of soil profile to the water table
- Soil physical and chemical properties

For this study it was not possible to obtain enough site-specific information to accurately determine leaching requirements. Therefore, a simplified method was used to approximate the leaching requirement. This value was then compared to local research data.

The electrical conductivity of the wastewater, based on measurements presented in the 1978 Master Plan, generally varies from about 1.7 mmhos/cm to 2.5 mmhos/cm (Ref. 1). An average value of 2.1 mmhos/cm was used in this analysis. The following equation, which considers only the electrical conductivity (EC) of the irrigation water (ECw) and the maximum salinity that can occur in the drainage water (ECdw), was used to compute the leaching requirements (LR):

$$LR = \frac{ECw \times 100}{ECdw}$$

The expected crop yield reductions with the wastewater for several crops, the ECdw values, and the computed LR's are presented in Table K-6. The calculated leaching requirements average about 6 percent. However, local research (Ref. 10) has shown that the simplified steady state salt balance equation does not adequately account for the local soil conditions. Actual leaching requirements will normally exceed the values in Table K-6 by 100 to 200 percent. This is especially true if relatively high efficiency sprinkler irrigation is used. Therefore, an average LR of 20 percent was used as the basis of irrigation design.

The application efficiency accounts for irrigation water that is not uniformly applied or is lost by runoff or evaporation. The net irrigation requirement is divided by the irrigation efficiency to give the total irrigation requirement. Reasonable application efficiencies for surface and sprinkler irrigation are 65 and 80 percent, respectively (Ref. 11).

The nutrients contained in the wastewater, especially nitrogen and phosphorus, are important to irrigation system design. They benefit crop production if applied at moderate rates. Normal wastewater irrigation rates often supply about the correct amount of nitrogen for crop production. However, some crops such as sugar beets and cotton suffer yield decreases if nitrogen is applied late in the growth cycle. Therefore, the nitrogen content of the wastewater may limit crop selection.

Another possible design limitation is high nitrogen loadings. If percolating irrigation water reaches ground waters that are used for drinking water supplies, it will usually be necessary to restrict concentrations of nitrate plus nitrite to 10 mg/l (as N) or less. In cases where influent nitrogen levels are high, it may be necessary to limit wastewater application rates or select only high nitrogen uptake crops to ensure that this criterion is met.

Storage. Wastewater flows, except for diurnal fluctuations, are normally quite constant. In Alexandria, during the summer tourist season, the peak month flows are only about 15 percent higher than the mid-winter flow rates. On the other hand, crop irrigation requirements vary greatly from season to season. The peak summertime irrigation requirement may be 3 to 5 times greater than the wintertime rate. If crop production is to be maximized, storage is necessary to match the wastewater flow to the irrigation requirement. The storage requirement may be 10 to 30 percent of the annual flow volume depending on the cropping pattern.

If minimizing capital costs and maximizing irrigation rates is the major objective, a system can be designed with little or no storage. This can be done by selecting water-tolerant crops, overirrigating in the winter and underirrigating in the summer, varying the amount of land under irrigation at any time, and selecting a cropping pattern with low summertime water requirements. Of course, the soil must be well drained to allow overirrigation. Also, it is possible to apply most of the leaching fraction during the period of low crop CU to help balance the irrigation rate with wastewater supply.

A storage volume equal to at least several days' flow should be provided for any type of system design. This minimum storage volume will allow for mechanical failure or mishaps in the irrigation system and will also permit some flexibility in the scheduling of irrigation.

Land Requirement. The land requirement is generally computed as the annual wastewater flow rate divided by the annual irrigation rate. Based on a 30- to 90-feddan land requirement per 1 Ml/day wastewater flow (from Table K-4), the net land area required for the year 2000 flow rate of 945 Ml/day would be 28,000 to 85,000 feddans. This does not include land for roads, buffers, and miscellaneous facilities.

For a given site and climatic conditions, the land requirement can vary greatly depending on the cropping pattern. High water-using forage crops, for instance, may require only 50 to 80 percent as much land area as some grain, vegetable, and fruit crops. Land requirements can also vary, depending on the level of wastewater pretreatment.

System Management. A wastewater irrigation system requires additional and somewhat different management than that required for conventional irrigation systems. The protection of public health and handling of the continuous wastewater flows must be considered. Mechanisms protecting health include minimizing public and operator contact with the wastewater and controlling crop selection and handling. A higher degree of management is also necessary to ensure continuous operation of a wastewater irrigation system.

The following general management practices are typically used for wastewater irrigation to ensure safe and effective operations:

- Control of public access (fewer limits are necessary with better quality wastewater)
- Provision of buffer areas between irrigated areas and homes, businesses, roads, and surface waters
- Prevention of runoff of untreated wastewater from the site

- Management of the time and location of sprinkler irrigation to minimize wind drift of wastewater beyond the boundaries of the site
- Control of irrigation rates to eliminate ponding and runoff
- Use of good general housekeeping practices to minimize odor and vector problems
- Careful scheduling of agricultural operations to ensure that enough land is always available for wastewater application
- Scheduling irrigation so that incoming wastewater can be handled by either storage or land application
- Selection of crops that are acceptable for irrigation with the expected quality of wastewater
- Design of the irrigation system to minimize operator contact with the wastewater
- Provision of drinking water and sanitary facilities for farm workers at convenient locations
- Monitoring wastewater flows, irrigation rates, ground-water depth and quality, soil conditions, and crop production to assure achievement of desired effects.
- An organized program of equipment maintenance

Rapid Infiltration

Rapid infiltration is a land application method also used to treat and dispose of wastewater. The wastewater is ponded in a basin and then treated by a combination of physical, biological, and chemical processes as it percolates through the soil. It requires a moderate to rapidly permeable deep soil, with flat to mild slopes. Generally no crop is grown, although water-tolerant grasses are often planted as an aid to infiltration. A very important aspect of the system is the periodic drying cycle, which promotes soil re-aeration and simultaneous decomposition of accumulated wastewater solids. Less land is generally required than for the other land application methods. The area required depends mainly on the soil permeability and hydraulic loading rate.

The system must be designed to provide a certain minimum quality of the percolated water. In many instances, this water is used to replenish

ground water supplies, if the expected quality is consistent with the intended use. In other cases, the ground water can be recycled by wells or underdrains located onsite and then reused for agriculture or discharged to surface water.

Treatment effectiveness, pretreatment requirements, hydraulic loading rate, ground-water storage, and system management are some of the key factors which must be considered in the design of a successful rapid infiltration system.

Treatment Effectiveness. Rapid infiltration systems can be designed and operated to achieve effluent quality equivalent to that of conventional tertiary treatment. The expected quality of treated water from typical systems is summarized in Table K-7.

BOD and suspended solids are almost completely removed by a combination of filtration and straining within the soil, with subsequent biological degradation (Ref. 4).

Removal of microorganisms, including bacteria, viruses, and parasitic protozoa and helminths (worms), is accomplished by filtration, adsorption, desiccation, radiation, predation, and exposure to other adverse conditions. Because of their large size, protozoa and helminths are removed primarily by filtration at the soil surface. Bacteria also are removed by filtration at the soil surface, although adsorption may be important.

Removals of fecal coliform bacteria observed at existing rapid infiltration systems range from > 99% removal after 7 m of travel at Hollister, California, to essentially complete removals at Lake George, New York, and Phoenix, Arizona, after 7 and 90 m of travel, respectively.

The primary removal mechanism for viruses is adsorption. Because of their small size, viruses are not removed by filtration at the soil surface, but instead, travel into the soil profile. Only a limited number of studies have been conducted to determine the efficiency of virus removal. At Phoenix, Arizona, results indicate that 90 to 99% of the applied virus is removed within 10 cm of travel when either primary or secondary effluent is applied and that 99.99% removal is achieved during travel through 9 m of soil following the application of secondary effluent.

Nitrogen removals are variable and may or may not be desirable depending on the ultimate use of the treated water. The primary removal mechanism is denitrification, which can be adjusted by varying the flooding-to-drying cycles, increasing the ratio of organic carbon to nitrogen in the applied wastewater, changing the application rates, or using vegetated rather than bare surfaces within the infiltration basins.

The primary removal mechanisms for phosphorus include adsorption and chemical precipitation. The removals can range from 70 to 90 percent depending on the physical and chemical characteristics of the soil, residence time of the water in the soil, and travel distance.

Pretreatment Requirements. If public access is adequately restricted, the pretreatment required for rapid infiltration is dependent primarily on hydraulic loading considerations. Biological treatment as high as secondary levels is sometimes provided to reduce problems with soil clogging and to increase application rates. However, primary effluent or screened wastewater can be used successfully if reduced application rates are considered acceptable and the basins are well maintained. Basin maintenance may involve frequent disking of the basin surface or planting the basin in a water-tolerant grass to improve infiltration.

The level of pretreatment prior to rapid infiltration also affects nitrogen removal efficiency. Higher organic carbon concentrations in the applied wastewaster promote denitrification and result in lower concentrations of nitrogen in the percolate. This is advantageous if the ground water is used for drinking water supply, but is somewhat undesirable if the ground water is recycled for irrigation use.

Hydraulic Loading Rate. Selection of the hydraulic loading rate is the most important and, at the same time, the most difficult step in the design of a rapid infiltration system. It is a function of the soil infiltration capacity, the loading cycle, the quality of the applied wastewater, the treatment requirements, and in some cases, the level of desired ground-water storage. Typically, hydraulic loading ranges from about 10 m per year in less permeable soils to well over 100 m for systems using low organic strength wastewater and coarse sandy soils.

The infiltration capacity of the soil is a difficult parameter to measure, with results varying widely depending on the method of measurement. For preliminary design with no detailed testing at the site, the vertical hydraulic conductivity (which is related to the permeability) is often used. It can be estimated for clean water based on a knowledge of the physical and chemical characteristics of the soil. However, infiltration rates with wastewater decrease as solids are filtered out at the soil surface. Therefore, somewhat lower rates are used for design. Experience with rapid infiltration systems indicates that the annual hydraulic loading rates should generally be limited to between 4 and 10 percent of the measured clear water rate for the most restrictive layer.

In many systems the organic and solids content of the wastewater limits the hydraulic loading rate. The permissible loading varies depending upon the loading cycle and the ability of the soil to re-aerate; but generally

most systems are able to accept about 225 kg/ha/day of both BOD and suspended solids without serious reductions in infiltration rates.

Ground-Water Storage. Once the percolate from a rapid infiltration system reaches the ground water it will be of high enough quality to be used for many forms of irrigation. A useful advantage of the system is that in many cases this water can be stored as ground water and withdrawn during the year at rates consistent with seasonal irrigation requirements. This concept is illustrated on Figure K-2.

The primary requirement is to have a site with sufficient depth of permeable soil to accommodate the volume of treated water to be stored. In addition, the system should always have at least 1 m of unsaturated soil between the bottom of the basins and the top of the ground-water mound as this is where most of the treatment occurs. If needed, the existing ground-water level can be lowered somewhat to ensure an adequate unsaturated soil depth. The ground-water mound can be managed by design of recovery well networks that confine percolated water within limited areas beneath the site.

System Management. If properly managed, rapid infiltration systems are generally easy to operate and provide a high degree of reliability. Normal operation consists of flooding and drying the basins on a regular cycle, maintaining basin surfaces, and operating recovery wells to meet irrigation demands and control the stored ground water. The major operational objectives are that the desired level of treatment takes place and that stored ground water is confined as desired. Most system activity takes place within the subsurface, out of sight of the operators. Consequently, achievement of both of these objectives requires regular monitoring and a good understanding of the principles involved.

The following are some general management practices that are typically used at rapid infiltration facilities to assure operation efficiency and protect public health:

- Control of public access to the site
- Prevention of runoff from the site
- Careful monitoring of application rates in order that infiltration capacities are not exceeded
- Regular maintenance of basin surfaces to maximize infiltration rates
- Monitoring of wastewater effluent and ground water beneath and adjacent to the site to evaluate treatment performance

- Adjustments to flooding and drying cycles as required for optimum treatment
- Monitoring of ground-water levels
- Adjustments to recovery well operations as required to assure that stored ground water remains within the desired location and that an adequate unsaturated zone is maintained within the soil
- Regular maintenance of the distribution system
- Provision of readily accessible potable water to workers

Overland Flow

Overland flow, like rapid infiltration, is used more as a treatment than a reuse or disposal method. In this process, wastewater is applied over the upper reaches of sloped terraces and allowed to flow across a vegetated surface to collection ditches. The wastewater is treated by physical, chemical, and biological means as it flows in a thin film down a relatively impermeable, grass-covered slope. Very little wastewater percolates below the root zone because the soils have limited permeability. Treatment is the main objective, and crop production is limited to low yields of grasses. The process can achieve a secondary quality effluent from primary treated wastewater (Ref. 4).

Overland flow is most frequently used for treatment of food processing and certain other industrial wastewaters, as it is most effective for removing BOD, suspended solids, and nutrients. It is somewhat less reliable for pathogen removal than are the irrigation and rapid infiltration processes, because most of the water does not flow through the soil and short circuiting can occur if the system is not operated properly.

Overland flow should be suitable for discharge to drains according to the requirements of Egyptian Law 93 (Ref. 3). However, it is unlikely that it could legally be discharged into waters of the Nile and irrigation canals. If used for uncontrolled irrigation and where public access is not limited, disinfection might be required. The effluent could certainly be reused without further treatment on an A/GOSD-controlled irrigation system if crops were limited to forage, fiber, and seed.

As discussed in the following section, overland flow was eliminated from further consideration because no suitable sites could be found. This is due to a combination of somewhat unsuitable soil conditions and to the unreliability of pathogen removal. However, the process may

have some merit for small local systems, but those are not in the scope of this study. The reader is directed to reports by EPA (Ref. 4) and Loehr (Ref. 2) if more information is desired on the process.

SITE INVESTIGATIONS

Land application systems can be developed to suit a wide variety of different site conditions. Consequently, the identification and evaluation of potential sites is a critical element since each site must be considered in conjunction with various design possibilities. In the process of site identification, an attempt was made to first identify all of the areas in the vicinity of Alexandria that could possibly be used for land application. These areas were then evaluated and screened so that attention could be focused on those which presented the best opportunities.

This section first describes the principal criteria that were developed for the identification of potential sites and then illustrates how these criteria were used for initial screening. Following this are descriptions of the more detailed site evaluations that were conducted for the two sites that were found to be most suitable.

Site Identification Criteria

The primary factors which must be considered in the identification of potential sites for land application are:

- Availability
- Existing and planned land use
- Land area requirements
- Soil and geologic characteristics
- Topography
- Ground water conditions
- Site location (distance from Alexandria and transport access)

Preliminary site identification criteria were developed for each of the factors, based on local conditions which are discussed in the following subsections.

Availability. For any land application system in Alexandria an important criteria is that the site be available for direct or indirect control by A/GOSD. This is necessary to ensure that A/GOSD can maintain control over wastewater handling and site management. Therefore, government-owned, public lands are preferred. Some privately-owned areas would also be acceptable, but would generally be economically infeasible due to the relatively large land requirements and high costs and problems associated with land acquisition. Military lands were assumed to be unavailable and were not considered in this analysis.

Land Use. Extensive areas of urban, semi-urban, industrial, commercial, historical, cultural, and military land use are generally incompatible with most land application systems and were therefore excluded from consideration. Agricultural lands with large numbers of small farms or lands where predominantly food crops are grown are also not generally suitable, although they can sometimes be used for wastewater irrigation following high levels of pretreatment. The preferred areas for land application are undeveloped, remote areas and some low-density, large-scale agricultural areas. These areas usually involve the least costs for development and afford the best opportunity for control of public access.

Land Requirements. With such large wastewater flows being considered, land area requirements were a major factor in site identification. The approximate land area needed for each land application process is as follows:

- Irrigation: 28,000 to 85,000 feddans
- Rapid infiltration: 1000 to 15,000 feddans
- Overland flow: 5000 to 28,000 feddans

Single, contiguous sites are preferred to minimize site development costs and to simplify management and operations. However, combinations of smaller sites are often adaptable and were considered as possibilities.

Soils And Geology. Soils and geologic characteristics are the most important physical factors considered in the selection of potential sites. These factors determine which, if any, of the three land application processes is most suitable at a specific location. While texture, effective depth, drainage, available water-holding capacity, salinity, and alkalinity were all considered in the site evaluations, the key soil characteristic for preliminary site identification is most often permeability. In general, the irrigation process can be adapted to a wide range of soil permeabilities, although special design features are often required for soils at both the high and low ends of the range. Locations

with deep, moderately rapid to rapidly permeable soils often have potential for rapid infiltration, while slowly permeable soils are required for overland flow.

Topography. The natural topography of the region surrounding Alexandria poses few constraints for land application. The one major exception is the area of limestone ridges near the Mediterranean Sea, in the Bahig area. Flatter slopes are usually preferred to reduce development costs, but slightly rolling or hummocky areas with slopes up to 10-15 percent may be acceptable. If the soils are slowly permeable, at least mild slopes are desirable to improve drainage. Most of the topography in the areas studied is suitable.

Ground Water. The depth to the ground-water table is an important site identification factor for most irrigation and rapid infiltration systems as it affects the need for drainage, the drainage method, and cost. Sites considered for wastewater irrigation should ideally have at least a 1- to 2-m depth to water table, while a minimum of 3 m of permeable soil above the water table is preferable for most rapid infiltration systems. Lesser depths than these can be tolerated if drainage is provided. The rate and direction of movement, quality, existing uses, and potential uses of the ground water are also important considerations. However, land application systems can normally be managed to adapt to those site conditions.

Location. The distance from Alexandria to a potential land application site will greatly affect the cost of the conveyance system. Closer sites will entail less cost for wastewater conveyance and are therefore preferable. However, the scale of this project warrants consideration of areas up to 100 km from Alexandria.

For rapid infiltration and overland flow systems, consideration must also be given to the potential site location relative to an acceptable point of ultimate disposal or reuse. Both of these systems provide a high level of treatment and preferably should be located near an irrigation area that can make good use of the renovated water.

Site Identification and Screening

Site identification criteria developed in the previous section were used to identify locations suitable for land application. The study area was then divided into seven large subareas, shown in Figure K-3, each having generally similar site characteristics. The subareas include:

- Subarea A--Irrigated lands near Alexandria

- Subarea B--Nahda Extension and Barseek area wetlands
- Subarea C--Government farming companies
- Subarea D--Bahig area
- Subarea E--West Noubaria Extension, Reclamation and Settlement Project
- Subarea F--Remote areas west and southwest of the El Nasser Canal
- Subarea G--Undeveloped areas along the Desert Road

Each of these subareas presents different possibilities for irrigation, rapid infiltration, and overland flow systems which are discussed in the following subsections.

Wastewater Irrigation Sites. Wastewater irrigation generally presents the largest range of possibilities for land application. In this case, all of the subareas were considered as possible irrigation sites.

Presently irrigated lands of Subarea A include the Abu Qir drain area, Abis area, Maryout sector and the Nahda area. Most of these were evaluated in the 1978 Master Plan. Although these areas are close to Alexandria, they are generally unsuitable for wastewater irrigation for the following reasons:

- Existing small farm sizes makes control of public access difficult, requires complex site management, and provides limited ability to control farming operations under a single authority. Without high-level wastewater treatment, the land would have to be consolidated into a single ownership, and the farmers moved from their lands.
- Existing cropping patterns are incompatible with primary effluent irrigation. Expensive, higher-level treatment would be required to protect public health.
- Existing drainage and irrigation systems are unsuitable for handling wastewater, as their use would result in mixing wastewater with waters of the Nile. Extensive modifications would be necessary to make the systems compatible.
- Soils in the area are generally poorly drained, making them marginally suitable for irrigation with the somewhat saline Alexandria wastewater.

The Amria northwest area is not presently irrigated but was included in Subarea A because it is close to Alexandria. It is much too small to accept a significant amount of wastewater. Also, because incompatible land uses are rapidly developing in the area, it was excluded from further consideration.

The remaining irrigated delta lands of Subarea A, generally the areas east of the Noubaria Canal, were also excluded. These lands have the same unsuitable features as the aforementioned sites, plus they are farther from Alexandria.

Subarea B, the Nahda Extension, and the Barseek area consist of wetlands the government is considering for reclamation. They were considered for wastewater irrigation because they are relatively close to Alexandria and presently undeveloped.

The Nahda Extension would contain about 5000 feddans of irrigated land. No decision has yet been reached as to whether the reclamation project will proceed or whether the area will be left as a wetland and fishery (Ref. 12). Consequently, the availability of this site is very questionable. Due to the small amount of land and questions on availability, the Nahda area was excluded from consideration except as a possible irrigation site for wastewater from a local plant such as the West Treatment Plant. This area could also be used as a sludge/garbage composting site or as a site for a pump station to convey Alexandria's wastewater to a remote irrigation site. No detailed investigations were performed on the Nahda Extension because it is presently covered with water and consequently little data are available.

The Barseek area wetland is included in existing government reclamation plans (Ref. 12). It is a part of Lake Idku and would contain about 20,000 feddans of reclaimed land. The western end of the site includes the 10,000-feddan area which the 1978 Master Plan proposed as an evaporation pond for the Abu Qir industrial wastewaters. Evaluation of the Barseek area indicates that it is feasible only as a site to be considered in the future for wastewater flows from the east end of Alexandria or other nearby areas. It is unsuitable for the present study for the following reasons:

- The land area could not accept more than about 25 percent of the Alexandria wastewater flow.
- The availability of the land is somewhat questionable because plans for irrigating it with Nile waters are already being developed.
- Development costs would be high because it would have to include initial reclamation and an extensive drainage system.

In addition, certain environmental features of the Barseek wetland make it a questionable area for any reclamation project. The penaeid shrimp fishery of Abu Qir Bay apparently depends on estuarine Lake Idku and associated wetlands for the maturation of post-larval shrimp (Ref. 6). Thus, the social and economic benefits anticipated with any reclamation project would have to be weighed against the potential decrease or irretrievable loss of the shrimp fishery.

Subarea C consists of relatively large (10,000 to 50,000 feddans) government farming companies. Included in this area are the Nahda Company, Maryout Company, Mechanized Farm, North Tahrir Company, Nuba Seed Company, and the Egypt Milk and Meat Company. The water supply to these farms is inadequate. However, the use of these lands for supplemental wastewater irrigation is not recommended for the full wastewater flow from Alexandria at this time for the following reasons:

- The Ministry of Irrigation is developing plans to pump drainage water into the Noubaria Canal to alleviate the water shortage problem.
- Some existing cropping patterns are incompatible with primary effluent irrigation; either the cropping pattern would have to be modified, or expensive, high-level treatment be provided.
- The existing farming companies would have to yield most of the control of the sites to A/GOSD to ensure that the wastewater is handled properly.
- Public access would have to be totally controlled, which would be difficult in several areas which have communities nearby or within the sites.
- Some areas have drainage and salinity problems, resulting from the design of existing irrigation systems and their operation. Extensive system and operational modifications would be required for the site to be suitable for wastewater irrigation.
- Multiple sites would be needed to handle Alexandria's total wastewater flow.

Although not recommended as a major alternative for this study, parts of Subarea C might be considered for wastewater irrigation in the future if the present water shortage continues. This might be a particularly attractive possibility if a conveyance line is built through this area to another land application site further south. With full cooperation among the governmental agencies, an agreement could be worked out that

might allow farming companies to receive future increased wastewater flows with strict control of the wastewater system by A/GOSD.

Subarea D includes the lands served by the Bahig Canal (on its north side) and the nonirrigated lands between that canal and the lands served by the El Nasser Canal.

The lands irrigated from the Bahig Canal have been suggested as a site for Alexandria's wastewater because they presently have an inadequate water supply. However, based on discussions with the Ministry of Irrigation (Ref. 12) adequate water will be provided when the next phase of the El Nasser Canal is completed. In other respects, the area has incompatibilities similar to those of the irrigated lands in Subarea A. These problems include the small farm sizes, somewhat poorly drained soils, mixing wastewater with water from the Nile, and controlling public contact with the wastewater. If existing agricultural facilties were to be incorporated into a wastewater system here, expensive, high-level pretreatment would be required to protect public health.

The presently unirrigated area south of the Bahig Canal extends in a narrow strip, about 6 km wide and 40 km long. The total land area is about 60,000 feddans and is mostly undeveloped. However, this site is generally unsuitable for the following reasons:

- About half of the area, 30,000 feddans, is unsuitable rockland and rockland-complex soils. The suitable soils occur mainly in small, noncontiguous areas, which would complicate operations and greatly increase costs.
- The planned city of New Amria is to be built in the middle of the area.
- An estimated 5000 feddans of the better soils have already been irrigated by individuals who pump water from the canal and shallow wells. Also, much of the better soil is used for dryland farming and livestock grazing.
- Drainage would be required to protect the more productive lands at lower elevations that are irrigated by the bordering projects.
- The area is inadequate for Alexandria's total wastewater flow.

Although Subarea D cannot be recommended for use as an irrigation site for Alexandria's wastewater, one 10,000-feddan part of the area may be suitable as a reuse site for New Amria's wastewater.

Subarea E includes the West Noubaria Extension, Reclamation and Settlement Project. The project covers about 90,000 feddans of irrigable land, mostly in the Abu Mena Basin. The soils in this area are generally suitable for wastewater irrigation. However, the ongoing development of the West Noubaria Project presents major obstacles to wastewater reuse. These include the following:

- Plans and construction of the West Noubaria Project are well underway, which would require high-level government and international financing organization decisions in order to be altered.

- Project plans (Ref. 13) call for the settlement of several thousand families on small 6-feddan farm plots. This would be incompatible with wastewater irrigation unless very high levels of pretreatment were provided.

- Project plans (Ref. 11) also call for extensive sugar beet and vegetable production, which would require high-level pretreatment. Sugar beet yields would be reduced by continuous wastewater nitrogen applications if primary effluent were used.

- Large areas of tight soils would increase the difficulties associated with high-TDS irrigation water.

The only practical possibility for land application at this site at this time would be if it could be made compatible with the ongoing reclamation project. Providing the high levels of pretreatment necessary to do this would be expensive and unreliable using conventional techniques, and would be impractical using rapid infiltration because of poor site conditions. Consequently, Subarea E appears to present little opportunity for wastewater irrigation and was excluded from further consideration.

Subarea F contains the largest area of undeveloped land within 100 km of Alexandria. Except for limited small and scattered areas, however. the soils are unsuitable for wastewater irrigation. Reconnaisance level soil surveys conducted by FAO (Ref. 14) and verified during this study are the basis of this evaluation. This information indicates that areas immediately south and southwest of the El Nasser Canal consist largely of plateau rockland (Mz mapping unit) and shallow, silty clay loam soils over rock (Pm 4/20, Lr, and Pn 4/21 mapping units). Land having soil characteristics favorable for wastewater irrigation is restricted to very limited areas where windblown sands have accumulated or where depression areas have been filled with silty clay loam soils.

Although detailed soil mapping is not available for locations further south and southwest of the canal, Elgabely, et. al. (Ref. 15),

classified the remaining area of Subarea F as limestone lithic ermo-lithosols. These soils consist only of a crust of physical weathering smoothed and polished by wind action. They have continuous rock just below desert pavement and were therefore eliminated from further consideration.

Subarea G is an undeveloped area located about 80 km south of Alexandria along the Desert Road. After preliminary evaluations, we concluded that this site has the most favorable properties for wastewater irrigation. The major advantages of this site are:

- Land area is sufficient to handle the total Alexandria wastewater flow.
- The site is mostly uninhabited and far from any villages.
- The site is readily accessible from the Desert Road.
- The sandy and well-drained soils are most suitable.
- The stratigraphic profile has few drainage restrictions.
- Depth to ground water is sufficient to maintain good drainage.
- Relative to other sites far from Alexandria, the elevation is low.
- The topography is hummocky and undulating but quite acceptable for irrigation, especially by sprinkler and drip methods.

Rapid Infiltration. A similar analysis of all seven subareas was performed to identify those areas with favorable characteristics for rapid infiltration. With the exception of Subareas C and G, none of the potential sites were found to have an adequate area of suitable soils for rapid infiltration. The southern portion of Subarea C is an area of deep permeable soils that could hold some possibilities for rapid infiltration. The area has several major limitations, however, including a very high existing water table, and the fact that it is presently used for irrigated agriculture.

Subarea G, on the other hand appears to be much more promising for rapid infiltration. It consists of open desert land with extensive areas of highly permeable soils, suitable ground water and geologic conditions, and outlets for beneficial reuse of the treated effluent.

Overland Flow. In the evaluation of potential sites for overland flow, Subareas A, B, C, and E were eliminated from further consideration for

many of the same reasons as for wastewater irrigation. These include: conflicting land use patterns, questions regarding land availability, inability to control public access, and an insufficient amount of undeveloped area. While the necessary slowly permeable soils do exist over much of these four subareas, their use for wastewater treatment would take 5000 to 28,000 feddans of good farmland out of production.

Subarea G was excluded from further consideration because the highly permeable soils, which are excellent for rapid infiltration, are not at all suitable for overland flow.

Some undeveloped portions of Subareas D and F have soil characteristics that are somewhat favorable for overland flow. These are generally found as shallow silt loams over slowly permeable clay loam subsoils. However, a major problem is that these soils are associated with unsuitable shallow and rocky soils and also occur in complexes with rockland. The suitable soils therefore occur in small, widely dispersed areas. With overland flow this is especially undesirable because both the wastewater distribution and runoff collection systems would be widely dispersed and therefore expensive. As much as three times the 5000-to-28,000 feddan net area may be required to develop an overland flow system in these areas.

Another major problem is that, as shown in the irrigation site screening, no suitable sites are near these locations for direct reuse of the treated effluent. As discussed earlier, overland flow cannot reliably treat wastewater to a suitable level for uncontrolled irrigation. Therefore the Bahig Canal and West Noubaria Project areas could not safely accept the overland flow effluent without further treatment.

Although no suitable locations could be found for a large-scale overland flow system, the process still has considerable merit as a treatment process for small communities or industries in the area.

Recommended Sites. The identification and screening of potential sites for land application are summarized as follows:

- Subarea A--Existing land use precludes any large-scale land application system.
- Subarea B--The Nahda extension is suitable for local treatment plant wastewater only, or as a location for a sludge composting facility or pump station.
- Subarea C--The government farming areas could be considered in the future for use of a portion of the Alexandria wastewater if the need arises. Limited opportunity for rapid infiltration in southern part.

- Subarea D--No areas are suitable for large-scale land application; however, small areas exist which might be used for local irrigation or overland flow systems.

- Subarea E--The currently planned West Noubaria Project precludes large-scale application; however, soil conditions are suitable for irrigation or possibly overland flow of locally generated wastewater.

- Subarea F--No large, contiguous areas of suitable soils could be located.

- Subarea G--Contains extensive areas of soils suitable for both irrigation and rapid infiltration.

Subarea G is the only location within 100 km of Alexandria that appears to have characteristics suitable for the land application of the entire wastewater flow being considered. In order to test this conclusion and the subjective evaluations made in this site screening analysis, a secondary analysis was performed after the major land application alternatives were developed (Ref. 55). In this analysis, a conceptual design was developed for a rapid infiltration system at Subarea C, as this was considered to be the second best opportunity for land application. In comparison, this system proved to have a slightly greater present worth cost but was much less desirable from the standpoints of technical feasibility, environmental impact, and reliability.

For convenience, Subarea G has been divided into an East and a West Site, each located on either side of Desert Road as depicted in Figure K-3. More detailed evaluations of both sites are summarized in the following two sections.

East Site

The East Site is the larger and the more favorable of the two sites. It is located approximately 80 km south of Alexandria in an area between the El Nasser and Noubaria Canals, and the Desert Road. A total of about 100,000 ha (240,000 feddans) was investigated. Site information was obtained from a variety of published sources (Refs. 14-30), and was verified by reconnaissance-level field investigations, laboratory analyses, and discussions with local experts.

Land Use. The East Site consists primarily of open desert terrain. It is bounded on the northwest and north sides by existing or planned irrigated agricultural areas. At this time, the site is mostly uncommitted for

reclamation or development, with the exception of one small area along the northwest edge of the site, which is allocated for an irrigation project but not yet developed. An air force base is located beyond the northern corner of the site and a railroad is presently being constructed along the north side. Other physical features such as canals, road drains, and power lines, which are on or near the site, are shown in Figure K-4. The site is very sparsely inhabited by Bedouin shepherds, who are concentrated primarily to the north near the Noubaria Canal.

Topography. The topography of the East Site is presented in Figure K-4. Ground elevations range from 5 to 35 m above sea level. In general, the site slopes toward the northeast at approximately 0.1 percent. The topography is undulating throughout the site and is characterized by isolated topographic highs (where windblown sand has accumulated) and topographic lows (where desert pavement, limestone, and lime concretions predominate). Topographic depressions can be as large as several feddans in area. In places, particularly the east-central portion, topography can be extremely rugged. More detailed topographic information will be required prior to design.

Soils. The soils of the East Site consist largely of windblown, sandy soils that exhibit medium-high to low dune topography. The sands are derived from sandy, river flood plain and deltaic deposits found to the southeast. Finer submarine sediment deposition occurs to the northwest. Soil characteristics, depth, and texture reflect differences in geologic parent material, and particularly in how the parent material was deposited in the environment (Ref. 14). In addition, soil-forming processes appear to be affected by minor topographic changes.

A soils maps of the East Site is presented in Figure K-5. Soil mapping units include Db, DB 11, Ds, Ds 11, Ds 12, Ds 18, Ds 25, and Du 18. Physical and chemical properties of these soils are summarized in Tables K-8 and K-9. Additional detailed soils information is contained in the soil survey report prepared by the FAO (Ref. 14).

Db soils are dune sand soils with undulating to rolling relief. In a typical setting, a few "valleys" lie between the dunes where underlying subsoil is exposed. The windblown soils are loose medium-fine to medium-coarse sand. The soil is generally poor in lime, but on occasion can be slightly indurated (Ref. 14).

Db 11 soils are texturally comparable to the Db soils. However, approximately 50 percent at the Db 11 soils are occupied by wide, gently undulating valleys where the underlying soils are exposed. These valleys are only partly covered by sheets of windblown sand or dunes. Valley soils range in texture from clay loam through loamy coarse sand and often contain lime concretions and may contain cemented layers in some areas.

Cemented layers can form a surface crust, and surface lime concretions appear to be remains of disintegrated crusts (Ref. 14). The extent and thickness of these crusts beneath higher elevation dune locations is unknown at this time. Field observations suggest that crusts are often associated with poorer drained locations. Uncertainties regarding the extent and thickness of lime cementation may result in some drainage problems. Drainage appears to be dependent on the degree of cementation, presence of cracks, and ability of irrigation water to leach calcareous salts through the soil profile. Further field investigations will be required to confirm the extent and thickness of cementation.

In the Db association (Db and Db 11 soils) permeabilities are believed to be high in noncemented locations. Infiltration rate estimates are 1 to 3 cm/hr, 4 to 16 cm/hr, and 20 cm/hr for clay loam, sandy loam and loose sand, respectively (Ref. 16). Typical dune location infiltration rates are probably closer to 20 cm/hr. Saturation percentage ranges from 18-66 percent, and decreases with increasingly coarse texture (Ref. 14). Field moisture content is estimated at 6-22 percent, indicating limited moisture retention properties (Ref. 19). Permanent wilting point is estimated at 3 to 11 percent (Ref. 19). Soil pH ranges between 6.9 and 8.0 and is suitable for general crop growth, retention of trace elements, and nitrification-denitrification reactions. Published values indicate that saturation extract conductivity may exceed 4 mmhos/cm and sodium adsorption ratios may exceed 15 (Ref. 14), making the soils both saline and alkaline in some areas (Ref. 15). Replacement of exchangeable sodium (using gypsum and/or acid forming amendments) and leaching may be required prior to crop planting. The effect of sodium on soil permeability would be most significant in the finer textured soils. Alkaline soils are defined in terms of exchangeable sodium percentage, a value that is generally equivalent to the sodium adsorption ratio (Ref. 19). However, sodium replacement may not be critical. Additional analyses will be required prior to design to better define soil chemistry and confirm reconnaissance-level field observations that suggest that published values of saturation percentage, saturated extract conductivity, and sodium adsorption ratio are overestimated. Lower range values are probably most representative for this and all soil mapping associations discussed in this appendix. Lowest values tend to be associated with best drained locations.

Ds are dune sand soils usually with gently undulating to undulating relief. In a typical profile, the underlying soils (sometimes loamy) are rarely exposed. These dune sands are slightly cemented (though not necessarily drainage-restrictive) with a surface covering of fine and medium gravel (Ref 14).

Ds 11 soils are texturally comparable to Ds soils. However, the underlying original soils are exposed in over 30 to 40 percent of the

area in gently undulating valleys. Valley soils include compact coarse loamy sand, sandy loam, and (seldom) clay loam. Valley locations are high in lime and can be completely cemented (Ref. 14). The extent, thickness and origin of these deposits need to be addressed prior to design.

Ds 12 soils occupy transitional locations between Db dunes and deltaic soils to the southeast. Dune and valley locations are generally loamy sands. Valley soils, which occupy 40 to 50 percent of the area, contain soft and hard lime concretions and limestone fragments (Ref. 14).

Ds 18 soils form low sand sheets or low ripple dunes. The relief is nearly flat to undulating. Hard lime concretions are often noted in the surface. The soil profile is very rich in lime and can exhibit partial cementation (Ref. 14).

Ds 25 soils differ from Ds soils only in relief. They are found in higher dune locations. Field observations suggest the presence of lime concretions in valley locations.

Soils of the Ds association have characteristics similar to those of the Db association. Relatively high permeabilities are expected in noncemented locations. Infiltration rate estimates are 1 to 3 cm/hr for sandy clay loams and 20 cm/hr for loose sands (Ref. 14). (Rates up to 40 cm/hr have been reported for similar soils in the area south of the current site boundaries, the South Tahrir project area). Infiltration rates that are significantly less than 1 cm/hr are expected for cemented profiles (Ds 18, for example). Saturation percentage ranges from 17 to 73 percent. Estimated field moisture percentages and permanent wilting point range from 6 to 24 percent and 3 to 12 percent, respectively (Ref. 19), suggesting limited moisture retention properties. Finer textured soils exhibit the greatest water-holding capabilities. Soil pH ranges from 6.8 to 8.4 and appears suitable for general crop growth, retention of trace elements, and nitrification-denitrification reactions. Electrical conductivity and sodium adsorption ratio values exceed 4 mmhos/cm and 15, respectively, in some areas, making the soil both saline and alkaline (Ref. 19). However, other combinations of salinity-alkalinity relationships are observed. Soil reclamation procedures would be dependent on local condit[illegible] For Ds 11 soils, simultaneous increases in sodium adsorption rate [illegible]d saturation percentages suggest some areas of locally restrictive drainage (probably associated with local cementation). The effect of sodium on soil permeability is limited, due to a generally coarse soil texture.

Du 18 soils show little distinct dune relief, but scattered dunes are present nearer to Db and Ds soils. Depth of sand reworking ranges from 20 cm to 300 cm. Below this depth the sand becomes more loamy and

cemented by lime, reflecting a probable submarine, parent material deposition (Ref. 14).

Permeabilities and infiltration rates in Du 18 soils vary according to textures. Infiltration rates probably range from 1 to 20 cm/hr, depending on the degree of near-surface and subsurface lime cementation. Soil pH ranges from 7.2 to 7.9 and is suitable for general crop growth, retention of trace elements, and nitrification-denitrification reactions. Within this association, local conditions will dictate the combinations of sodium replacement-leaching techniques that are required to maintain crop yield. The presence of sodium (and its effect on permeability) is most significant here due to this association's generally finer texture.

Nutrient analyses are not available for any soil association, and the need for crop amendments cannot be evaluated at this time. However, low levels of organic matter, nitrogen, and available phosphorus are generally associated with soils of desert regions (Ref. 23). A favorable crop response to wastewater applied nutrients is therefore expected. Additionally, the extent of salt accumulation throughout the soil-geologic profile is unknown, making estimates of salt transport inappropriate at this time.

Geology. The East Site lies within the deltaic plain of the Nile River. The stratigraphic sequence reflects both aggradational and degradational stages that accompanied changes in sea level during Pleistocene and Recent times (Refs. 14, 17 and 20). Surficial geology has been modified by the importation of windblown sands. Stratigraphic information (Ref. 20) indicates sand and gravel accumulations well in excess of 100 m through the middle portion of the site. Thin clay stringers are also noted in drilling logs. Sand and gravel thickness decreases westward toward the Desert Road, but is still approximately 100 m deep. At the extreme northeastern end of the site, a relatively thin clay lens is found at a depth from 20 to 30 m below ground surface. The entire site is underlain by clay (or shale) at depths ranging from 100 to 300 m.

Ground Water. The East Site lies above the Tahrir Aquifer, an unconfined aquifer composed of the same sands and gravels discussed previously in the geology section. Depth-to-water-table contours are presented in Figure K-6. As is observed from the figure, depth to water is influenced by seepage from the Noubaria and El Nasser Canals and increases to 30 m. Ground water flows in a south-southwesterly direction away from the canals (Refs. 24 and 25). Changes in ground-water elevation and flow direction are expected with the startup of the local irrigation projects. Hydraulic conductivities are estimated between 15 and 65 m/day. Ground-water slope ranges from 0.001 to 0.005. Flow velocities are estimated between 0.09 and 0.39 m/day.

Ground-water salinity increases in a southwesterly direction from 0.6 mmhos/cm in the east to nearly 2.0 mmhos/cm in the west. The ground

water of the Tahrir Aquifer is used for both domestic and agricultural supply. Its value will increase with the development of Sadat City, which is to be located about 30 km south of the East Site.

Vegetation and Wildlife. Desert vegetation at the East Site thrives on low dunes and sand sheets where windblown soils have sufficient water-holding capacity. Vegetation is generally not found on rocky or cemented soils of exposed valley areas or on the shifting sands of high dunes. Vegetation throughout the East Site consists mainly of desert shrub (mitnam) and camel thorn (Ref. 14). Mitnam bushes (Thymalea sp.) are scattered throughout the gently undulating terrain of windblown sands. This perennial bush is characterized by a woody stem, smooth bark, succulent leaves, and yellow flowers (March-April). Terrestrial snails (Helix sp.) are found on mitnam, and empty shells are scattered throughout the terrain. Mitnam is spaced from 1.5 m to 5 m apart in the areas favorable for growth. Camel thorn, a small thorny shrub, is interspersed between the mitnam.

Wildlife is typical of desert Egypt and includes fox, rabbit, birds, and other small animals. Sheep and goats are also raised throughout the area by the Bedouins. Cheetah have been spotted on rare occasions.

Historical and Cultural Features. Eleven tombs of potentially historical interest have been located on the East Site (Ref. 22). Most are in the north and southwest and some are already impacted by the irrigation-reclamation projects discussed previously.

Suitability and Limitations. Site suitability and limitations are summarized in Tables K-10 and K-11. Site characteristics are generally favorable for either rapid infiltration or wastewater irrigation. Site topography is generally level and areas of deep, well-drained soils are present. Few drainage restrictions are noted in the stratigraphic profile, and depth to ground-water is sufficient to maintain unsaturated flow in the top metre of soil. The majority of the site is available and sparsely inhabited.

Design limitations (and questions warranting further investigation) include the extent of lime cementation, presence of clayey subsoils, distribution of soil salinity-alkalinity, location of shallow ground-water table and maintenance of ground-water integrity. Additionally, site layout will have to make accommodations for physical structures, population centers, public access, wildlife, and sites of historical interest. These limitations would be addressed in a detailed field investigation program prior to final design.

West Site

The West Site is located south of El Nasser Canal and west of the Desert Road approximately 80 km from Alexandria (Figure K-3). A total of about 47,000 ha (113,000 feddans) was investigated. Site information was obtained from mostly the same sources as for the East Site, including a variety of published sources (Refs. 14-30) with reconnaissance-level field verifications and discussions with local experts.

Land Use. The site consists primarily of open desert terrain bounded on the north by a growing area of irrigation-reclamation projects served by the El Nasser Canal. It is very sparsely inhabited by Bedouin shepherds. For the most part, the site is uncommitted for reclamation or development, although several areas to the north, which are already allocated for irrigation projects but not yet developed, are also included within the site. Physical features such as canals, roads, drains, and power lines which are on or near the site are shown in Figure K-7.

Topography. The topography of the West Site is presented in Figure K-7. The eastern portion of the site is generally flat to undulating and ranges in elevation from 30 to 40 m above mean sea level. Several islands of higher elevation (>40 m) are located in the southeast portion of the site. The topography of the northwest corner is characterized by a depression which drops to less than 10 m elevation. Slopes on the depression's eastern boundary exceed 1 percent and can be significantly greater locally. More detailed topographic information will be required for design.

Soils. Like the East Site, the soils of the West Site consist largely of windblown, sandy soils that exhibit dune topography. The sands are derived from sandy river flood plain and deltaic deposits found to the southeast. Finer, submarine sediment deposition occurred in the north and shallow marine and lacustrine deposition occurred in the west and northwest sectors of the site. Soil characteristics (texture, depth) reflect differences in parent material, particularly in transitional depositional environments (Ref. 14). Additionally, changes in pedogenic processes appear to be associated with topography.

A soils map of the West Site is presented in Figure K-8. Soil mapping units include DB 11, Ds 11, Ds 12, Du 18, Pl 20, and Pm 4/20. Physical and chemical properties of these soils are presented in Tables K-12 and K-13.

Characteristics of the Db 11, Ds 11, Ds 12, and Du 18 have previously been discussed for the East Site. Pm 4/20 soils generally lie lower in the terrain. The soils are shallow to moderately deep over limestone bedrock.

Some soils are very shallow, and rock occurs at the surface. Rock-free portions of the soil have a deep to very deep compact silt loam to clay loam profile, and are very rich in lime. Cementation increases in depth.

Infiltration rates in the Pm 4/20 soils are believed low (1 cm/hr), particularly where bedrock surfaces or the profile is heavily cemented. In areas where significant accumulations of soil exists, soil pH is suitable for general crop growth, retention of trace elements, and nitrification-denitrification reactions. Nonsaline-nonalkaline and saline-alkaline conditions exist within the soil series. The influence of sodium on soil permeability may be significant in these finer textured soils.

Pl 20 soils range in texture from very deep to shallow. Within the West Site they are generally associated with areas of near surface limestone bedrock. The soils are medium-textured (coarse sandy loam or loam). Deeper substrata consist of grayish loam, presumably of lacustrine origin. Hard and soft lime concretions occur at depths from 20 to 80 cm, increasing in quantity and hardness with depth (Ref. 14).

Infiltration rates coincide with differences in soil texture. Sandy topsoils have estimated infiltration rates in excess of 20 cm/hr, while subsoils are often too compact for measurable infiltration. Where subsoil measurements are possible, rates between 10 and 50 cm/hr have been recorded, with higher values being generally associated with subsoil cracks. The soils are saline-alkali, with a sodium adsorption rate of up to 25 and an electrical conductivity of approximately 10 mmhos/cm (Ref. 14).

As with the East Site, nutrient analyses are not available, and the need for crop amendments cannot be evaluated at this time. However, low levels of organic matter, nitrogen, and available phosphorus are generally associated with soils of desert regions (Ref. 23). A favorable crop response to wastewater-applied nutrients is expected. Additionally, the extent of salt accumulation throughout the soil-geologic profile is unknown, making estimates of salt transport inappropriate at this time.

Geology. The West Site lies within the deltaic plain of the Nile River. The stratigraphic sequence reflects both aggradational and degradational stages that accompanied changes in sea level during the Pleistocene era and recent times (Refs. 16, 17, and 20). A surficial geology map shows sand sheets and dunes east of the 30-m topographic contour (see Figure K-7), and sandy limestone, limestone, sandstone, and gypsiferous sandy clays (Duricrusts, Magif, Solymanya and Muluk Formations) west and approximately parallel to the 30-m topographic contour. The near-surface distribution of these rock formations results in increasingly shallow soils west from the Desert Road.

Stratigraphic data (Ref. 20) indicate 100 m of sand and gravel (aeolian and deltaic), along the Desert Road. Clay stringers are also noted on regional driller's logs (Ref. 30).

Ground Water. The West Site lies above the West Noubaria Canal aquifer system, a semi-confined aquifer composed essentially of deltaic, permeable deposits. The aquifer is confined by semipervious clays and carbonates (aquitard) near the surface, and impermeable marine clays below. The thickness of the aquifer decreases in a northwesterly direction and becomes less confined as it approaches the Tahrir aquifer system (toward the Desert Road). As shown in Figure K-9, depth to ground water ranges from approximately 30 to 50 m below ground surface and increases as water flows in a generally southwesterly direction below the site (Ref. 24). Elevation changes are anticipated with the extension of the El Nasser Canal and other local irrigation projects.

Coefficients of hydraulic conductivity are estimated at 25 m/day for the aquifer and from 1 to 10 m/day for the aquitard (includes clayey sands, limestones, and sands).

Ground water is of poor quality, ranging from 0.7 to 10 mmhos/cm over the site, and decreases in quality in a southwesterly direction.

Vegetation and Wildlife. Vegetation and wildlife descriptions for the West Site are similar to those for the East Site.

Historical and Cultural Features. Three tombs of potentially historical interest have been located on the West Site (Ref. 22). They are generally in the northwest portion of the site where soil conditions are less favorable for land treatment.

Suitability And Limitations. Site suitability and limitations are summarized in Tables K-14 and K-15. In the central and eastern portions of the site, site characteristics are generally favorable for either rapid infiltration or wastewater irrigation. At these locations site topography is generally level, and areas of deep, well-drained soils are known to exist. Limited stratigraphic information is available, but it suggests few drainage restrictions through depth to unconfined ground water. The majority of the site is available and sparsely inhabited.

Design limitations throughout the site (but particularly in the western part) include the extent of lime cementation, depth to bedrock, presence of clayey subsoils, distribution of soil salinity-alkalinity, and locations of potentially perched ground water above subsurface aquitards. Surficial bedrock eliminates depression areas from further consideration for rapid infiltration or irrigation sites. However, wastewater or pumped

ground-water storage facilities in these locations remains a definite possibility. In addition to the above limitations, physical structures, minor population centers, public access, wildlife, and sites of historical interest will have to be taken into account during design.

Recommendations

The preliminary site investigations clearly indicate that site characteristics, particularly soils and geology, favor land application at the East Site over the West Site. Therefore, the East Site is recommended as the primary site, with the West Site being considered as a backup if the East Site becomes unavailable. Requirements for the detailed field investigation and analyses that must be conducted prior to design are presented in Table K-16.

Suitable locations for the rapid infiltration system are identified in Figure K-10. In those areas identified as suitable, the combination of generally level topography; deep, high infiltration rate soils; and a water table in excess of 5 m below ground surface provides a favorable environment for rapid infiltration. These areas also provide the most favorable conditions for wastewater irrigation. The combination of undulating topography and high infiltration rates favors overhead irrigation as light, frequent irrigations are best for sandy soils. Deep ripping, salt leaching, sodium replacement, leveling and/or drainage may be required in some areas where wastewater irrigation boundaries extend into less suitable soils. Similar procedures may also be required in valley locations of the Db, Ds, and Ds 25 soils.

DEVELOPMENT OF LAND APPLICATION ALTERNATIVES

The objective of the land application study was to develop one or several alternatives in sufficient detail to be compared to other alternative methods of treatment and disposal of Alexandria wastewater. In so doing, an attempt was made to identify the best combination of technical feasibility, overall cost, beneficial reuse of wastewater, reliability, and protection of public health. Based on the results of the preliminary field investigations, conceptual designs were developed for two alternative systems that would accomplish these objectives to varying degrees. Both would utilize the East Site for irrigation and would be comprised of the components shown schematically on Figure K-11.

The first alternative, designated IIIA, is a combination of rapid infiltration and irrigation. It would consist of pretreatment by anaerobic lagoons followed by rapid infiltration for additional high-level treatment and storage. The soil and ground-water conditions at the site are such that all of the excess wastewater flows during the

winter could be stored in the ground water for subsequent recovery during the peak irrigation season in the summer. Once recovered, the ground water would be of high quality, with few limitations for crop irrigation. Consequently, high value crops could be selected that would maximize agricultural returns to the system. Because of the highly permeable sandy soils and undulating topography at the site, sprinkler irrigation by center pivots was selected as the optimum means of distribution.

Alternative IIIB, the second land application, entails irrigation of forage crops following primary level treatment. It would involve significantly less capital cost than the first alternative but would provide fewer benefits and less crop return to offset the annual cost of the system. Pretreatment would again be by anaerobic lagoons but with additional emergency storage capacity. No seasonal storage would be provided, so overirrigating in the winter and removing some land from production in the summer would be required. With only primary level treatment, the selection of crops would be limited to forage and other nonfood crops. As with the other alternative, irrigation would be by center pivot sprinklers.

The major concepts of each of these alternatives are described in the following subsections. A short discussion of other design options that are less feasible than Alternatives IIIA and IIIB but should nevertheless be considered is also included.

Alternative IIIA

Alternative IIIA is the best technical alternative that was identified. It would provide essentially complete wastewater treatment together with effective reuse of the water. Because these two aspects of the system are so interrelated, they are initially considered together. It may be possible, however, to subsequently split some of the responsibility for development and operation.

Conceptual design data for the major components of the system from pretreatment through irrigation distribution and drainage are summarized in Table K-17. Each of the major components is described below. Following this are discussions of the possible management and operation of the system and the associated infrastructure that would be required.

Pretreatment. Initial pretreatment by anaerobic lagoons would be provided in order to optimize the design of the rapid infiltration system. With high organic strength wastewater from Alexandria, the organic loading would be the critical design parameter for rapid infiltration. Consequently, reducing levels of BOD and suspended solids would permit a higher application rate and more design flexibility.

Anaerobic lagoons were selected as the most appropriate process because they can provide the required level of treatment simply and inexpensively. They are designed for high-strength wastes with depths deep enough to maintain anaerobic conditions throughout the lagoon contents. Like anaerobic sludge digestion, they depend on a balanced two-step treatment mechanism in which acid-forming bacteria convert complex organic material into volatile acids. These are then converted into methane and carbon dioxide by methane bacteria. This process, referred to as alkaline fermentation, often takes 3 to 6 months to become fully established but, once established, provides very effective treatment and odor control (Refs. 17 and 30).

Nevertheless, mild objectionable odors are a common occurrence with anaerobic lagoons; consequently, the lagoons proposed for this system would be located in the interior of the site and isolated from any development.

The key design parameter for anaerobic systems is the detention time. There is a wide range of opinion in the literature as to what this should be, with recommendations ranging from 2 to 40 days (Refs. 33, 34, and 35). Most of the treatment in anaerobic lagoons takes place in a very short time, mainly from settling of suspended solids with subsequent digestion of the sludge. Experience with other municipal systems with conditions similar to those in Alexandria indicates that detention times of 2 to 5 days will provide consistently adequate pretreatment for the rapid infiltration system.

Other methods of pretreatment were also considered; however, most of them would provide a higher level of treatment than required and all would be more expensive. One of the options investigated was facultative lagoons, which would provide reliable treatment with less potential for offensive odors. Although BOD and suspended solids removal would be considerably greater than with anaerobic lagoons, the algae that would be produced could plug the rapid infiltration basin surfaces, causing reduced infiltration rates. Land area requirements would also be considerably greater for facultative systems. Assuming a typical design surface loading of 200 kg of BOD/ha/day, approximately 2,800 ha would be required, compared to approximately 210 ha required for anaerobic lagoon treatment. The questionable benefits are not justified by the additional land requirement and resultant cost increase which would be incurred.

Another possibility that was considered was locating the pretreatment facilities near Alexandria.

If some form of lagoon treatment were used it could also provide flow equalization, thus reducing conveyance costs. With anaerobic lagoons,

odors would be a much more serious problem near developed areas and would require large buffer areas around the site. Other forms of lagoon treatment would also require large amounts of land, which can cost as much as LE 10,000/feddan in the agricultural areas near Alexandria. For these reasons it was decided to locate the pretreatment system at the land application site and provide offline storage of daily peak flows near Alexandria.

One other advantage of locating anaerobic lagoons at the land application site is that the anaerobically digested sludge could be applied directly to the agricultural lands. With high-strength wastewater and short detention times, the anaerobic lagoons could become half-filled with sludge as quickly as every 2 years. They would be designed with enough excess capacity that they would be taken out of service when they reached that point, dried for 6 months, and then emptied by truck hauling over an additional 18-month period.

Rapid Infiltration. An extensive area of Ds, Ds 25, and Db soil series in the East Site are suitable for rapid infiltration. The soils are deep and the ground-water conditions are such that it should be possible to effectively control the percolated water and use it to provide seasonal storage for irrigation.

No actual measurements of the infiltration rates of these soils have been taken, although, based on the textural characteristics, the infiltration rate for clear water has been conservatively estimated at 20 cm/hr. With a rate as high as this, the organic loading will be the critical design parameter. If the BOD loading is limited to 225 kg/ha/day the resulting hydraulic loading would be 27 m/yr. This would be [illegible] percent of the estimated infiltration rate for clear water and is well within normal design criteria.

Because the percolated water is to be directly reused for irrigation, high levels of nutrient removal are not necessarily advantageous. Phosphorous removals should be quite high because of the calcareous soil conditions and long distances of wastewater travel through the soil. A total phosphorus concentration of 2 to 3 mg/l in the percolate is probably the maximum that can be expected. Nitrogen removals should also be quite high as a result of the large ratio of organic carbon to nitrogen. This can be minimized by keeping flooding periods as short as possible. In this case an application cycle of 1 day of flooding followed by 6 days of resting has been assumed for preliminary design, but even so, nitrogen removals greater than 50 percent can be expected.

The annual storage requirement for a typical crop pattern of high-value crops is calculated to be about 19 percent of the total flow. This would result in a ground-water fluctuation of about 12 m over the

year. With an extensive network of recovery wells, the ground-water mound could be managed such that essentially all of the applied water would be recovered for irrigation and an adequate unsaturated zone is maintained at all times beneath the infiltration basins.

Figure K-12 shows a cross section of the rapid infiltration and ground water recovery system in operation. The cross section runs directly north-south through the site as indicated in Figure K-13. The ground-water profile is taken from 1977 depth-to-water-table contours presented in Figure K-6. As can be seen in the cross section, the ground water flows in a southerly direction from a high point in the north (which is influenced by the El Nasser Canal). During the winter months when storage requirements are maximum, the ground-water mound would be allowed to rise beneath the basins. Irrigation supplies would be drawn primarily from perimeter wells, thus maintaining a negative ground-water gradient into the system. During the summer months, the center wells would do most of the pumping, thus creating a ground-water depression while minimizing inflow from the surrounding areas.

One option that has been suggested would be to design the rapid infiltration completely independent of the ground-water recovery and irrigation systems. This might be easier to implement and would be advantageous if the wastewater treatment and agricultural operations were two completely separate responsibilities. By spreading the infiltration area over the site instead of in one concentrated location, it may be possible to design the system such that the ground-water recharge mounds might dissipate naturally in the event that no recovery of water took place. However, this would add to the overall cost of the project, sacrifice some efficiency of ground-water recovery, and could also create adverse ground-water conditions down-gradient from the site. Therefore this option was not considered further.

The envisioned distribution system would be a network of concrete-lined canals leading from the anaerobic lagoons to the infiltration basins. Because of the short flooding periods that are proposed, applications would be limited to about an 0.5-m depth of water. Consequently, the basins could be quite shallow. The recovery wells would be located on the dikes between basins and in a ring around the rapid infiltration area. Two methods of collecting the recovered water are possible. In the first method, the wells would discharge directly into a pipe network that would be part of the irrigation distribution system. Alternatively, if canals were utilized for irrigation distribution mains, the wells could discharge directly into canals. The first of these would be much simpler and more reliable than the second but would also be significantly more costly.

Irrigation. Alternative IIIA is designed to maximize crop production and net returns. Since the recovered ground water would be of essentially

drinking water quality, it would not limit crop selection except in regard to salt tolerance. Only crops that will not suffer a large yield decrease with irrigation water of 2.3 mmhos/cm electrical conductivity should be considered.

Selection of a specific cropping pattern requires a detailed analysis of soil conditions and wastewater quality as well as crop marketability, yield, value, and production costs. Although much of this analysis is beyond the scope of this study, some generalizations can be made based on available data. For purposes of conceptual irrigation system design, a cropping pattern consisting of mainly vegetables and berseem has been assumed. Many other high value crops could also be grown and should be thoroughly evaluated prior to design. These include fruit and lumber trees, fiber, sugar, and oil crops.

In this cropping pattern, mainly berseem would be grown on 50 percent of the area during the winter. Vegetables would be grown on most of the other 50 percent during this time and over the entire area during the summer. The monthly irrigation requirement is listed in Table K-18 and is based on the following computation:

$$IR = \frac{CU - ep}{E} \times (1 + LR)$$

Where: IR is the irrigation rate, CU is the weighted average consumptive use for berseem and composite vegetables from Table K-5, ep is effective precipitation from Table K-5, E is the application efficiency for sprinkler irrigation of 80 percent, and LR is the leaching requirement of 20 percent. The total annual irrigation requirement for the high income crop pattern is 1,491 mm/yr.

The nitrogen and phosphorus application rates and normal uptake rates for berseem and vegetables are presented in Table K-19. The high end of the range in uptake rates represents the maximum amount of nutrients for which the crop responds with increased yields. The lower end of the range represents typical application rates where the fertilizer must be purchased by the farmer. The amount of applied nutrients is based on nitrogen and phosphorus contents in the recovered water of 10 mg/l and 2 mg/l, respectively. The individual irrigation rates for each crop were used in the computation.

The proposed irrigation rate for berseem will supply about 80 kg/ha of nitrogen, which is about half of what the crop can use. However, since berseem is a legume, it can take nitrogen from the atmosphere when there is not an adequate amount in the soil. Some supplemental phosphorus may be needed.

The average wastewater irrigation rate used for the vegetables will supply about 160 kg/ha of nitrogen. This should be adequate for the needs of most possible crops. Likewise, the phosphorus application rate falls within the normal range of requirements for vegetable crops. In general, no supplemental nitrogen or phosphorus should be needed.

In order to maximize crop production in Alternative IIIA, storage of the percolated water is provided in the ground-water system so that it can be reused in accordance with irrigation requirements as they vary over the year. The water balance for the assumed crop pattern is presented in Table K-20. On the basis of this, a storage capacity of approximately 18.6 percent of the annual flow will be required.

The combination of undulating topography and highly permeable soils at the East Site favors sprinkler irrigation methods over surface irrigation. Sprinkler irrigation delivers water more efficiently; can be more easily adjusted to meet crop consumptive use demand and leaching requirements; and can more easily apply the light, frequent irrigations required on sandy soils. It also minimizes the need for land leveling and permits the irrigation of steep or undulating lands without the hazard of soil erosion or excessive water loss. Runoff collection or recycle ditches are not required as is the case for surface irrigation.

The most feasible and least expensive method of sprinkler irrigation for these conditions is low pressure, downward spraying center pivots. These would be similar to those that are currently in use or being installed in the newly reclaimed agricultural areas near the East Site along the Desert Road. The relatively large orifice nozzles and low pressure sprinklers on the proposed center pivots have the advantage of requiring less pumping head and the ability to pass larger solids.

Although center pivot irrigation is considered the method best suited for the conditions in this alternative, further consideration of other distribution methods is warranted. One good possibility would be to use drip irrigation for trees or selected high-value crops in areas between the center pivots. The quality of the wastewater would be sufficiently high after rapid infiltration and the pumping and pipe distribution network could be the same as that for the center pivots. Another possibility would be to use surface irrigation in some of the flatter areas of the site if conditions warranted. Neither of these two possibilities would greatly affect the overall economics of the system so they are left for further investigation in the design phase.

Several designs are possible for distributing the water to the center pivot rigs. As noted in the discussion on rapid infiltration, the distribution system could be a comprehensive pipe network directly from the recovery wells or it could consist of canals from the wells to

numerous distribution pump stations. Although the pipe distribution system has the advantages of simplicity and reliability, the canal system was chosen for the preliminary design because it is considerably less expensive.

A conceptual layout of the irrigation system is illustrated on Figure K-13. The layout shows collection canals for recovered water from rapid infiltration and the irrigation distribution mains. The irrigated area north of the rapid infiltration basins can be served totally by gravity; however, several lifts are required in the canals serving the southern area. The canals are designed to serve 20 distribution pump stations scattered over the site from which the water would be pumped to approximately 24 center pivots each. A typical distribution zone, or area served by one pump station, is shown on Figure K-14.

Drainage. The soils at the East Site are mostly very well drained; however, analysis of available data and preliminary field investigations indicate some local drainage problems. These could result from lime cementation or clayey subsoils. Development of a shallow groundwater table is possible in the extreme northern part of the site. This could cause some problem with drainage. Lacking a detailed geotechnical investigation, determination of exactly where potential problems might develop and how extensive they might be is not possible. For conceptual design and cost estimating purposes, we have assumed that one area in the north and another in the south of the site, comprising 20 percent of the total area, would require tile drainage. Main collector drains are provided along the northwest and northeast edges of the site.

Management And Operation. The land application system would involve a very large-scale wastewater and agricultural operation. A management program must be developed along with physical development plans to ensure efficient and reliable operation at a reasonable cost. Several major options are available:

- A/GOSD could manage and operate the entire system.
- A/GOSD could retain overall management responsibility and operate all of the wastewater system but contract the farming operation to private or public companies.
- A/GOSD could manage and operate the system only up through rapid infiltration. The irrigation and agricultural operations would come under the responsibility of some other agency.

The organization that is envisioned for this system would be comprised of a general administration office located at the site; treatment,

irrigation, and support services departments; and an office for coordination with the farming operation.

The general administration office would be responsible for overall management of the system, communications and coordination with A/GOSD headquarters, and business-related functions such as purchasing, accounting, and clerical services. The staff would include a general manager, a small engineering and technical group, and accounting and office support personnel.

The treatment department would be responsible for the operation and maintenance of the anaerobic lagoons, rapid infiltration basins, and recovery wells. Some key areas of responsibility include:

- Removing and hauling sludge from the anaerobic lagoons
- Maintaining and periodically disking the rapid infiltration basin surfaces to ensure continued high infiltration rates
- Managing the stored ground water through scheduling the recovery well operation
- Coordinating the recovery well operation with the irrigation department

The irrigation department would operate from a central control facility and a number of operations centers located throughout the site. It would be responsible for operation and maintenance of the distribution system including all pump stations, the center pivots, and the drainage system. In addition, it would be responsible for the day-to-day coordination of irrigation schedules with the farming operation.

The support services department would provide the following general services:

- Maintenance of the site including roads, ditches, fences, windbreaks, and buildings
- Maintenance of vehicles, portable equipment, pumps, valves, motors, and utility systems
- Procurement and storage of replacement parts
- Monitoring
- Control of public access

The department's facilities would include an environmental testing laboratory and automotive, mechanical, and electrical maintenance shops.

A key to successful operation of the land application system would be effective coordination with the farming company. A contractual arrangement would have to be developed which clearly specified areas of responsibility, quantities of water that the farming company would be obliged to take, irrigation scheduling, and selection and limitations on crops. A contract administration and liaison office as part of the A/GOSD management organization would be advisable to ensure that coordination is maintained at all required levels between the wastewater and farming operations.

Infrastructure. A land application system and related agricultural operation of this magnitude will require a large direct and indirect labor force. Since the project is far from any populated areas, complete new communities and service industries would have to be developed in the area to accommodate these people. In addition, new crop processing and transportation facilities would be required to serve the large-scale agricultural operation.

These infrastructure facilities must be designed and developed in close coordination with the land application system. However, most of the costs and benefits involved relate primarily to the new agricultural activity that would be created and are not directly attributable to wastewater treatment and disposal. Consequently, development of the infrastructure is considered only in general terms at this stage.

Alternative IIIB

Alternative IIIB is similar in many respects to the previous alternative except that rapid infiltration is not included. Without rapid infiltration, neither high levels of pretreatment nor seasonal storage of wastewater to match irrigation requirements are economically warranted. Consequently, Alternative IIIB entails the use of primary effluent for irrigating forage crops, with appropriate controls to safeguard public health. In order to match irrigation requirements with wastewater flow rates, crops would be overirrigated in the winter and a portion of the agricultural area would be taken out of production in the summer.

Table K-21 summarizes conceptual design data for the major components of the system. These are described in the following sections.

Pretreatment. Pretreatment is provided in this alternative primarily to reduce suspended solids in order to avoid gross solids deposition in the

irrigation distribution system and plugging of sprinkler nozzles. Anaerobic lagoons were again selected for this purpose for many of the same reasons as in the previous alternative. The design, however, would be different in several respects.

Because no additional treatment would follow anaerobic lagoons, a two-stage process is proposed. This would improve the overall treatment performance of the system and increase reliability. Two sets of lagoons connnected in series with detention times totaling about 9 days would be provided. In addition, a storage capacity of about 5 days' flow would be built into the system for emergencies and for equalization of minor fluctuations of wastewater flows and irrigation scheduling.

Screening would be provided before every pump station and traveling water screens were assumed for this purpose.

Irrigation. The cropping pattern and irrigation rates proposed for Alternative IIIB are selected to minimize capital costs. In accordance with widely accepted practice and Egyptian Law 93 (Ref. 3), the crops must be limited to forage, fiber, and seed crops if primary level effluent is used.

The crop pattern selected for conceptual design purposes consists of equal areas of water-tolerant grasses and alfalfa. Both crops use high amounts of water and nutrients and could be adapted to a pattern using no seasonal storage. They are well suited from a public health standpoint since the labor requirement is low, thus reducing the potential for public contact. In addition, the livestock that feed upon the forage provide an added link in the food chain between the wastewater and public consumption. The marketability of forage crops should be increasingly good in the future as a result of a concerted effort to increase meat production in Egypt (Ref. 36).

Special management will be required for crop rotation and irrigation scheduling, since no seasonal storage is provided. Both of the proposed crops can accept overirrigation in the winter. In the summer a portion of the area will have to be left unirrigated to allow for a full water supply for the remainder of the site. This can be scheduled to coincide with reestablishment of the perennial crops, which will be required every 3 to 5 years.

Table K-22 presents the water balance for Alternative IIIB in which the net irrigation requirement is shown relative to the actual wastewater supply and the amount of area in service. To achieve a balance between winter overirrigation and reduced irrigated area in the summer, we assumed that the annual wastewater supply was 20 percent greater than the annual net irrigation requirement. This

results in a net overirrigation of 31 percent for those areas that are in service year-round compared to the actual leaching requirement of 20 percent that was derived earlier. The maximum area that would have to be taken out of service in the summer is 27 percent. This is an acceptable amount from the standpoint of crop reestablishment.

With high application rates for forage crops, the area requirement for Alternative IIIB is less than that for IIIA. A total of 14,400 ha of irrigated area will be required for this alternative compared to 23,000 ha for the previous one.

The nitrogen and phosphorus application rates and normal uptake rates for grass and alfalfa are presented in Table K-23. The application rates are based on an anaerobic lagoon effluent with 26 mg/l of total nitrogen and 10 mg/l of phosphorus. The forage crops cannot utilize all of the nitrogen or phosphorus applied with the wastewater. However, further analysis shows that ground-water quality will not be adversely affected.

An analysis was performed to determine if the percolate nitrogen content would exceed the 10 mg/l limit for drinking water. The following equation was used to compute the percolate nitrogen content:

$$Cp = \frac{(Ln-U-D-S)}{0.1\ Pu}$$

Where Ln = nitrogen loading = 622 kg/ha/yr
U = crop nitrogen uptake = 500 kg/ha/yr
D = denitrification = 62 kg/ha/yr
S = nitrogen storage = 0 kg/ha/yr
Pu = percolation rate = 84 cm/yr

The nitrogen loading is the average for alfalfa and grass as shown on Table K-23. The crop nitrogen uptake rate was conservatively assumed to be 500 kg/ha/yr, even though rates up to 650 kg/ha have been noted. The denitrification rate was conservatively estimated at 10 percent of the total applied nitrogen, which should be typical for the sandy soils encountered at the site. The percolation rate was derived from the normal irrigation rate (2442 mm/yr) plus annual precipitation (100 mm/yr) minus the average crop consumptive use (1700 mm/yr).

The results of the analysis indicate that a percolate concentration of 7.1 mg/l total nitrogen can be expected, which is in an acceptable range.

The phosphorus loadings shown in Table K-23 will greatly exceed crop requirements. With the highly calcareous soils at the site, most of the excess should be adsorbed and precipitated in the soil profile. Percolate concentrations of 1 to 2 mg/l can be expected.

The proposed distribution system is similar to that of Alternative IIIA. It would consist of a network of concrete-lined canals leading to 12 distribution pump stations and then to center pivot sprinklers. The primary level effluent used in this alternative would make it necessary to provide screens ahead of pumping. It would also result in an increased operation and maintenance requirement because of greater problems with solids settling and clogging of nozzles. The downward spraying, low-pressure nozzles on the center pivots minimize these problems and, in addition, minimize the production of aerosols to reduce workers' contact with the wastewater.

Drainage. With increased rates of wastewater application, drainage requirements may be more extensive in Alternative IIIB than in IIIA. For conceptual design and cost estimating purposes, a system similar to IIIA was assumed but with tile drainage provided for 30 percent of the total area rather than 20 percent.

Management and Operation. Irrigation with primary level effluent, with no seasonal storage will require much greater control than with the previous alternative. Consequently, fewer options are available to A/GOSD for management and operation of the system. The two major possibilities are either that A/GOSD manage and operate the entire system, or that they operate the wastewater system, including the irrigation distribution, but contract the farming operation to a private or public company. Again, the first possibility was assumed for purposes of financial comparisons.

The operation of this system would be simpler than with Alternative IIIA and would require a smaller staff. This reflects the fact that there is no rapid infiltration system, the irrigated area is less, and the cropping pattern is much simpler. The proposed organization is the same and would consist of a general administration office; treatment, irrigation and support services department; and a farming coordination office.

An independent farming company would not have as much incentive with this alternative because of the strict limitation on crop selection and irrigation scheduling. Nevertheless, we believe that the system would run most efficiently and at the least overall cost if the agricultural operation were contracted to an entity that stood to gain by optimizing crop returns.

Infrastructure. The required infrastructure for Alternative IIIB would be considerably less than that required for Alternative IIIA. The combination of less irrigated area and the production of forage crops instead of a variety of high income crops would reduce the farm labor requirement. In addition, the type of required support industries would be much less diverse.

Other Options

Although Alternatives IIIA and IIIB were considered the most feasible alternatives for large-scale land application, other possibilities cannot be completely ruled out at this point. These other design options may have some merit if used in combination with the major alternatives for a portion of the flow or as backup measures in the event that either of the major alternatives cannot be fully implemented. Some of the options have already been introduced in this appendix:

- The use of irrigation techniques other than center pivots such as drip irrigation of high value crops or surface irrigation
- The use of the West Site
- The incorporation of several large, existing irrigated areas into the wastewater management program

At this point, none of these options offers any significant advantage over Alternatives IIIA and IIIB for handling the entire wastewater flow from Alexandria; however, they could be investigated in greater detail if the need arises.

EVALUATION OF LAND APPLICATION SYSTEMS

Both alternatives, IIIA and IIIB, would provide significant benefits if implemented, but at a considerable cost. In order that the advantages and disadvantages of these alternative can be effectively compared to those of the other major alternatives for Alexandria, an evaluation of the two systems is provided in this section. The principal factors that are considered are cost, environmental impact, implementability, and reliability.

Costs

The total costs of the land application systems are a combination of the capital cost required for construction plus the annual cost of operation and maintenance. The annual costs will be significantly offset in both cases by revenues from the agricultural products associated with the systems.

The preliminary estimates of the capital and annual costs which follow are based on the unit costs and procedures presented in Appendix B. Prices

for the end of 1980 were assumed. The estimates are for fully constructed systems capable of handling the design flow for the year 2000. A more detailed estimate that considers the effects of construction staging and other factors is presented as part of the present worth analysis in Chapter 5.

Capital Cost. A preliminary estimate of the capital costs for Alternatives IIIA and IIIB is presented in Table K-24. This estimate was developed from preliminary layouts of the two systems described in the previous section and is broken down by the major components of the systems. In both cases, the largest cost component is irrigation. The designs of the two irrigation systems are similar, and the difference in costs is primarily attributable to the difference in amount of area irrigated. Alternative IIIA entails the irrigation of approximately 23,000 ha of high income crop while only 14,400 ha of forage crops are irrigated in Alternative IIIB. The costs for irrigation include land preparation, distribution canals and pipelines, center pivot irrigation rigs, pump stations, electrical power, roads, and windbreak trees.

The pretreatment component comprises anaerobic lagoons for both alternatives plus rapid infiltration for Alternative IIIA. The cost for anaerobic lagoons in Alternative IIIB is considerably higher because of the increased detention time and emergency storage capacity provided. The costs included in pretreatment are basin excavation, distribution canals and piping, a small pump station for drainage, roads, and windbreak trees. The same cost items are encountered in larger quantities in the rapid infiltration system; in addition, costs are included for ground-water recovery wells and electrical power distribution.

The costs for drainage include ditch excavation and tile drains. Tile drainage is provided for a larger percentage of the irrigated area in Alternative IIIB, but the total cost is slightly less than that of Alternative IIIA.

The administrative and general costs are similar for both alternatives and include buildings, a potable water system and other utilities, monitoring wells, vehicles, and miscellaneous equipment.

The total capital costs including contingencies and fees for engineering, legal, and administrative services are approximately LE 184 million for Alternative IIIA and LE 121 million for Alternative IIIB. When viewed on a per area basis, these costs are much closer together at LE 8000 and 8400 per irrigated ha, respectively.

In addition to the costs for the land application system itself, it is also useful to consider the cost of the associated infrastructure.

This includes the new communities, transportation, and processing facilities that will need to be developed as a consequence of the project. These costs are not directly attributable to wastewater management; therefore, they should not be included in direct cost comparisons. Nevertheless, they can be used for overall budgetary planning purposes.

Based on estimates developed for the West Noubaria Extension, Reclamation, and Settlement Project (Ref. 37), the cost for community development, social services, utilities, and communications systems would be about LE 500 per feddan of total project area. This figure is probably conservative for the land application systems being considered, as the labor requirement per unit area is far less than that anticipated for the West Noubaria Project.

The cost for developing agro-industries would vary greatly depending on the types of crops grown on the site. For the West Noubaria Project, the cost was estimated to be about LE 2000 per feddan (at 1980 cost levels). The major part of this cost was for a sugar beet processing plant but it also included costs for dairy products; sunflower oil; and vegetable grading, curing, and canning industries.

Annual Costs. Estimates for annual costs are presented in Tables K-25 and K-26 for Alternatives IIIA and IIIB, respectively. The costs are for the operation of the land application system including irrigation distribution but do not include the cost of the farming. The costs for farming were taken into account in the development of net farm income. The annual costs are broken down into labor, power, fuel, and materials for the years 1980, 1990, and 2000. For both alternatives, materials comprise the most significant cost items, with power being the second largest item for Alternative IIIA and labor being the second largest item for Alternative IIIB.

Materials comprise replacement parts for repairs and normal maintenance. The annual cost is estimated as a percentage of the capital cost for each major item. This percentage varies depending upon the item, from as high as 4 percent for some mechanical equipment to as low as 0.2 percent for pipelines.

The labor costs are derived from preliminary staffing estimates which were based on the management and operation plans described previously. Total staff requirements of 247 and 186 people are estimated for Alternatives IIIA and IIIB, respectively, for the year 2000.

Energy costs have been estimated using a rate schedule obtained from the Alexandria Company for Electrical Distribution. The schedule is for services at medium voltage with connected loads of more than 500 kW. The

major difference in power requirements is the additional power required for ground-water recovery in Alternative IIIA. The other major area of power consumption for both alternatives is that of distribution pumping for irrigation.

The costs for fuel are for vehicles and other powered equipment.

As in the case of capital costs, the annual costs for both alternatives are quite similar when compared on a per unit area basis. The annual cost for Alternative IIIA is approximately LE 164 per irrigated ha and is approximately LE 168 for Alternative IIIB.

Agricultural Returns

An accurate estimate of the economic returns that can be expected from crop production requires detailed analyses that are beyond the scope of the present study. It is necessary to first make final crop selections and estimate detailed production costs, yields, and future commodity prices. These analyses are complicated by the fact that many agricultural prices are controlled and many items subsidized. It is possible, however, to make a preliminary estimate of returns based on generalized conditions and existing data.

Available sources of information included consultations with Dr. M.A. Khalil of the University of Alexandria, Department of Agricultural Economics; published information from the office of the Agricultural Attache of the U.S. Embassy in Cairo (Refs. 38-44); reports on the West Noubaria Project (Ref. 45); the Ministry of Agriculture (Ref. 53); the Ministry of Reclamation (Ref 54); and various other references (Refs. 46-48 and 52).

Based on this information, a conservative estimate was made of net income per feddan, which is presented in Table K-27. The net income was computed from the estimated crop yields, farm gate prices, gross income, and production costs, which are also included in Table K-27. The high income crop pattern is typical of what could be used for Alternative IIIA, whereas the forage crop pattern is typical of Alternative IIIB. Net income was computed as the sale value of the crop minus the production costs, which included farm labor, machinery ownership and operation, seeds, pesticides, sludge or manure applications, and farm management. The costs do not include the nitrogen and phosphorus fertilizers, as those will be mostly supplied by the wastewater.

The data presented in Table K-27 is based on averages from several references. The available information varies greatly because of the differences in agricultural regions and reporting formats and in

The net results of these effects would be somewhat greater with Alternative IIIA because of the higher value crops that would be grown and the larger area involved.

The major adverse impact would be that of potential localized odor problems. The primary source of these odors for either alternative would be the anaerobic lagoons, which should consequently be situated in locations isolated from any new population developments. There is some potential for occasional mild odors in the rapid infiltration system of Alternative IIIA if parts of the system are overloaded or the basin surfaces are not maintained. Following rapid infiltration, however, no noticeable odors whatsoever should be present in the irrigation system. In Alternative IIIB, occasional mild odors can be expected to occur in the irrigation system as a result of using primary effluent.

With both alternatives, the potential adverse impacts associated with site development on cultural and historic resources is very minor and can be mitigated. A collaborative literature search and field survey with local experts should locate known sites, ascertain archeological significance, and develop recommendations for preservation and/or excavation.

Public Health. With proper maintenance, land application systems provide a large measure of safety for public health. In general, they offer a greater amount of protection against parasites and viruses, trace metals, nitrate, and trace organics than conventional treatment systems (Ref. 51).

The risks of direct contact with waterborne disease organisms is negligible in the case of Alternative IIIA. The potential for contact in the operation of the rapid infiltration system is extremely small since it would operate by gravity and could be easily controlled with a small staff. The recovered water from the rapid infiltration system would be almost completely treated and would pose very negligible risk, even for unrestricted irrigation. The potential risks would be somewhat greater using primary effluent for irrigation in Alternative IIIB, but these also would be minimal if proper controls were utilized. One key safeguard would be the provision of high quality potable water onsite so that workers are not tempted to use wastewater.

One of the greatest health concerns with irrigation systems in Egypt is the propagation of the vectors and intermediate hosts responsible for schistosomiasis and malaria. Irrigation systems can transform low-intensity malaria and schistosomiasis problems of arid lands from a seasonal problem to a permanent hyperendemic situation (Refs. 49 and 50). Poor planning, maintenance, and operation have generally been the primary cause rather than irrigation per se. These debilitating

diseases can reduce worker productivity and detract from many of the anticipated social and economic benefits.

Under worst-case conditions, the incidence of schistosomiasis at perennial irrigation sites may increase to 80 or 90 percent of the population from a moderate rate of 10 to 15 percent at nonirrigated arid areas (Ref. 49). Disease transmission requires the presence of the intermediate host (an aquatic snail--Biomphalaria boissyi or Bulinus truncatus) and Schistosoma-infected workers (Ref. 50).

Control of the disease requires that the source of either of these be eliminated. With both of the land application alternatives, control of aquatic snails is made considerably easier by the fact that the irrigation system will be hydrologically isolated from the existing Nile River canal systems. These canal systems would otherwise be a continuous source of the host organisms. The only parts of the land application systems that could potentially support aquatic snails are the main distribution canals. These could, however, be kept free of the snails if adequately protected and if sufficiently high velocities are consistently maintained. Controlling access to the canals could also reduce the possibility of contamination by humans and consequently preclude the introduction of Schistosoma organisms.

Malaria problems can result at perennial irrigation sites due to the increased density of vector mosquitoes (Anopheles sp.) and the presence of the disease parasite (Plasmodium) in workers (Ref. 50). The most effective method of control is to prevent any areas of standing water, which the mosquitoes require for breeding. Conditions that commonly foster mosquito breeding and which should consequently be controlled include:

- Pools or unwanted channels
- Borrow pits for canal embankments that collect water
- Excessive water supply to poorly drained areas
- Improperly maintained canals

Control of mosquito propagation is made somewhat easier at the recommended site by the fact that soils are mostly very well drained. It is not likely that malaria would become a problem in the outlying areas around Alexandria since the disease does not appear to be endemic at present. In any case, the mitigation measures for mosquito control should be instituted to avoid general nuisance conditions.

An additional area of potential concern for Alternative IIIB is the possibility of soil contamination by the eggs of Ascaris and other

year-to-year cost differences. Data from the Ministry of Reclamation, which most closely represents the proposed project (i.e., reclaimed desert land as opposed to Delta lands), were most often used as the basis of the analysis. Professional judgement and experience were used to develop estimates where data were lacking. The net income values are based on 1980 farm gate prices and do not account for effects of government price controls and subsidies.

As with other reclamation projects, crop production--and hence net income--is very low during the first 2 to 5 years of operation. This is due to the need to leach salts from the root zone and to incorporate organic matter into the soil. This was taken into account in the financial analyses in Chapter 5. The sludge produced from the anaerobic lagoons can help supply the much-needed organic matter, but no sludge will be available until about the third year of operation. The applied wastewater will contain nutrients and some organics, especially in Alternative IIIB. This will help the site reach full production sooner than if Nile water were used.

Environmental Impacts

The impacts of a large-scale land application and agricultural project on rural desert environments are complex and varied. Some of the potential impacts, both beneficial and adverse, are presented here for both land application alternatives. The impacts associated with the conveyance pipeline are identical for the two alternatives and are described in Appendix M.

A detailed analysis of projected impacts is beyond the scope of this appendix; however, the major impacts anticipated and the means to mitigate the potentially adverse ones are outlined. Those areas that cannot be fully evaluated at this time and which require additional data collection and analysis are noted.

The major beneficial effects associated with both land application alternatives include:

- Reuse of wastewater
- Increases in food and fiber production
- Extension of irrigated agriculture to previously nonproductive lands
- Eventual development of new communities consistent with federal policy of population decentralization

- Provision of employment

Changes in baseline environmental conditions (public health, ground-water quality, etc.) are anticipated as a consequence to the development. However, most of these changes are similar to those experienced with conventional agricultural reclamation projects in arid lands. Public health is a natural concern with any wastewater system, but safeguards can be developed and implemented to minimize onsite pathogen exposure levels, eliminate inputs of toxic wastes into the collection system, reduce pathogen concentrations prior to crop use, and control disease vector and intermediate host propagation at the irrigation site.

The most significant environmental changes will be those associated with the ground water. These will result mostly from irrigation since the rapid infiltration system would have very little impact outside of a confined area (see Figure K-15). Approximately 25 to 35 percent of the applied irrigation water could percolate past the root zone to the ground water. All of the salts from the wastewater would be carried in this stream, so concentrations of total dissolved solids could become as high as 3,000 to 6,000 mg/l. A portion of the increased ground-water levels could be intercepted by a drainage system and discharged to the Noubaria Canal (assuming that its capacity can be enlarged to accept these flows). For the most part, however, the higher ground-water levels will result in increased flows in the same direction as present towards Wadi Natroun. Existing ground water in this area is intercepted by saline lakes and evaporated. A detailed hydrogeologic study will be required to determine the extent of the potential changes that will take place and their impacts.

The potential impacts of the two land application alternatives are summarized in Tables K-28 through K-30 with respect to social and economic, public health, and environmental considerations. Alternative IIIA would have a greater net positive effect than Alternative IIIB, but either alternative, if properly executed, should be acceptable from an environmental standpoint. Significant environmental issues presented in the tables are discussed in the following three subsections.

Social And Economic. Potentially beneficial effects that are anticipated from either land application system include:

- Reclamation of new lands made possible by reuse of wastewater
- Development of decentralized rural communities
- New employment

parasites. The adverse impact of this can be mitigated by requiring that workers wear proper footwear.

Natural Environment. Few significant environmental impacts are anticipated with either alternative and those that do occur will be similar to ones encountered with conventional reclamation projects. The greatest impact of the project will be on the ground-water quantity and quality. As discussed, the aquifer below the site moves generally south towards Wadi Natroun. A large-scale irrigation project will certainly increase this flow as other nearby reclamation projects already have. The quality of the irrigation percolate should be extremely good with respect to all constituents of possible concern, except that the total dissolved solids concentrations will be significantly increased. The overall effect of this on future downgradient uses of the ground water will need to be addressed in detail as part of the geohydrologic investigations.

Implementability

With such large land application systems being considered, the implementability of the two alternatives is naturally a very important consideration. The major factors that will affect this include:

- Availability of the necessary resources
- Constructibility of the planned facilities
- Acceptability of the concept of wastewater irrigation

Availability of Resources. Significant quantities of land, energy, construction materials, and labor would be necessary for the development and operation of the project. These must be made available if the project is to be implemented.

The most critical resource required for the project is the land. The East Site, which is the recommended site, is apparently still uncommitted for development and would have to be made available by the Egyptian Government. The West Site could be considered as a backup site, but it is less desirable and would decrease the feasibility of the project.

Other important required resources are generally available but would need to be committed to the project. Approximately 72 to 127 million kWh of electricity per year would be required for pumping the wastewater on the site. This significant energy requirement could affect the planning of power facilities. Most of the required

construction materials are conventional and could be supplied from a combination of Egyptian and United States sources. Some items such as certain sizes of pipe are required in sufficiently large quantities to justify the development of new fabrication facilities in the local area. Operational manpower should be readily available as very few special skills will be required.

Constructibility. Construction of the land application facilities should present no major obstacles for implementation. Most of the construction, earthwork, well drilling, pipeline installation, and center pivot assembly require no special technology or equipment that is not commonly available and used in Egypt. The only condition that may impede construction activities are occasional wind or dust storms.

Acceptability of the Concept. The concept of wastewater reuse has been the subject of considerable discussion and debate in Egypt in recent years. In general, it has strong support from most of the principal agencies and individuals concerned with the project, including authorities of the Office of the Governorate of Alexandria. There are, however, a number of concerns held by various others that will need to be addressed prior to implementation. Further study of the project's effects in greater detail should answer most of the questions that may be raised. It is equally important, though, that there also be strong government backing and coordination at a high level to ensure that any areas of potential conflict are resolved.

Reliability

The land application alternatives developed in this appendix would both provide a high degree of reliability relative to other types of wastewater systems if properly operated. Generally, Alternative IIIA would provide a more consistently reliable operation than Alternative IIIB because of the extra high degree of treatment and storage provided by the rapid infiltration system. In the discussion that follows, the ability of both alternatives to handle the most common areas of potential difficulties are considered, including:

- Failure of mechanical equipment
- Failure of treatment mechanisms
- Disruption of power supply

Mechanical Equipment. The mechanical components associated with Alternative IIIA consist primarily of pumps and motors for wells and distribution pumping stations as well as center pivot sprinkling equip-

ment. For Alternative IIIB, the mechanical components would include the same sprinkler equipment and pump stations for a similar variety of pump stations, as well as screens prior to pumping. In both cases, all of the components are conventional and could be readily maintained and repaired.

The effect of a failure of an individual unit on the overall operation of the system would normally be very small because in each case a large number of individual units could be isolated, taken out of service, and repaired.

The systems would also be designed to handle potential failures in pipelines and canals. The distribution networks are extensive enough that sections could be taken out of service for repair, keeping the remaining parts of the network in service.

Storage provides an additional safety factor for both alternatives. In Alternative IIIA, a potentially very large storage capacity would be provided by rapid infiltration. Five days of storage capacity for emergencies is provided in the anaerobic lagoons in Alternative IIIB.

Treatment Mechanisms. The primary treatment mechanism in anaerobic lagoons is the settling of suspended solids with subsequent digestion of the sludge layer on the bottom of the lagoon. This treatment is normally extremely reliable and will continue under a wide range of conditions. Upsets of the system can occur as a result of large toxic spills in the sewer system or other causes. However, these will generally affect only the second stage of the fermentation process causing severe odor problems but will have little affect on treatment performance.

The complex physical, chemical, and biological mechanisms of the rapid infiltration system are also very reliable under most conditions. However, the system does require an adequate zone of unsaturated soil for proper treatment to take place. A good monitoring program is required, particularly during the peak storage season, to ensure that the ground-water mound does not rise too close to the infiltration basins at any point in the system. Operation of the recovery wells can be adjusted to control high spots and potential problem areas.

Power Supply. Operation of the entire irrigation distribution system would be affected by disruptions in electrical power supply. However, both the anaerobic lagoons and the rapid infiltration system are operated by gravity; consequently, storage capabilities would not be affected.

Power outages could cause problems for crop production, especially during the hottest period of the year. If an outage lasted for several days, some crops would be stressed and possibly damaged by the lack of water.

CONCLUSIONS AND RECOMMENDATIONS

As a result of this preliminary investigation of land application, we concluded the following:

- Irrigation systems using primary treated effluent are feasible with proper selection of crops and control of public access to the site.
- Discharge of treated wastewater to canals used for unrestricted irrigation is not acceptable in most instances.
- Using wastewater with a moderately high salt concentration, such as that from Alexandria, for irrigation is feasible with appropriate crops and leaching.
- Storage of wastewater to account for fluctuations in annual irrigation requirements is advantageous in many cases; however, other management techniques exist that can be used instead of storage.
- Irrigation, rapid infiltration, and overland flow methods of land application are all technically feasible given the proper site conditions and system requirements.
- Irrigation combines beneficial reuse of wastewater with treatment. It can be designed in a wide variety of ways to suit many different site conditions.
- Rapid infiltration is a treatment process which will achieve an effluent quality equivalent to that of conventional tertiary treatment. Renovated water may be disposed of in the ground water or may be recovered for subsequent reuse.
- Overland flow is a treatment process which produces a high quality effluent. Renovated water collected from the bottom of the slopes can be discharged or reused.
- The areas close to Alexandria are mostly unsuitable for large-scale land application systems because of conflicts with existing land use or because of poor or marginal site conditions.
- Land application on a smaller scale using wastewater flows from local communities is potentially feasible in many areas.
- The most feasible locations for large-scale land application of Alexandria wastewater are the East and West Sites located on

either side of the Cairo-Alexandria Desert Road, south of the El Nasser Canal.

- Both the East and West Sites are mostly uncommitted for reclamation and development at this time. Physical characteristics over portions of each site are favorable for both irrigation and rapid infiltration.

- Of the two potential sites, the East Site is preferred.

- There are two major possibil for land application at the East Site. Alternative IIIA nes rapid infiltration with irrigation of high income crops. Alternative IIIB consists of irrigation of forage crops with primary effluent.

- The overland flow process is infeasible, except possibly for small local sewerage systems, due mainly to a lack of large suitable sites.

- Alternative IIIA would entail pretreatment by anaerobic lagoons, rapid infiltration for high level treatment and storage, recovery of the ground water, and irrigation of 23,000 ha of high income crops using center pivot sprinklers. The total land area required would be 31,000 ha (or 73,000 feddans).

- Alternative IIIB would entail pretreatment by anaerobic lagoons with emergency storage capacity provided, screening, and irrigation of 14,400 ha of forage crops using center pivot sprinklers. With no seasonal storage provided, crops would be overirrigated in the winter and some areas would be taken out of production in the summer. The total land area required would be 18,500 ha (or 44,000 feddans).

- Other design options could be considered in the future in combination with the above alternatives or as backup measures in the event that either IIIA or IIIB cannot be fully implemented. These other options will not affect the overall evaluation of major wastewater alternatives available to Alexandria.

- Alternative IIIA would have a higher capital and annual operation and maintenance cost than Alternative IIIB. However, on the basis of cost per area irrigated, Alternative IIIA has a slight cost advantage.

- Annual economic returns from crop production cannot be accurately estimated without final crop selections and detailed estimates of

production costs, yields, and future commodity prices. Preliminary estimates show that returns per unit of area from the high income crops of Alternative IIIA may be about two times greater than those from the forage crops of Alternative IIIB.

- On the basis of social and economic, public health, and environmental considerations, Alternative IIIA would have a greater net positive effect than Alternative IIIB. Both alternatives, if properly executed, should be acceptable from an environmental standpoint.

- The social and environmental impacts of both alternatives are generally positive and include reclamation of new lands, development of decentralized rural communities, and new employment. The most significant adverse impact would be potential odors from the anaerobic lagoons. This problem could be mitigated by proper siting and isolation of the lagoons from populated areas.

- With proper maintenance both alternatives would provide a large measure of safety for public health. Alternative IIIA would have an advantage in this regard because of the high level of treatment obtained in the rapid infiltration system. Positive measures of control would be required in both cases to prevent the propagation of schistosomiasis and malaria.

- The only significant environmental effect in both cases would be on the ground water. Ground-water flows would be increased and the quality would be affected by the high salt concentration of percolate from the irrigation systems. A detailed geotechnical investigation would be required to determine the overall effects on future down-gradient uses.

- The most critical factor affecting the implementability of land application is the availability of the required land area.

- Both land application alternatives would have a high degree of reliability and should be able to handle most difficulties arising from the failure of mechanical equipment or treatment mechanisms and from brief electrical power disruptions.

In light of these conclusions, both Alternatives IIIA and IIIB should be considered as major wastewater alternatives for Alexandria. Should either of these compare favorably with the other major alternatives being considered, initiation of a more detailed site investigation will be required as quickly as possible. This would include the production of an accurate base map, a detailed soil survey, and a

geohydrological study. At the same time it will be necessary to further study the agricultural economics of the proposed alternatives.

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Table K-1
WASTEWATER QUALITY REQUIREMENTS FOR DISCHARGE AND CROP IRRIGATION

Constituent	For Discharge to the Nile and its Branches[a]	For Discharge to Drains		For Crop Irrigation				
				Public STP			Industrial	
	Industrial[b]	Industrial[b]	Sewage[b]	Sandy Soil	Clay Soil	Private STP	Sandy Soil	Clay Soil
BOD, ppm	20	60	40	—	80	40	—	80
COD, ppm	15	40	30	—	50	30	—	50
Settleable Matter, ppm (unless otherwise indicated)	30	80	—	3 cm^3/l[c]	—	—	3 cm^3/l[c]	—
pH	6-9	6-9	—	—	6-9	—	—	—
Sulfides as Sulfate, ppm	1	1	—	5	0.1	—	5	0.1
Cyanides, ppm	0.1	0.1	—	—	0.1	—	—	0.1
Oil and Fats, ppm	10	10	—	20	5	—	20	5
Temperature, °C	35	35	—	—	—	—	—	—
Poisons to Fish or for Drinking Water	N.A.	—	—	—	—	—	—	—
Phenols, ppm	—	0.1	—	—	—	—	—	—
Chlorine, ppm	—	1	—	—	—	—	—	—
Cr + Ag + Cu + Hg + As + Cd + Ba + Se + Pb + Ni, ppm	—	1	—	—	—	—	—	—
TDS, ppm	—	5,000	—	—	2,000	—	—	2,000
Color, cm clarity	—	10	—	—	—	—	—	—
Pesticides	—	N.A.	—	—	—	—	—	—
Suspended Matter, ppm	—	—	50	—	80	50	—	80
Chlorination Required	—	—	—[f]	—	—	—	—	—

Source: Reference 3

[a]No discharge of sewage allowed to the Nile or its branches

[b]Where industrial and municipal sewage are combined, standard for sewage for settleable matter, BOD, and COD apply in addition to the industrial limits.

[c]By volume after 5 minutes

[d]Up to 10 ppm if 3 km from populated areas

[e]N.A. = none allowed

[f]Minimum chlorine residual after 20 minutes = 0.5 ppm

Table K-2
DESIGN FLOWS FOR LAND APPLICATION ALTERNATIVES
(Ml/d)

Year	Average Annual Design Flows
1980	450
1990	730
2000	945

Note: Developed in Appendix C

Table K-3
DESIGN WASTEWATER QUALITY

Constituent	Concentration
BOD	560 mg/l
Suspended Solids	500 mg/l
Total Nitrogen	33 mg/l
Phosphorus	10 mg/l
TDS	1,060 - 1,600 mg/l
Electrical Conductivity	1.7 - 2.5 mmhos/cm

Source: Reference 1

Table K-4

COMPARISON OF DESIGN FEATURES FOR LAND APPLICATION PROCESSES

Feature	Irrigation	Rapid Infiltation	Overland Flow
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface
Annual application rate, m	1 to 3	6 to 170	3 to 20
Area required, feddans[b]	30 to 90	0.5 to 15	4.5 to 30
Typical weekly application rate, cm	2 to 12[c]	12 to 325	6 to 40
Minimum preapplication treatment provided in the United States	Primary sedimentation[c]	Primary sedimentation	Screening and grit removal
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required

[a]Includes ridge-and-furrow and border strip

[b]Field area in feddans not including buffer area, roads, or ditches per 1 Ml/day flow

[c]Depends on the use of the effluent and the type of crop

Table K-5
CONSUMPTIVE USE FOR SELECTED POTENTIAL CROPS
(mm per month)

Month	Alfalfa	Grass	Wheat	Corn	Berseem	Composite for Vegetables	Effective Precipitation
January	37	78	70	0	72	51	18
February	49	90	91	0	83	59	12
March	104	133	140	0	120	87	4
April	157	154	116	0	97	100	0
May	199	189	51	38	42	123	0
June	232	202	0	113	0	131	0
July	253	220	0	198	0	143	0
August	225	205	0	193	0	133	0
September	175	175	0	73	17	113	0
October	112	131	8	0	39	86	0
November	71	84	23	0	61	55	14
December	48	68	39	0	58	44	21
TOTAL	1,662	1,729	530	615	589	1,125	69

Source: Reference 1

Table K-6
LEACHING REQUIREMENTS

Crop	Yield Reduction (%)	EC_{dw} (mmhos/cm)	LR (%)
Forages			
Alfalfa	10	28	7
Salt-tolerant grasses	0	36	6
Berseem	<10	21	10
Grains/Field Crops			
Wheat	0	40	5
Barley	0	44	5
Corn	0	18	12
Sugar beets	0	42	5
Cotton	0	42	5
Soybeans	0	26	8
Vegetables/Produce Crops[a]			
Beets	0	32	7
Spinach	0	20	10
Melons	10	20	10
Tomatoes	<10	22	9
Broccoli	<10	26	8
Cabbage	10-15	26	8
Potatoes	10-15	20	10
Fruit Crops[a]			
Date palms	0	48	4
Figs, olives, pomegranates	0-5	28	8
Average	--	--	6[b]

Note: Source for yield reductions for irrigation water with EC of 2.1 mmhos/cm and for EC_{dw} values was Ayres (Ref.5).

[a]Other vegetables and fruit crops (i.e., citrus) that would experience 25 percent or greater yield reduction with the expected water quality were not considered.

[b]Note: 20 percent LR used for design. See discussion in text.

Table K-7
TYPICAL TREATMENT EFFECTIVENESS FOR RAPID INFILTRATION

Constituent	Percolate Concentration (mg/l) Average	Maximum
BOD	2	<5
Suspended Solids	2	<5
Ammonia Nitrogen as N	0.5	<2
Total Nitrogen as N	10	<20
Total Phosphorus as P	1	<5

Table K-8
MAPPING UNIT DESCRIPTIONS, EAST SITE

Mapping Unit	Area (ha)	Field Description[a]
Db	10,200	Loose sandy soils of undulating-rolling, medium and high dunes, partly barkhan or longitudinal arrangement.
Db 11	3,300	Like Db. Coarse sandy loam to clay loam soils or subsoils between isolated high dunes and low dunes. Valleys make up 50 percent of mapping unit area.
Ds	13,400	Loose sandy soils of medium and low dunes.
Ds 11	5,700	Like Ds. Coarse sandy loam to clay loam soils or subsoils between medium and low dunes. Valleys make up 30 to 40 percent of mapping unit areas.
Ds 12	9,700	Like Ds. Loamy sandy soils or subsoils, partly with rocky crusts, between low and some higher dunes. Valleys make up 40 to 50 percent of mapping unit area.
Ds 18	2,400	Like Ds. Low dunes but with $CaCO_3$ concentrations. Partly with loamy sand-sandy loam subsoils.
Ds 25	16,800	Like Ds. Predominantly medium to high dunes.
Du 18	4,000	Undifferentiated sandy soils with scattered low sand sheet and longitudinal dunes. Below the depth of wind disturbance, the sand is moderately cemented with $CaCO_3$. Locally compact loamy subsoils may occur.

Source: Reference 14

[a]The presence of $CaCO_3$ can make field descriptions appear texturally finer than lab analyses.

Table K-9
SELECTED CHEMICAL AND PHYSICAL ANALYSES,
EAST SITE

Associations[a]	Saturation Percentage[b]	pH	Saturated Extract Conductivity[b] (mmhos/cm)	Sodium Adsorption Ratio[b]	Textural Analysis
Db	18-66	6.9-8.4	5.7-125	21.7-92.5	Sandy loam to medium coarse sand
Ds	17-73	6.8-8.4	0.5-17.2	3.5-42.8	Sandy clay loam to coarse sand
Du	17-60	7.2-7.9	1.2-18	4.0-31.6	Clay to medium coarse sand

Source: Reference 14
Note: Range for entire soil profile at select sampling locations.

[a]Db includes Db and Db 11 mapping unit.
Ds includes Ds, Ds 11, Ds 12, Ds 18, and Ds 25 mapping units.
Du includes Du 18 mapping units.

[b]Field observations and subsequent chemical analyses suggest that published values of saturation percentage, saturated extract conductivity, and sodium ratio are overestimated. Lower range is probably more representative of site conditions. Lowest values tend to be associated with better drained locations.

Table K-10

SUITABILITY LIMITATIONS FOR RAPID INFILTRATION, EAST SITE

Parameter	Suitability	Limitations
Topography	Slopes are generally not excessive; significant problems are not anticipated.	Rugged topographic locations should be avoided or levelled.
Soils		
Physical Properties	Db, Ds, and Ds 25 soils have the most favorable structural, textural, and infiltration properties.	The extent and origin of the lime concentration in Db 11, Ds 11, and Ds 12 soils need to be investigated; the effect on drainage needs investigation.
Chemical Properties	Coarse textures offset the effect of sodium on soil permeability; soil ph is favorable for denitrification-nitrification reactions, and retention of trace elements.	Salt transport is anticipated primarily in poorly drained locations where soil salinity is high. Salinity distribution throughout the geologic profile needs verification.
Geology	Available stratigraphic information suggests few drainage restrictions through depth to ground water.	The extent of clay stringer, limestone, and lime crusts needs to be verified.
Hydrogeology	Depth to ground water is generally sufficient for unsaturated flow and ground-water mound management.	Shallow ground-water areas (<5 m) should be avoided to minimize drainage problems; ground-water quality needs to be maintained for down-gradient needs.
Location/ Land Use	Majority of site is undeveloped and available.	Wastewater conveyance distance is long; facility layouts require design around El Nasser Canal, Nouberia Canal, Komombo, and power lines. Buffers are required around public accesses. Loss of Bedouin grazing lands.
Vegetation/ Wildlife	Area sparsely vegetated, site clearing minimal.	None.
Historical/ Cultural	Few sites of historical interest noted.	Tombs need to be accurately located and catalogued and design accommodations made.

Table K-11

SUITABILITY LIMITATIONS FOR WASTEWATER IRRIGATION, EAST SITE

Parameter	Suitability	Limitations
Topography	Slopes generally not excessive; runoff, erosion, and farming problems not anticipated.	Surface irrigation would require extensive land leveling; overhead irrigation would require minor smoothing; small rugged areas re-require much leveling or avoidance.
Soils		
Physical Properties	Deep, well-drained soils are available throughout the sites; overhead irrigation can best supply crop consumptive needs in high intake rate soils.	Cemented, clayey subsoils and/or shallow soils in valley locations may require artificial drainage or deep ripping.
Chemical Properties	Salinity-alkalinity problems are not anticipated in well-drained locations; soil pH is generally favorable for crop growth.	Salinity-alkalinity associated with poorly drained locations. Leaching and/or reclamation required in those areas.
Geology	Stratigraphic information suggests few drainage restrictions through depth to groundwater table.	Extent of clay stringers, limestone, and lime crusts needs to be verified.
Location/ Land Use	Majority of site is undeveloped and available.	Wastewater conveyance distance is long; facility layouts require design around El Nasser Canal, Nouberia Canal, and power lines. Buffers are required around public accesses; relocation of small resident population may be required.
Vegetation/ Wildlife	Area poorly vegetated; site clearing minimal.	None.
Historical/ Cultural	Few sites of historical interested noted.	Sites need to be accurately located and catalogued and design accommodations made.

Table K-12
MAPPING UNIT DESCRIPTIONS, WEST SITE

Mapping Unit	Area (ha)	Field Description[a]
DB 11	1,350	Coarse sandy loam to clay loam soils or subsoils between isolated high sandy dunes. Valleys occupy 50 percent of mapping unit area.
Ds 11	4,500	Coarse sandy loam to clay loam soils or subsoils between medium and low sandy dunes. Valleys occupy 30 to 40 percent of mapping unit areas.
Ds 12	2,500	Loamy sandy soils or subsoils, partly with rocky crust between low and some higher sandy dunes. Valleys occupy 40 to 50 percent of mapping unit area.
Du 18	11,600	Loose sandy soils with $CaCO_3$ cementation, partly with loamy sand-sandy loam subsoils.
Pm 4/20	5,400	Shallow silty clay loam soils over rock, or rock in surface.
Pl 20	2,300	Coarse sandy loam soils of varying depth with thin sheets of windblown sands.

Note: The presence of $CaCO_3$ can make field descriptions appear texturally finer than lab analyses.

Source: Reference 14

Table K-13
SELECTED CHEMICAL AND PHYSICAL ANALYSES,
WEST SITE

Associations[a]	Saturation Percentage[b]	pH	Saturated Extract Conductivity[b] (mmhos/cm)	Sodium Adsorption Ration[b]	Textural Analyses
Db 11	18-66	6.9-8.4	5.7-125	21.7-92.5	Sandy loam to medium coarse sand
Ds 11	17-73	6.8-8.4	0.5-17.2	3.5-42.8	Sandy clay loam to coarse sand
Ds 12	26	7.4	7.4	28.4	Medium fine loamy sand
Du 18	17-60	7.2-7.9	1.2-18	4.0-31.6	Clay to medium coarse sand
Pm 4/20	20-39	7.6-7.7	1.8-21.8	3.4-29.5	Clay loam to sandy loam
Pl 20	--[b]	--[b]	--[b]	--[b]	Medium coarse and very coarse sandy loam to loam

Source: Reference 14
Note: Range for entire profile at select sampling locations.

[a]Field observations suggest that published values of saturation percentage, saturated extract conductivity, and sodium adsorption rates are overestimated. Lower range is probably more representative of site conditions. Lowest values tend to be associated with best drained locations.

[b]Comparable data not available.

Table K-14
SUITABILITY LIMITATIONS FOR
RAPID INFILTRATION, WEST SITE

Parameter	Suitability	Limitations
Topography	Slopes are generally excessive; significant problems not anticipated.	Slopes in depression area can be significant.
Soils		
Physical Properties	Db 11, Ds 11, and Ds 18 have favorable textural and structural properties in non-valley locations.	Pm 4/20 soils are not suitable. Extent of lime cementation in Db 11, Ds 11, Ds 12, Ds 18, and Pl 20 soils needs to be investigated.
Chemical Properties	Coarse textured soil offset the effect of sodium on soil permeability; soil pH is favorable for denitrification-nitrification reactions and retention of trace elements.	Salt transport is anticipated primarily in poorly drained locations where soil salinity is high. Salinity distribution throughout the geologic profile needs verification.
Geology	Deep, well-drained subsurface strata are believed present in central and eastern locations, surficial bedrock areas in depression can be used as storage areas.	Impermeable to poorly permeable sandy limestone, limestone, sandstone, and gypsiferous clays have surficial distribution on the western portion of the site. Subsurface distribution needs to be investigated.
Hydrogeology	Depth to ground water in unconfined locations are generally sufficient to maintain significant unsaturated flow, which lends them to ground-water mound management.	Aquitards present in semi-confined locations may result in local ground water perching in central and western portions. Subsurface distribution needs to be addressed.
Location/ Land Use	The majority of site is undeveloped and available; minor population relocation required.	Conveyance distance is long; facility layouts require design around El Nasser Canal and irrigation projects. Buffers are required around public access routes. Loss of Bedouin grazing land.
Vegetation/ Wildlife	Area sparsely vegetated; site clearing minimal.	None.
Historical/ Cultural	Sites of historical interest noted in poorest soil areas.	Sites need to be accurately located and catalogued and accommodated during design.

Table K-15
SUITABILITY LIMITATIONS FOR WASTEWATER IRRIGATION,
WEST SITE

Parameter	Suitability	Limitations
Topography	Slopes are generally not excessive; in central and eastern portions of site, runoff, erosion, and farming problems not anticipated.	Slopes in depressions can be significant.
Soils		
Physical Properties	Db 11, Ds 11, and Ds 18 have favorable textural and structural properties in non-valley locations (see limitations).	Pm 4/20 soils not suitable. Lime cementation in Db 11, Ds 11, Ds 12, and Ds 18 and Pl 20 may pose drainage problems, particularly in valley locations. Extent of cementation needs to be established.
Chemical Properties	Soil pH is generally favorable for crop growth.	Salinity-alkalinity associated with poorly drained locations; leaching and/or reclamation may be required.
Geology	Deeper well-drained strata believed present in central and eastern locations.	Surficial bedrock locations in western portion of site are not suitable for cultivation.
Hydrogeology	Depth to ground water in unconfined locations (central and east) generally sufficient to maintain good drainage.	Aquitards present in semi-confined locations may result in local ground water perching and poor drainage.
Location/ Land Use	Majority of site is undeveloped and available; minor population displacement required.	Conveyance distance is long; facility layouts require design around El Nasser Canal and local irrigation projects. Buffers are required around public access. Loss of grazing land for Bedouins.
Vegetation/ Wildlife	Area sparsely vegetated; site clearing minimal.	Replacement of wildlife may occur.
Historical/ Cultural	Sites of historic interest noted in poorest soil areas.	Sites need to be accurately located and catalogued and accommodated in design.

Table K-16
DATA NEEDS AND DESIGN PHASE REQUIREMENTS

Parameter	Data Need	Design Phase Requirements
Topography	Better definition of site topography.	Topographic survey.
Soils	Better definition of soil physical properties for drainage and earthwork considerations.	Grain size analyses, Atterberg limits, bulk density, particle density, percent moisture, percent porosity, compaction, horizontal and vertical permeability, infiltration testing, more intensive soil survey, aerial photography.
	Better definition of soil chemical properties to establish optimum soil reclamation procedures and crop/nutrient requirements.	Standard agricultural analyses for salts, cations, pH, nutrients, organic matter, and trace elements.
Geology	Better definition of surface and subsurface distribution of bedrock and lime concretions.	Deep exploration borings, geophysical survey.
Hydrogeology	Better definition of depth to ground water, flow regime, hydraulic characteristics, and quality.	Installation of monitoring systems and test wells, and ground-water modeling study.
Historical/ Cultural	Accurate location of all historical sites.	Collaborative literature search and field survey with local experts.
Environmental/ Social	Definition of the impact on environment and inhabitants.	Environmental assessment.

Table K-17
CONCEPTUAL DESIGN DATA,
ALTERNATIVE IIIA

Parameter	Design Value
General	
Design flow, year 2000 (Ml/d)	945
Influent quality	
BOD (mg/l)	560
Suspended solids (mg/l)	500
Total nitrogen (mg/l)	33
Total phosphorus (mg/l)	10
Electrical conductivity (mmhos/cm)	2.1
Total dissolved solids (mg/l)	1,330
Location	East Site
Pretreatment	
Type	Anaerobic lagoons
Average detention time (days)	3
Total pond volume	
Days	8
Million cubic metres	7.6
Average depth (metres)	4.5
Gross area requirement (ha)	210
Sludge production	
Million kg/year	254
Percent solids	30
Effluent quality	
BOD (mg/l)	300
Suspended solids (mg/l)	200
Total nitrogen (mg/l)	26
Total phosphorus (mg/l)	10
Rapid Infiltration	
Loadings	
Hydraulic (m/year)	27
BOD (kg/ha/day)	225
Suspended solids (kg/ha/day)	150
Infiltration area (ha)	1,280
Gross area requirement (ha)	1,540
Infiltration basins	
Dimensions (metres)	100 x 400
Average depth (metres)	1.0
Number	320

Table K-17 (continued)
CONCEPTUAL DESIGN DATA,
ALTERNATIVE NO. IIIA

Parameter	Design Value
Rapid Infiltration (continued)	
Loading cycle	
Flooding (days)	1
Drying (days)	6
Method of distribution	Concrete-lined canals
Quality of percolate	
BOD (mg/l)	5
Suspended solids (mg/l)	2
Total nitrogen (mg/l)	10
Total phosphorus (mg/l)	2
Electrical conductivity (mmhos/cm)	2.3
Total dissolved solids (mg/l)	1,460
Ground Water Storage and Recovery	
Total required storage	
Percent of annual flow	19
Million cubic metres	65
Depth to ground water (metres)	
Existing	10-15
Low storage season	15
High storage season	3
Recovery wells	
Design capacity (l/s)	125
Diameters (metres)	0.65
Depth (metres)	43
Average operating depth (metres)	20
Number	223
Excess capacity (%)	50
Irrigation	
Crop pattern	High income
Average annual application (metres)	1.5
Leaching requirement (%)	20
Area requirements (ha)	
Irrigated area	23,000
Gross area	29,000

Table K-17 (continued)
CONCEPTUAL DESIGN DATA,
ALTERNATIVE NO. IIIA

Parameter	Design Value
Irrigation (continued)	
Distribution system	
Type	Center pivot
Area per unit (ha)	52
Number of units	443
Operating pressure at pivot (kg/cm^2)	3
Drainage	
Type	Subsurface tile with open main collector ditches
Average depth (metres)	3
Percent of irrigation area	20
Total area (ha)	4,600
Tile spacing (metres)	250
Tile depth (metres)	2
Total Land Area	
Hectares	31,000
Feddans	73,000

Table K-18
MONTHLY WASTEWATER IRRIGATION RATES
FOR THE HIGH INCOME CROP PATTERN

Month	Weighted Average Consumptive Use (mm)	Irrigation Rate (mm)
January	62	66
February	71	89
March	104	150
April	99	148
May	83	125
June	131	196
July	143	215
August	133	199
September	65	98
October	63	94
November	58	66
December	51	45
TOTAL	1,063	1,491

Table K-19
ANNUAL NUTRIENT UPTAKE RATES
AND APPLICATION FOR THE HIGH INCOME CROP PATTERN

Crop	Nitrogen (kg/ha)		Phosphorus (kg/ha)	
	Uptake Rates	Applied	Uptake Rates	Applied
Berseem	100-250[a]	80	15-20	10
Vegetables	50-250	160	10-30	20

Source: Reference 14

[a]Legumes will also take nitrogen from the atmosphere if not available in the soil.

Table K-20
WATER BALANCE AND STORAGE REQUIREMENTS
FOR THE HIGH INCOME CROP PATTERN

Month	Available Wastewater[a]	Wastewater Irrigation Rate[a]	Change in Storage[c]	Cumulative Storage[d]
January	7.9	4.4	+3.5	16.7
February	7.8	5.9	+1.9	18.6[e]
March	7.9	10.0	-2.1	16.5
April	7.9	10.0	-2.1	14.4
May	8.2	8.3	-0.1	14.3
June	8.5	13.1	-4.6	9.7
July	8.9	14.4	-5.5	4.2
August	9.2	13.4	-4.2	0
September	8.8	6.6	+2.2	2.2
October	8.4	6.4	+2.0	4.2
November	8.3	4.4	+3.9	8.1
December	8.2	3.1	+5.1	13.2
Annual	100	100	--	--

Note: Values given are as percentage of annual flow volume.

[a]Based on data presented in 1978 Master Plan (Reference 1)

[b]Based on Table K-18

[c]Change in storage = available wastewater less the wastewater irrigation rate

[d]Cumulative storage as of the last day of the month

[e]Required storage volume = 18.6% of total annual flow

Table K-21
CONCEPTUAL DESIGN DATA,
ALTERNATIVE IIIB

Parameter	Design Value
General	
Design flow, year 2000 (Ml/d)	945
Influent quality	
BOD (mg/l)	560
Suspended solids (mg/l)	500
Total nitrogen (mg/l)	33
Total phosphorus (mg/l)	10
Electrical conductivity (mmhos/cm)	2.1
Total dissolved solids (mg/l)	1,330
Location	East Site
Pretreatment	
Type	Anaerobic lagoons
Stages	2
Average detention time (days)	8.75
Additional storage capacity (days)	5
Average depth (metres)	
Low storage	3.5
High storage	5.5
First stage	
Average detention time (days)	4
Redundancy for sludge handling (%)	100
Pond volume (million m^3)	15.8
Second stage	
Average detention time (days)	4.75
Redundancy for sludge handling (%)	29
Pond volume (million m^3)	10.9
Gross area requirement (ha)	520
Sludge production	
Million kg/year	340
Percent solids	30
Screening	Traveling water screens

Table K-21 (continued)
CONCEPTUAL DESIGN DATA,
ALTERNATIVE NO. IIIB

Parameter	Design Value
Pretreatment (continued)	
Effluent quality	
BOD (mg/l)	225
Suspended solids (mg/l)	100
Total nitrogen (mg/l)	25
Total phosphorus (mg/l)	10
Electrical conductivity (mmhos/cm)	2.2
Total dissolved solids (mg/l)	1,390
Irrigation	
Crop pattern	Forage crops
Average annual application (metres)	2.4
Leaching requirement (%)	20
Area requirements (ha)	
Summer irrigated area	10,500
Winter irrigated area	14,400
Gross area	18,000
Distribution system	
Type	Center pivot
Area per unit (ha)	52
Number of units	276
Operating pressure at pivot (kg/cm^2)	3
Drainage	
Type	Subsurface tile with open main collector ditches
Average depth (metres)	3
Percent of irrigation area	30
Total area (ha)	4,300
Tile spacing (metres)	250
Tile depth (metres)	2
Total Land Area	
Hectares	18,500
Feddans	44,000

Table K-22
MONTHLY IRRIGATION RATE AND WATER BALANCE
FOR THE FORAGE CROP PATTERN

Month	Net Irrigation Requirement[a] (mm)	Wastewater Supply[b] (mm)	Overirrigation (mm)	Shortage (mm)	Irrigated Land Area (%)
January	50	193	143	--	100
February	72	190	118	--	100
March	143	193	50	--	100
April	195	193	--	2	99
May	243	200	--	43	62
June	271	208	--	63	77
July	296	217	--	79	73
August	268	225	--	43	80
September	219	215	--	4	98
October	152	205	53	--	100
November	80	203	123	--	100
December	46	200	154	--	100
Total	2,035	2,442	641	234	--

[a]Average of alfalfa and grass. Does not include leaching requirement.
[b]Design wastewater flow rate divided by the total irrigated area.

5.17

Table K-23
ANNUAL NUTRIENT UPTAKE RATES
AND APPLICATION FOR THE FORAGE CROP PATTERN
(kg/ha)

Crop	Nitrogen		Phosphorus	
	Uptake Rates	Applied	Uptake Rates	Applied
Grass	150-650	635	30-80	250
Alfalfa	225-540[a]	609	20-35	245

Source: Reference 4

[a]Legumes will also take nitrogen from the atmosphere if not available in the soil.

Table K-24
ESTIMATED CAPITAL COSTS, LAND APPLICATION
ALTERNATIVES IIIA AND IIIB
(1000 LE)

Item	Alternative	
	IIIA	IIIB
Pretreatment		
Anaerobic Lagoons	7,937	12,321
Rapid Infiltration	24,879	—
Irrigation	87,875	64,846
Drainage	4,349	3,746
Administrative and General	2,924	2,769
Subtotal	127,964	83,682
25% Contingency	31,991	20,921
Subtotal	159,955	104,603
15% Engineering, Legal, and Administrative	23,993	15,690
TOTAL	183,948	120,293

Note: Costs are based on prices at the end of 1980 for a fully constructed system capable of handling the design flow for the year 2000. The effects of construction phasing are discussed in Chapter 5.

Table K-25
ESTIMATED ANNUAL O&M COSTS,
LAND APPLICATION ALTERNATIVE IIIA

Item	1980 Quantity	1980 Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	30,900	--	49,400	--	61,880	--
Cost	--	400	--	640	--	800
Power						
1,000,000 kWh	63.4	--	101.4	--	126.7	--
Cost	--	659	--	1,054	--	1,317
Fuel						
1000 litres	290.7	--	465.1	--	581.4	--
Cost	--	10	--	16	--	20
Materials						
Cost		823	--	1,317	--	1,646
TOTAL	--	1,892	--	3,027	--	3,783

Note: Costs based on prices at the end of 1980.

Table K-26
ESTIMATED ANNUAL O&M COSTS
LAND APPLICATION ALTERNATIVE IIIB

Item	1980 Quantity	1980 Cost (1000 LE)	1990 Quantity	1990 Cost (1000 LE)	2000 Quantity	2000 Cost (1000 LE)
Labor						
Mandays	23,400	--	37,400	--	46,800	--
Cost	--	311	--	498	--	622
Power						
1,000,000 kWh	36	--	58	--	72	--
Cost	--	265	--	424	--	530
Fuel						
1000 litres	315	--	503	--	629	--
Cost	--	11	--	18	--	22
Materials						
Cost		629	--	1,006	--	1,257
TOTAL	--	1,216	--	1,946	--	2,431

Note: Costs based on prices at the end of 1980.

Table K-27

ANNUAL NET INCOME FROM CROP PRODUCTION

Crop Pattern	Yield[a] (Mt/fd)	Farm Gate Prices (LE/Mt)	Gross Income (LE/fd)	Production[b] Cost (LE/fd)	Net[c] Income (LE/fd)	Proportion Cropland Area (%)
High Income						
Winter						
Berseem	12	11	132	57	75	
Wheat	1.3	77	100	60	40	
Barley	1.3	60	78	40	38	
Cabbage	9	27	243	110	113	
Peas	2	162	324	110	214	
				Average	110	100%
Summer						
Sugar Beets	14	25	350	85	265	
Breadbeans	1.0	120	120	32	88	
Tomatoes	7.0	60	420	130	290	
Potatoes	7.0	70	490	260	230	
Squash	7.0	87	609	190	419	
Carrots	10.4	30	312	100	212	
Corn	1.6	71	114	65	49	
				Average	222	100%
				Annual Total	322	
Forage						
Alfalfa Hay	23	22	506	160	346	50%
Grass Hay	10	10	100	45	65	50%
				Average	206	
				Annual Total	189[d]	

[a]Full yield after 5 years of reclamation.

[b]Includes all normal costs except irrigation and fertilizers.

[c]Net income = gross income - production costs (1980 costs).

[d]Alternative IIIB has varying amounts of land out of production during the summer, which will reduce the overall average crop returns by about 8 percent.

Table K-28
SOCIAL IMPACTS

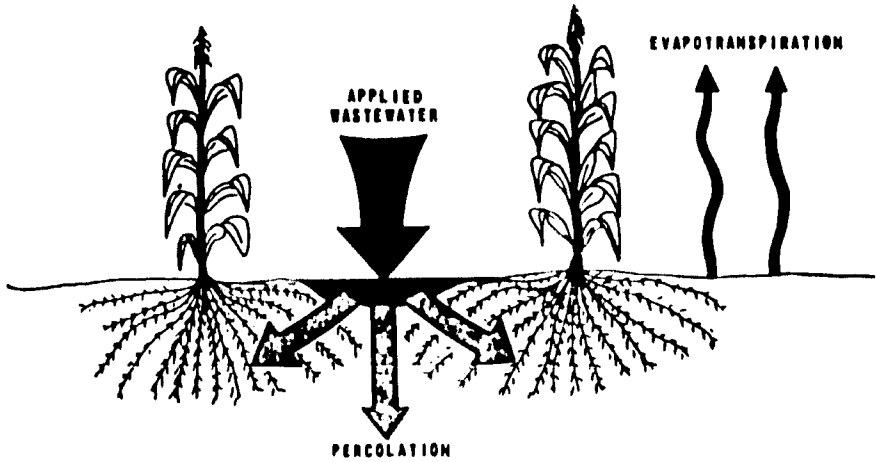
Impact Category	Alternative IIIA	Alternative IIIB
Agricultural Reclamation of Arid Areas	Uses wastewater to reclaim previously nonproductive land. Increases production of high-income crops. Affects pricing and marketability of locally grown crops.	Similar to IIIA except that production is limited to lower-value forage crops.
New Communities	Would result in development of rural communities consistent with national policy of population decentralization. Increases associated commercial and industrial activities. Relocation of Bedouins represents a minor adverse impact.	Similar to IIIA except less infrastructure is required.
Employment	Employment provided for technical, agricultural, clerical, and mechanical personnel.	Similar to IIIA except less infrastructure is required.
Odors	Odors associated with anaerobic lagoons; negligible odor problems anticipated with infiltration ponds and wastewater irrigation.	Odors associated with anaerobic lagoons and to a lesser extent the open canals conveying primary effluent.
Cultural/Historical	Locations of tombs and other potential resources at the East Site will be determined and catalogued. Design accommodations would be made as necessary.	Same as IIIA.
Social/Religious Acceptability	Provision of public water onsite for consumptive use and for religious ablutions should enhance acceptability.	Same as IIIA.

Table K-29
PUBLIC HEALTH IMPACTS

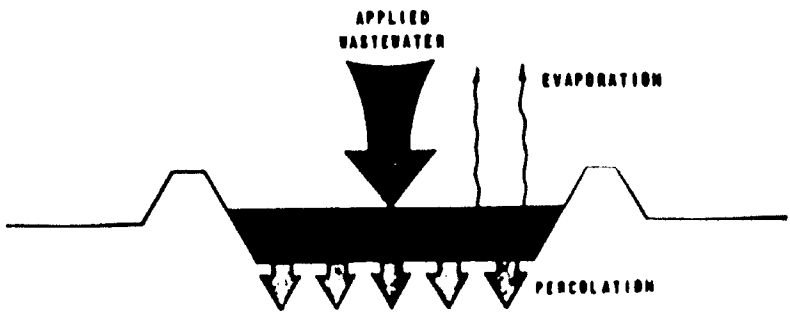
Impact Category	Alternative IIIA	Alternative IIIB
Toxic Chemicals	Minimal impact from effluent, since anaerobic treatment and rapid infiltration effectively remove many toxic chemicals. Industrial pretreatment necessary to reduce concentrations of hazardous substances in sludge. Agricultural management will require conventional use of pesticides to maximize crop production.	Same as IIIA.
Direct Contact With Waterborne Disease Organisms Prior to Crop Irrigation	Negligible risk due to the following design features: o Rapid infiltration effectively removes most pathogens. o Public access is limited. o Potable water supplied onsite. o Workers trained to follow sanitary procedures. o Low-pressure sprinklers minimize aerosol formation. o Windbreaks reduce aerosol transport.	Somewhat greater risk in irrigation distribution system than with IIIA, but risk is still acceptable.
Epidemiological Hazards Associated with Concrete-Lined Irrigation Canals	Potential adverse impacts associated with malaria vector and schistosomiasis intermediate host propagation are less severe than noted for other irrigation systems in arid lands. See text for mitigation measures.	Similar to IIIA, except disease control may be more difficult due to localized algal growth and siltation problems in some canals.
Direct Contact With Soil Pathogens in Agricultural Fields	Negligible impact, since most parasitic eggs and cysts are removed during rapid infiltration.	Adverse impacts can be minimized by training workers to wear boots and to avoid eating raw crops.
Pathogen Content of Crops After Harvest	Negligible impact since recovered water is safe for unrestricted crop selection.	Negligible impact since there will be no direct human consumption. Primary effluent precludes growth of all but forage and fiber crops.
Pathogen Content of Land Applied Sludge	Negligible impact since pathogen content is minimized by digestion, stabilization, and drying.	Same as IIIA.

Table K-30
ENVIRONMENTAL IMPACTS

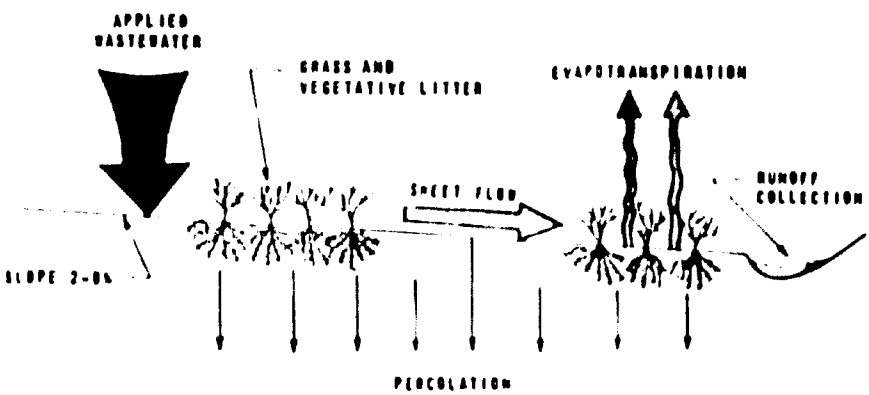
Impact Category	Alternative IIIA	Alternative IIIB
Terrestrial Ecology	32,000 ha of arid land replaced by irrigated lands. Negligible impact, but increased population density and other projects could hasten development of remaining arid lands.	Similar to IIIA, but 19,300 ha of arid land replaced by agricultural fields.
Surface Water Quality Quality and Drainage	Only subsurface drainage allowed to flow offsite. Twenty percent of irrigated area may require tile drainage to avoid waterlogging.	Similar to IIIA, except 30 percent of irrigated area may require tile drainage to prevent waterlogging.
Ground Water Quality and Quantity	Increased level of TDS expected below rapid infiltration basins. Height and movement of ground water managed by interceptor wells and drainage.	No adverse impacts anticipated, since percolate N concentrations are expected to be less than 10 mg/l.
Air Quality	Construction will increase dust levels. Particulate concentrations will be equivalent to surrounding agricultural areas.	Same as IIIA.
Local Climate	Localized reductions in ambient air temperatures at land application sites.	Same as IIIA.



IRRIGATION



RAPID INFILTRATION



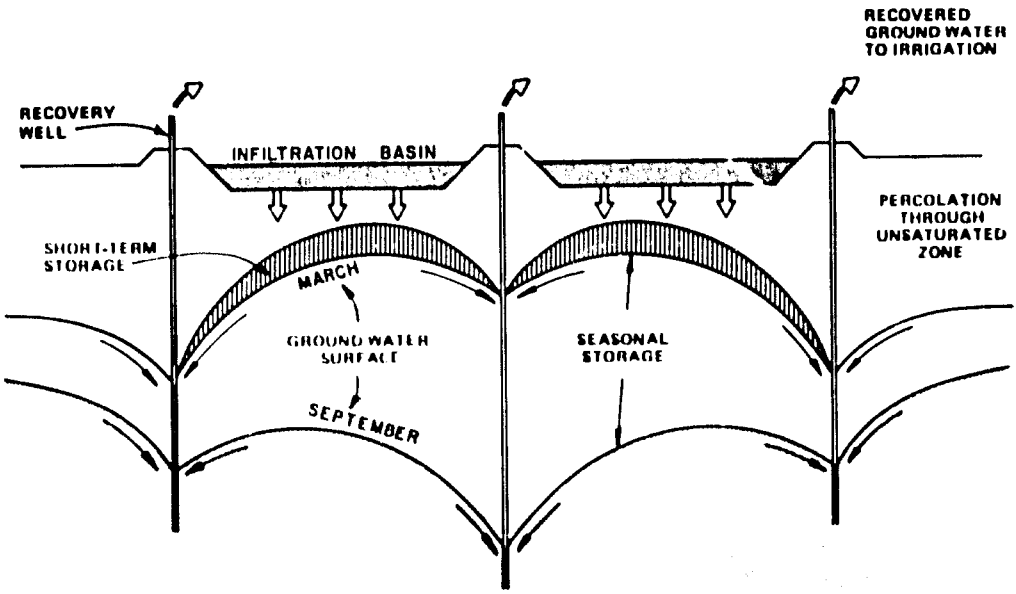
OVERLAND FLOW

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981

FIGURE K-1

LAND APPLICATION PROCESSES

A GOSO WWCG WECO



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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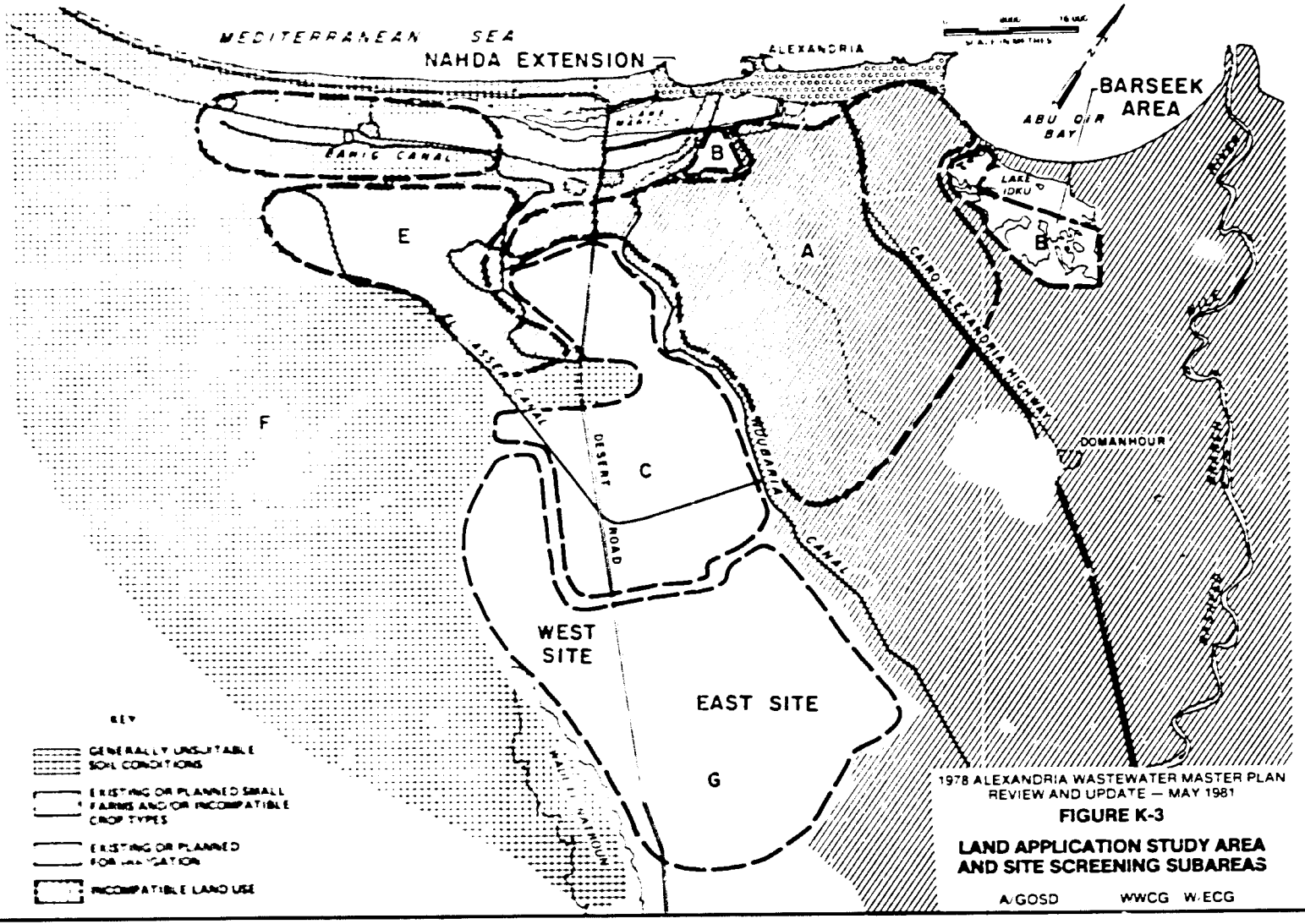
FIGURE K-2

GROUND WATER STORAGE
RAPID INFILTRATION SYSTEM

A/0/50 WWCO W ECO

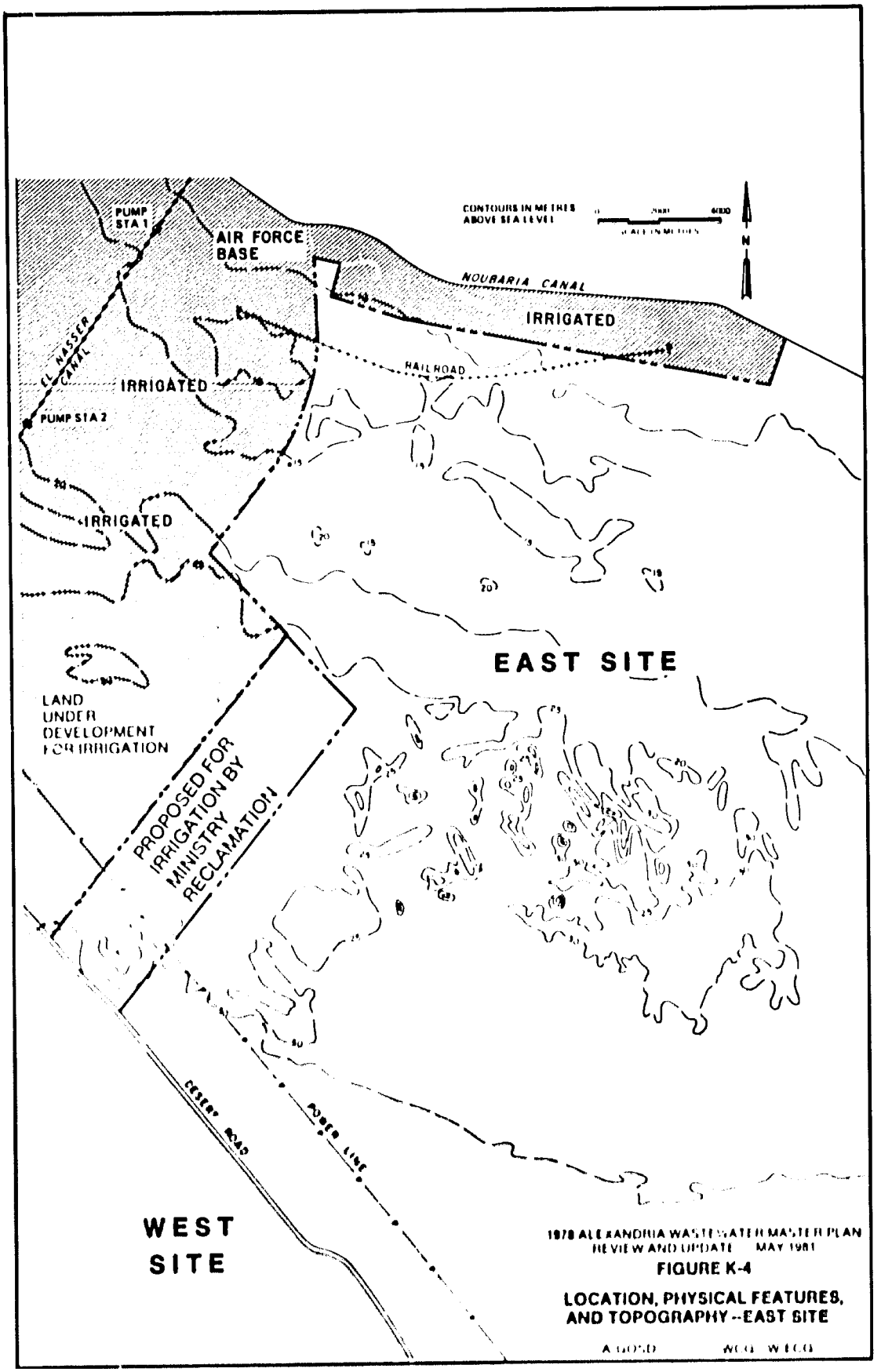


1978 ALEXANDRIA WASTEWATER MASTER PLAN REVIEW AND UPDATE — MAY 1981

FIGURE K-3

LAND APPLICATION STUDY AREA AND SITE SCREENING SUBAREAS

A/GOSD WWCG W/ECG

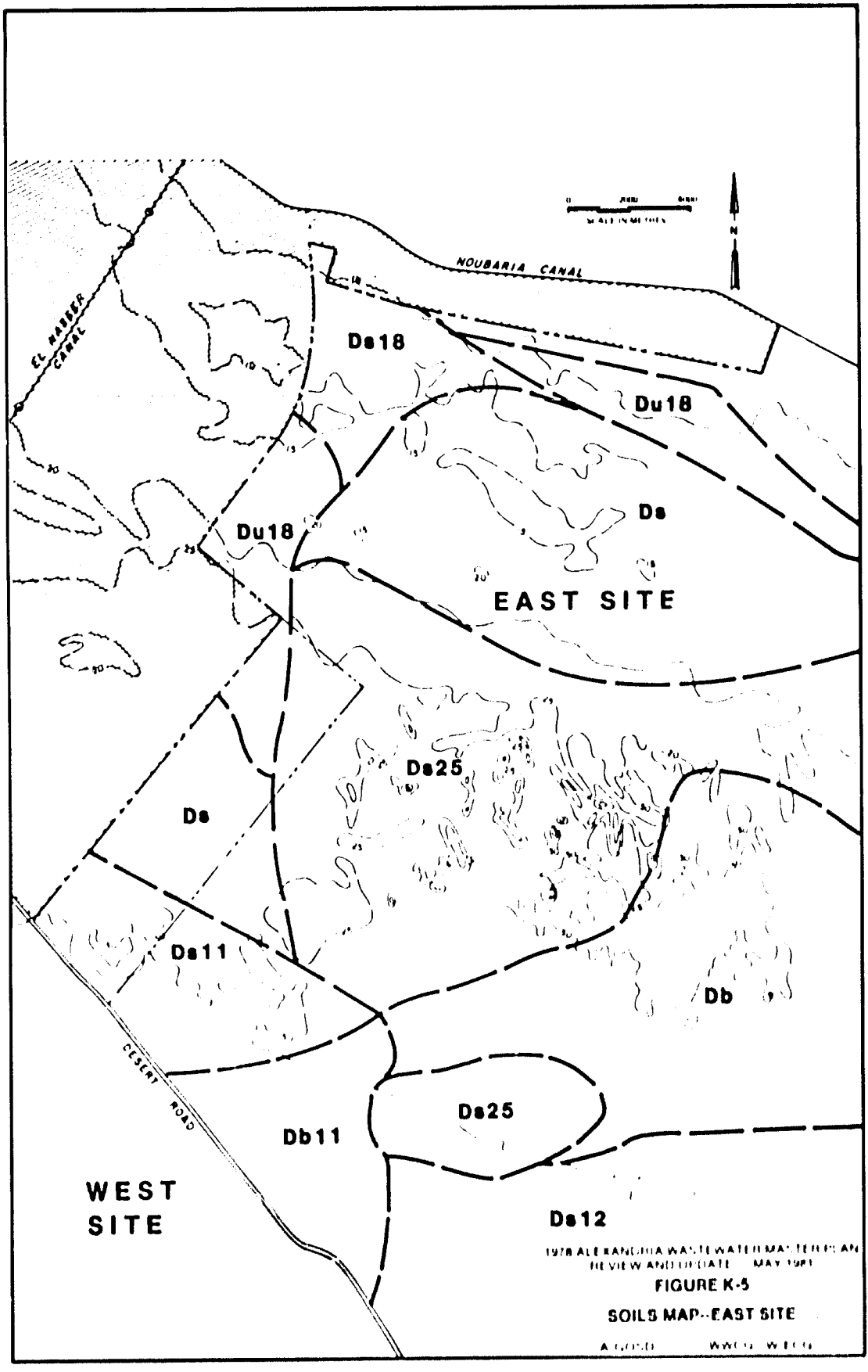


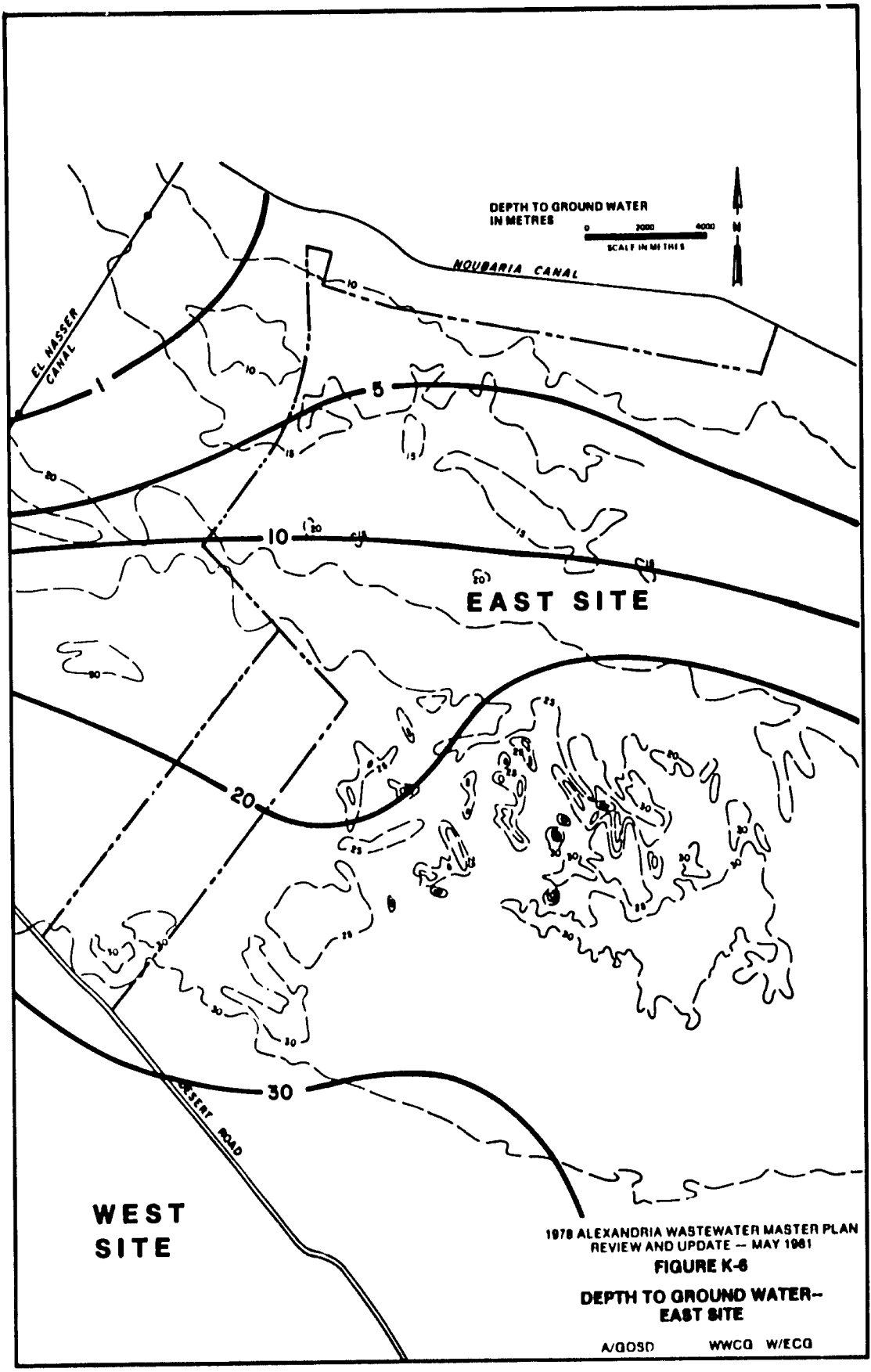
1978 ALEXANDRIA WASTEWATER MASTER PLAN REVIEW AND UPDATE MAY 1981

FIGURE K-4

LOCATION, PHYSICAL FEATURES, AND TOPOGRAPHY--EAST SITE

A GOSD WEG WECG


NOUBARIA CANAL
EL MAHMUDIYA CANAL
Ds18
Du18
Du18
Ds
EAST SITE
Ds25
Ds
Ds11
Db
DESERT ROAD
Db11
Ds25
WEST SITE
Ds12
1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE MAY 1981
FIGURE K-5
SOILS MAP--EAST SITE



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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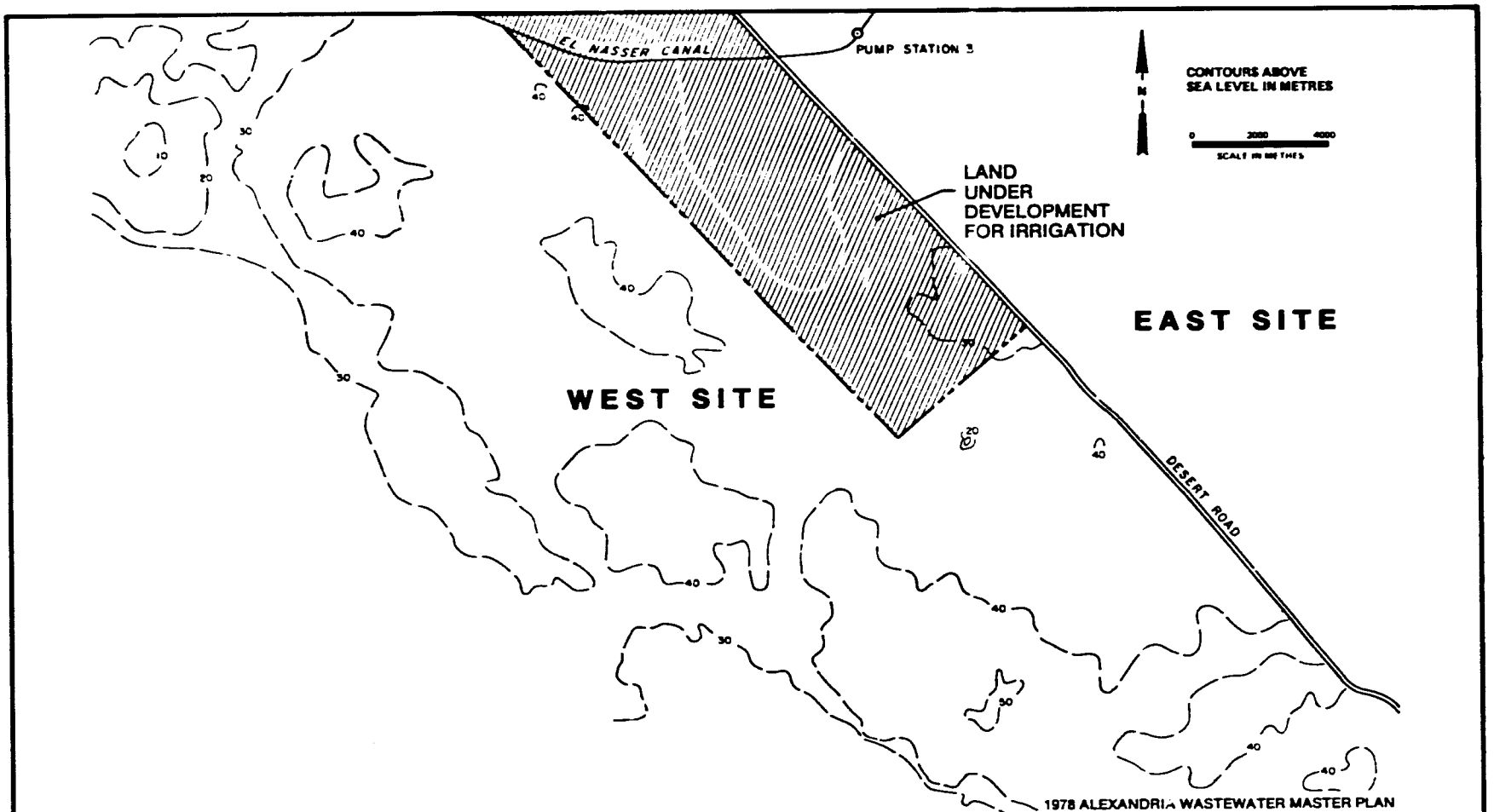
FIGURE K-6

**DEPTH TO GROUND WATER—
EAST SITE**

A/GOSD WWCO W/ECO

5



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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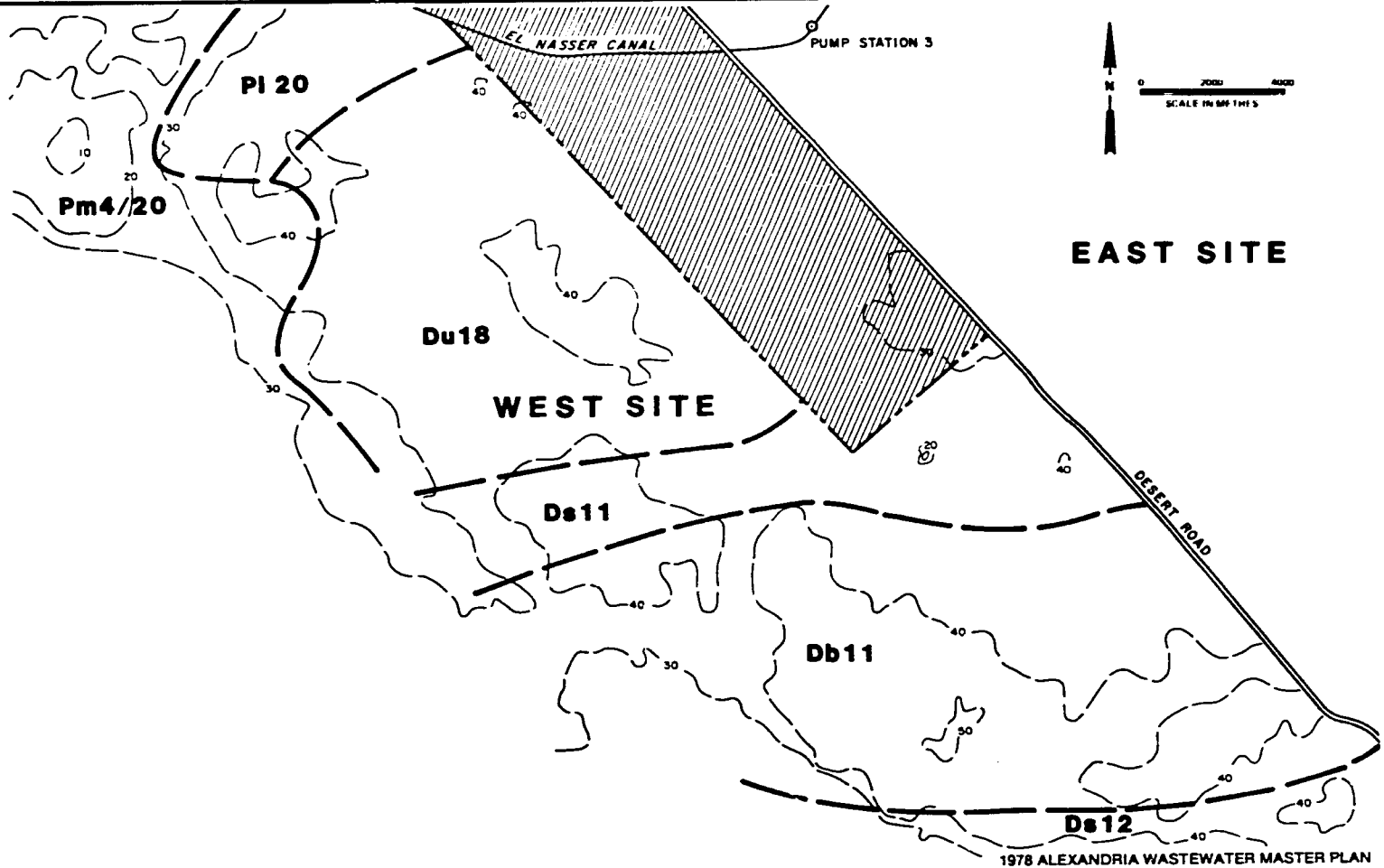
FIGURE K-7

LOCATION, PHYSICAL FEATURES, AND TOPOGRAPHY—WEST SITE

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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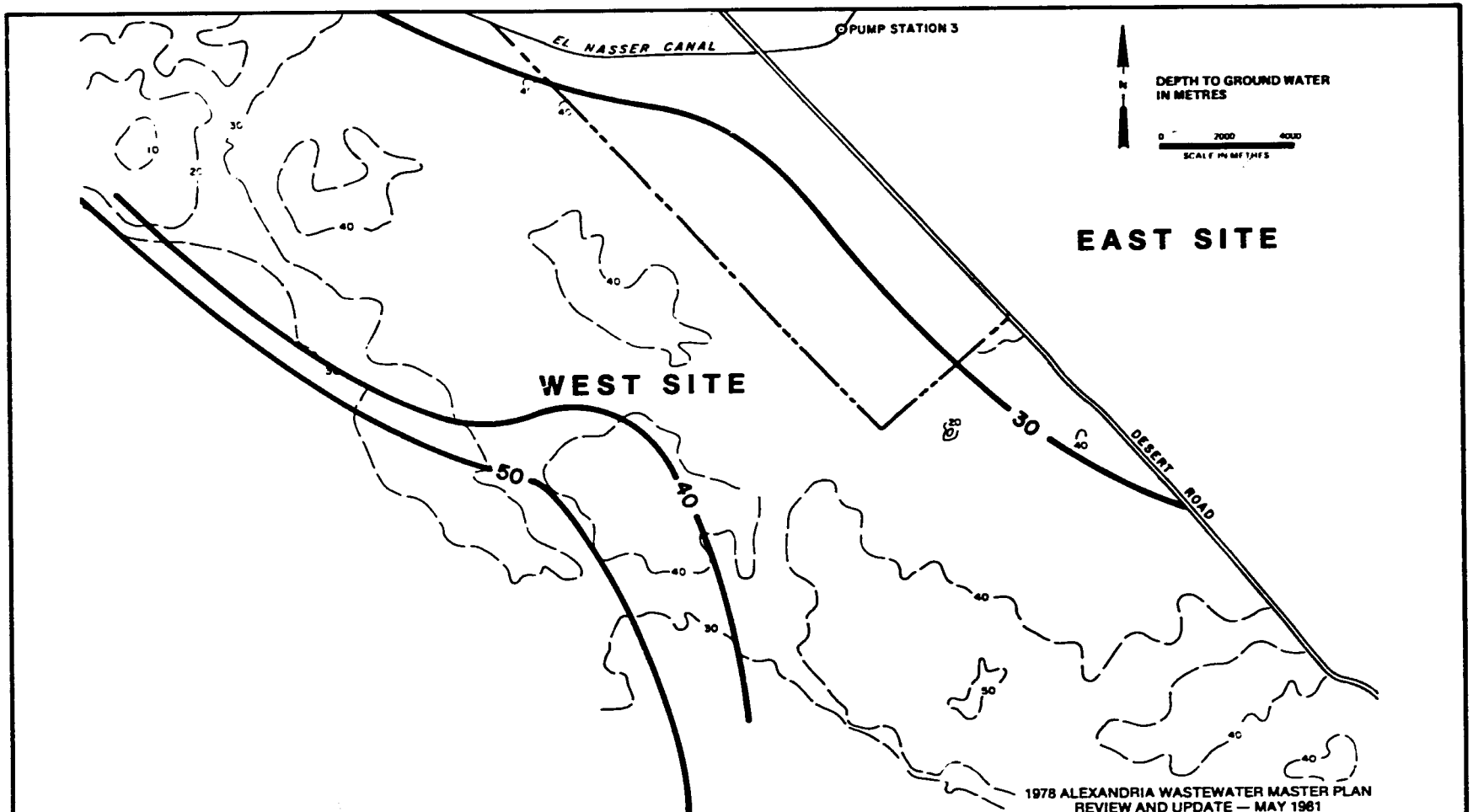
FIGURE K-8

SOILS MAP—WEST SITE

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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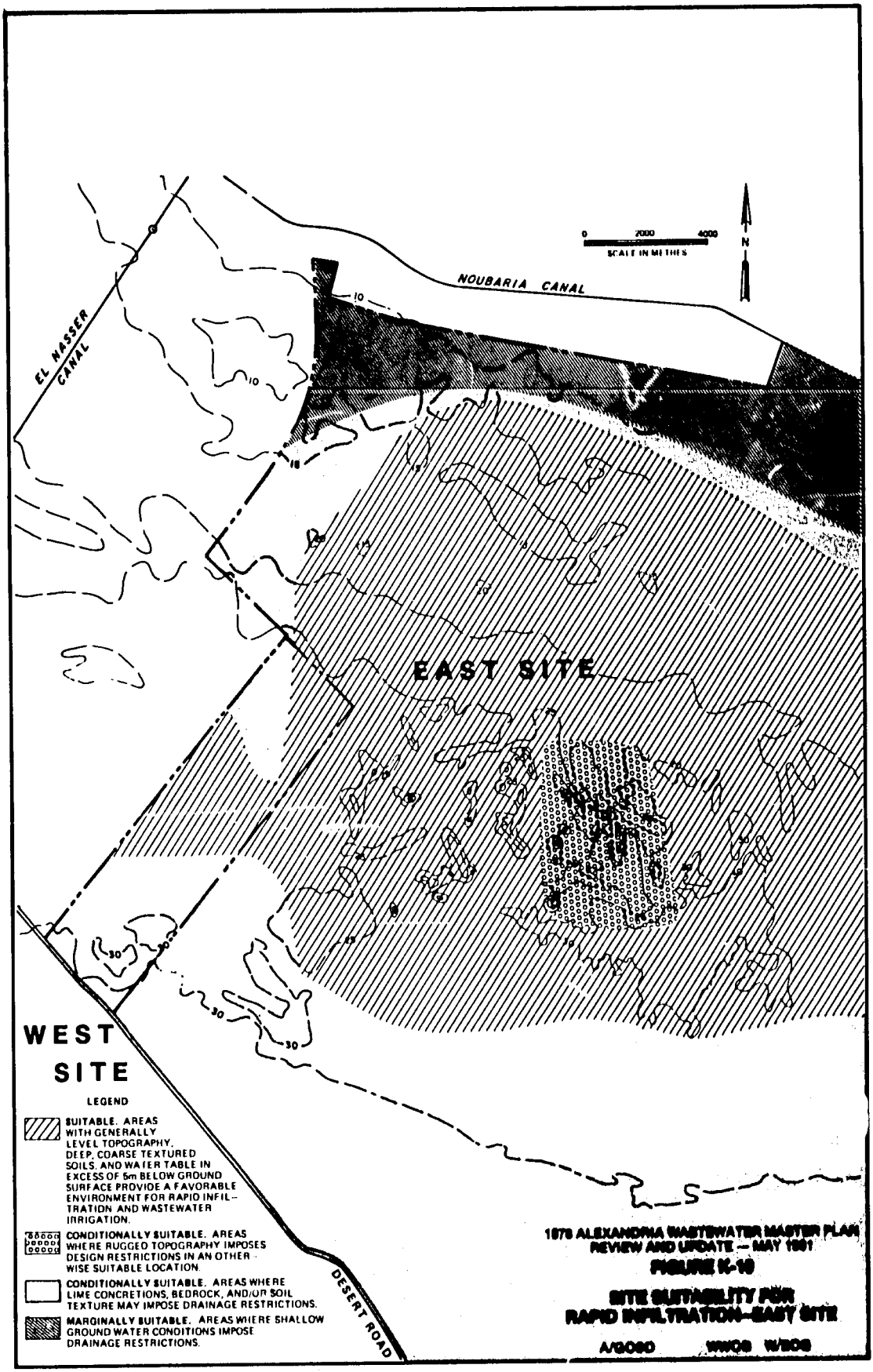
FIGURE K-9

**DEPTH TO GROUND WATER--
WEST SITE**

A/GOSD WWCG W/ECG

592



1979 ALEXANDRIA WASTEWATER MASTER PLAN
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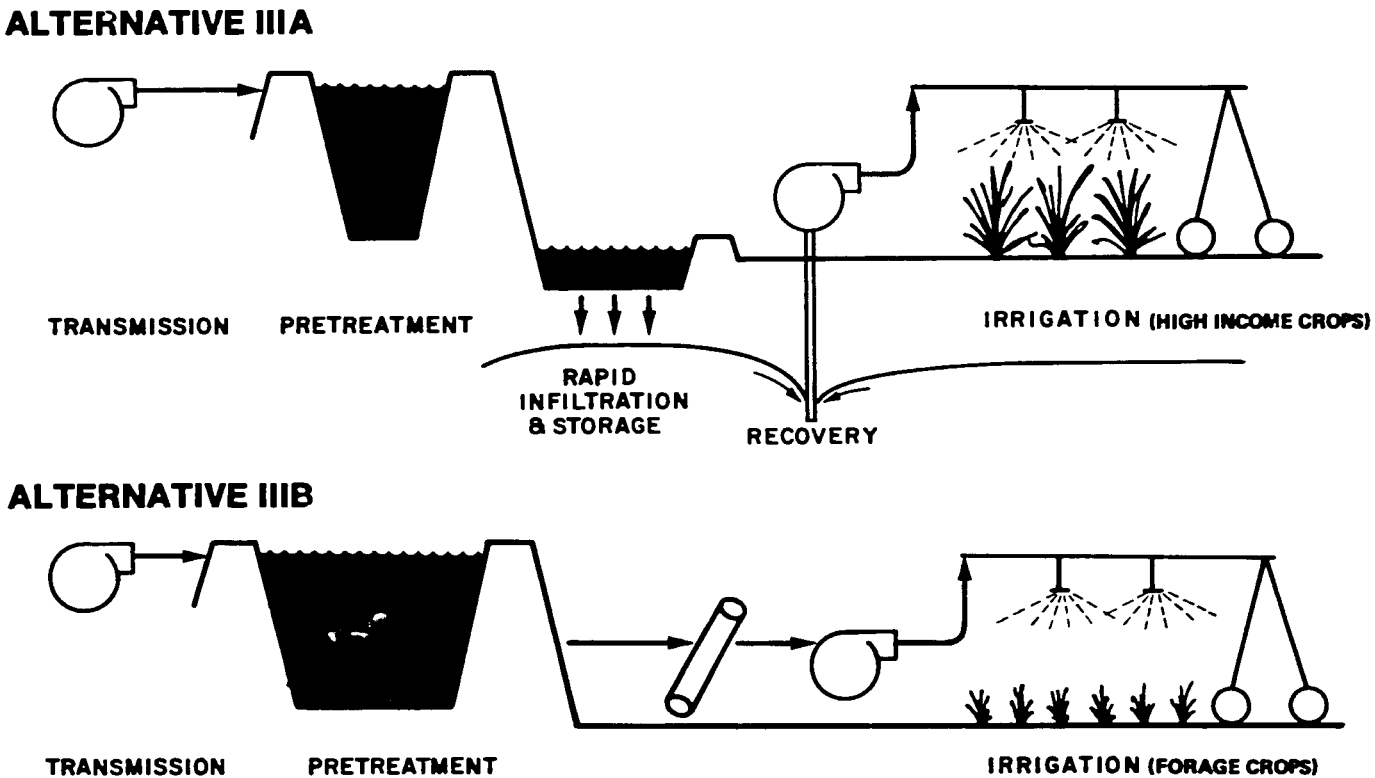
FIGURE K-10

SITE SUITABILITY FOR
RAPID INFILTRATION—EAST SITE

A/GOBO WWOB W/BOB



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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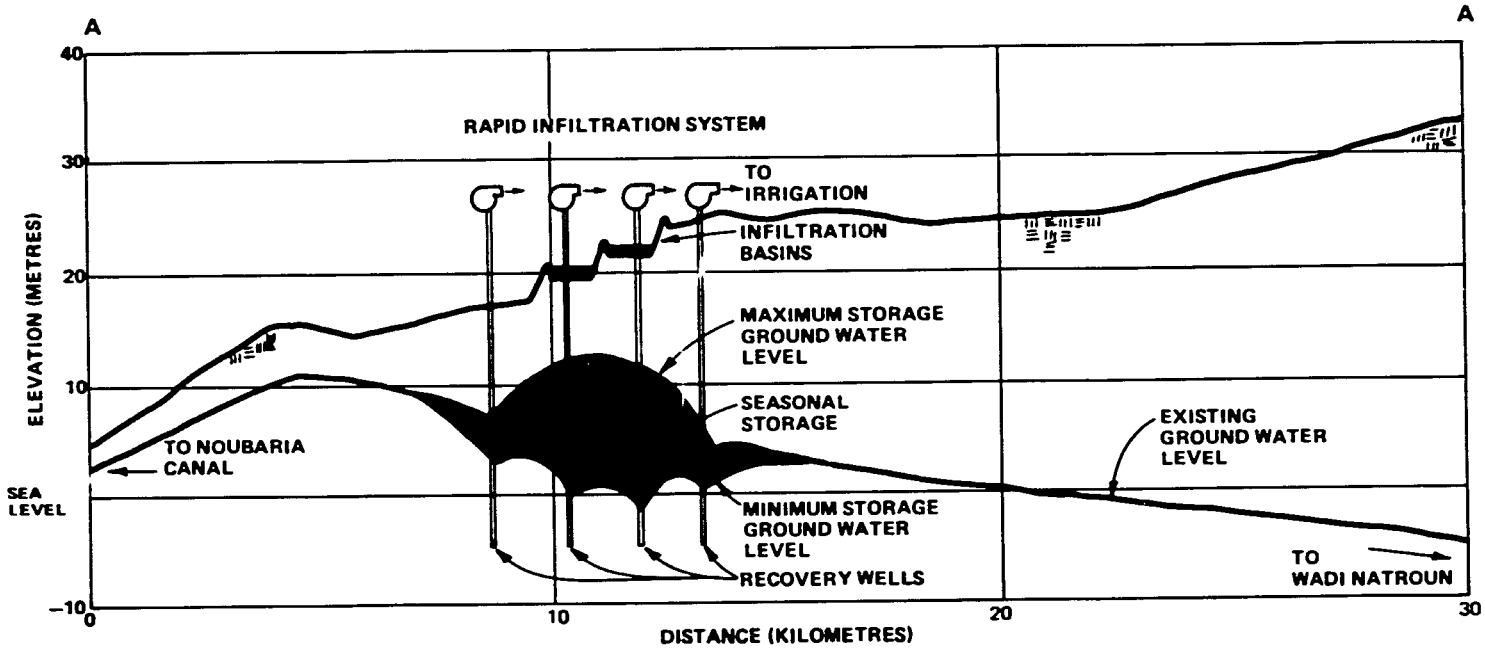
FIGURE K-11

SCHEMATIC DIAGRAMS OF LAND APPLICATION ALTERNATIVES

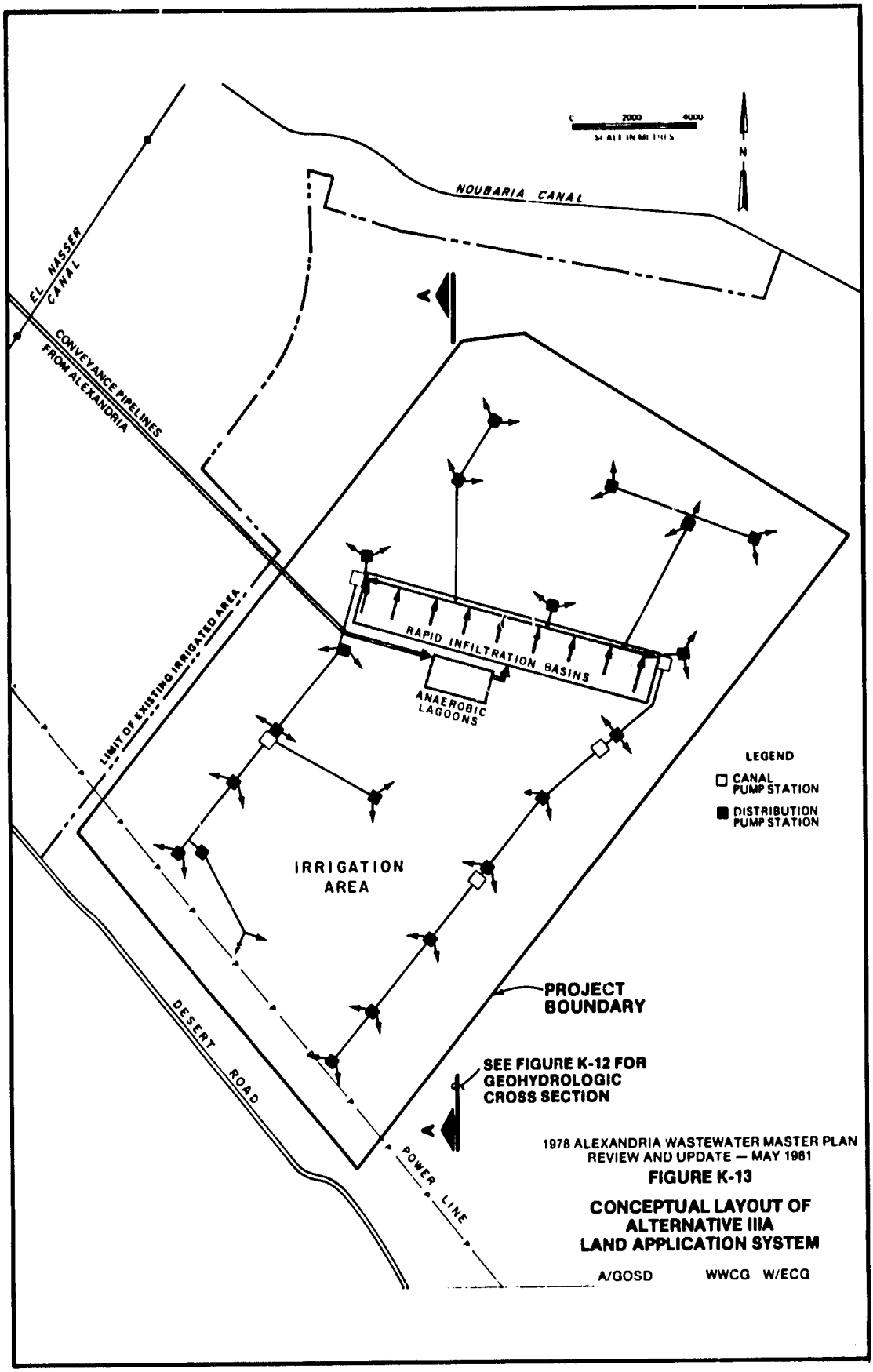
A/GOSD WWCG W/ECG



NOTE: SEE FIGURE K-13 FOR CROSS SECTION LOCATION

FIGURE K-12
GEOHYDROLOGIC CROSS SECTION RAPID INFILTRATION EFFECT ALTERNATIVE IIIA

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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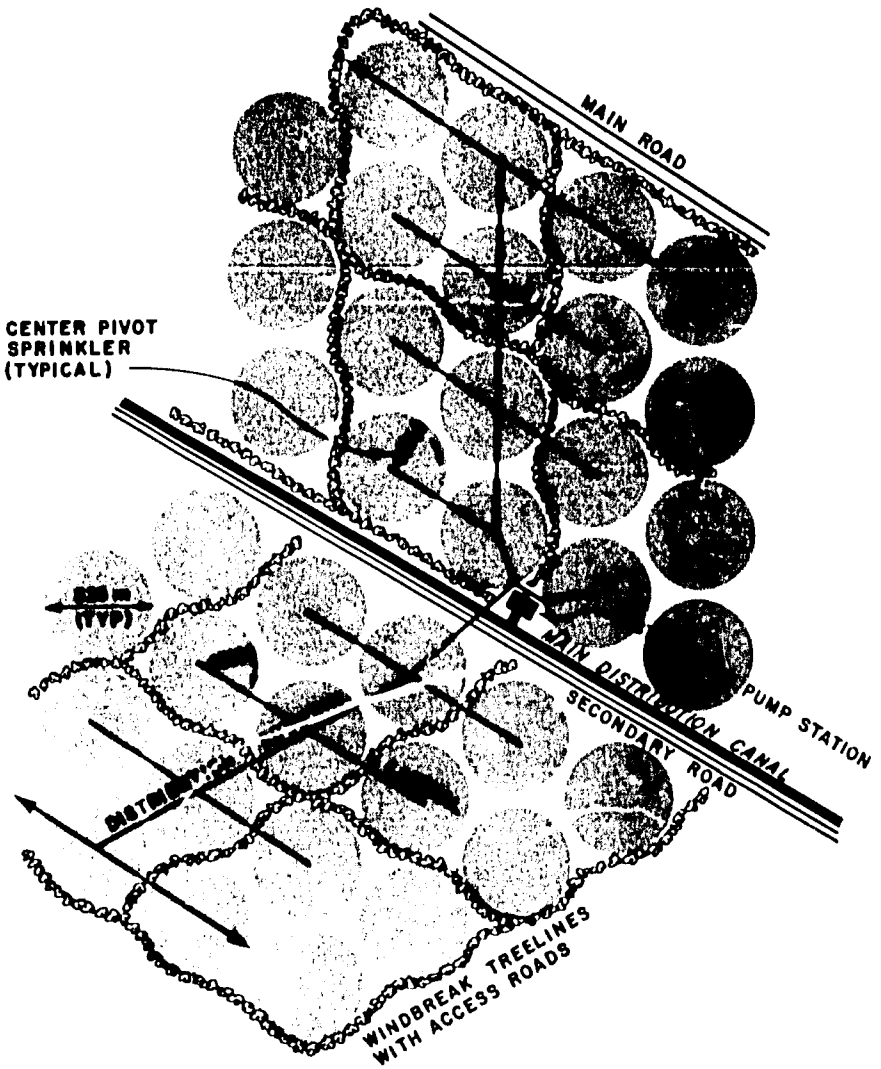
FIGURE K-13

CONCEPTUAL LAYOUT OF ALTERNATIVE IIIA LAND APPLICATION SYSTEM

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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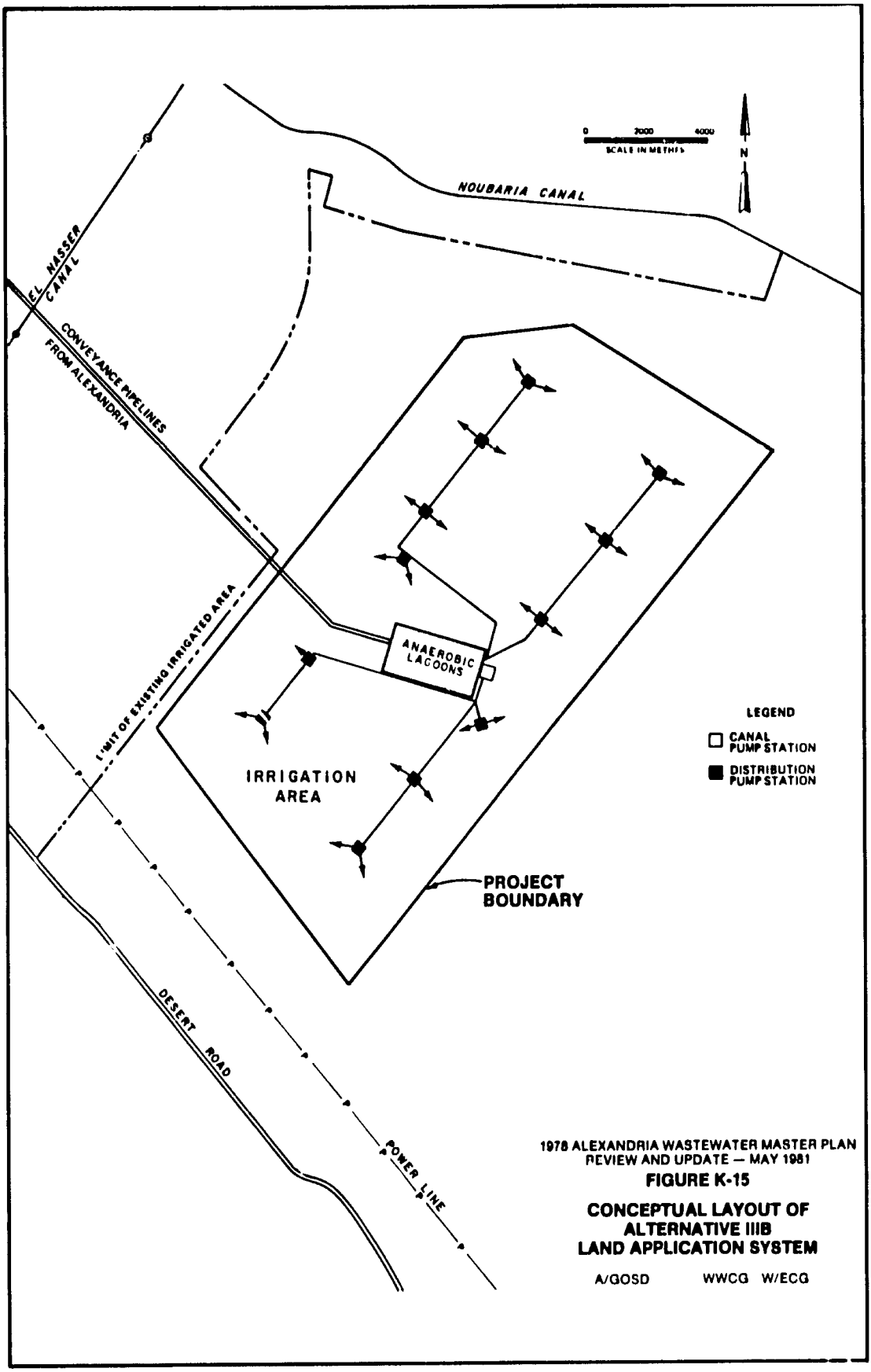
FIGURE K-14

DETAIL OF TYPICAL
IRRIGATION DISTRIBUTION ZONE

A/GOSD WWCG W/ECG



1978 ALEXANDRIA WASTEWATER MASTER PLAN
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FIGURE K-15

CONCEPTUAL LAYOUT OF ALTERNATIVE IIIB LAND APPLICATION SYSTEM

A/GOSD WWCG W/ECG

APPENDIX L

SEA DISPOSAL AND OUTFALL REVIEW

INTRODUCTION

This appendix summarizes the review of sea disposal of treated wastewater from Alexandria as proposed by the 1978 Master Plan and subsequent modifications resulting from the environmental impact statement (EIS). The modified concept proposed that primary treatment and disinfection by chlorination would precede disposal through two long (8 km and 10 km) outfalls extending seaward from Kait Bey and Sidi Bishr, respectively.

Data contained both in published Master Plan reports and unpublished memoranda were reviewed, and additional library information available in the United States was also utilized. In addition, Egyptian consultants to WWCG from the academic community provided review comments and supplied summaries of various reports, theses, and published technical papers pertinent to the water quality and marine ecology of the Alexandria region. Computer models, particularly "PLUME" (a dispersion model) were employed to assist WWCG in the review. A bibliography at the end of this appendix lists and abstracts the major documents reviewed.

Figure L-1 shows the currently proposed outfall locations as well as the original Sidi Bishr alignment. The decision to add primary (rather than preliminary) treatment to the sea disposal plan resulted in a need to move the proposed Sidi Bishr outfall alignment eastward approximately 2450 m. The current WWCG scope of work excludes gathering new field data; accordingly this review, particularly as regards the Sidi Bishr outfall is hampered by the lack of a bottom profile and geophysical data.

During our review, WWCG learned of the existence of certain offshore current meter transects, which resulted from studies sponsored by the UNDP-UNESCO and conducted by the Egyptian Academy of Scientific Research and Technology. Efforts to obtain this information, which may be helpful to supplement the limited current studies performed during the 1978 Master Plan studies, proved fruitless.

1978 MASTER PLAN RECOMMENDATION

The 1978 Master Plan (as subsequently modified) recommends sea disposal of chlorinated, primary effluent generated by wastewater flows from the eastern, central, and western sewerage zones of Alexandria. The outfalls proposed are briefly summarized in Table L-1.

No discharge criteria for marine disposal had been established by either the Egyptian Government, the United Nations Environmental Program, or other international organizations concerned with land-based discharge to the Mediterranean Sea as of 1978. No new specific discharge criteria have been established since the Master Plan. The only guiding principles are Egyptian Public Law No. 93, which prohibits discharge of substances harmful to beaches and aquatic life, and the MED POL agreements started in 1978. These agreements are contained in the Mediterranean Action Plan (United Nations 1978). They include the Convention for the Protection of the Mediterranean Sea Against Pollution, which acknowledges that appropriate measures to prevent pollution from outfalls be taken, and the Protocol for the Prevention of Pollution of the Mediterranean Sea by Dumping from Ships and Aircraft, which includes a list of forbidden substances but does not apply to land-based sources. The water quality criteria used to evaluate sea disposal in the 1978 Master Plan were therefore developed based on discharge requirements for countries other than Egypt. These criteria, hereinafter referred to as the 1978 criteria, were developed by the 1978 Master Plan consultant and are listed in Table L-2.

REVIEW OF FIELD STUDIES

Several categories of field studies were reported in Vol. IV of the 1978 Master Plan. These studies are discussed and evaluated in the following sections.

Water Quality and Bacteriological Studies

Specific water quality criteria for the ocean had not been set by Egyptian law during the work performed in 1976-1978. Egyptian Law No. 93 of 1963 states that: "It is permitted to discharge all kinds of wastewaters into the sea and lakes if they do not contain any matter which may harmfully affect the beaches, navigation establishments, fish and other aquatic life."

1978 criteria for BOD, suspended solids, phosphorus, and nitrogen were set by using the equation:

$$S = \frac{(C_e - C_o)}{(C_r - C_o)}$$

where C_r = the maximum permissible concentration limit, C_o = background concentration, C_e = effluent concentration, and S = dilution. This is based upon the equation for heavy metals discharge criteria in California, U.S.A.

BOD. For BOD, a standard was adopted by setting C_r = 4.4 mg/l so that the average dissolved oxygen (DO) concentration of 6.4 mg/l would not be reduced to less than 2.0 mg/l. A background concentration of 1.0 mg/l (= C_o) was used. Resulting required initial dilutions (S) were 141 for Sidi Bishr, and 150 for Kait Bey.

Suspended Solids. For suspended solids, C_e = 600 mg/l and 536 mg/l for Sidi Bishr and Kait Bey, respectively, for the year 2000 if preliminary treatment were adopted. A standard of one turbidity unit per suspension of 1.0 mg/l and equal to a suspension of 1 mg/l SiO_2 was adopted.

A background value for C_o of 0.1 mg/l, employing a Secchi depth of "many tens" of metres, and a criteria value of 1.0 mg/l = C_r, equal to a Secchi of approximately 10 m were used. Resulting dilutions would be 567 and 595 for Sidi Bishr and Kait Bey.

The rate of settling solids resulting from the suspended solids was estimated using a settling rate of 37 m/day, and the equation of x = uh/w, where x = distance of area traveled, u = ocean current velocity, h = height of rise, and w = width. Using u = 0.10 m/sec, results in x = 4700 to 7000 m, and w = approximately 1 km. Settling velocities in the ocean were assumed similar to those in a primary clarifier and the effect of saline water buoyancy and saline induced coagulation and flocculation were assumed to be mutually compensating. The 1978 Master Plan also suggested that there would be little problem with suspended solids accumulation since grab samples within 100 m of the present (Kait Bey) outfall showed clean sand.

Nitrogen and Phosphorus. A background C_o of 0.12 microgram-atoms (ug/at) per IP and 0.10 ug-at/l inorganic N (NO_3 + NH_4) was set. Nitrogen was claimed to be the limiting element, and the 1978 criteria were based on N. Limits were based on a C_r = 0.028 mg/l (2.0 ug-at/l) based on C_r = 1/10 inshore inorganic N levels. C_e's are 31 mg/l N for Sidi Bishr and 24 mg/l at Kait Bey. Required dilutions would be 1165 and 902, respectively.

Bacteria. A bathing water standard of 100 to 1000 fecal coliforms (FC) per 100 ml was used for designing outfalls 10 km offshore. The rationale for this approach was as follows: undiluted effluent has a bacterial concentration of 1 x 10^9 FC per 100 ml. Total dilution is 10^3 before reaching shore, reducing bacterial concentration to 10^6. The time of transport from outfall to beach is 30 hours, allowing a decay rate of 10^4-10^3. The decay rate is based on a daytime T_{90} of 1 hour and a nighttime T_{90} of 40 hours.

Toxicants. No standards were set.

Evaluation of 1978 Criteria

As seen in Table L-2, the designed initial dilution of 150 in conjunction with only preliminary treatment would result in violations of all water quality standards within the zone of initial dilution (ZID) adopted by the 1978 criteria except BOD (DO). Total dilution (initial and subsequent) was relied upon to meet other water quality standards. Since the influence on DO due to BOD takes place on a time and space scale equivalent to the influence of suspended solids on turbidity and nitrogen on algae, using a different dilution basis is inconsistent. Since no standards for toxicants were adopted, the requirement of Law No. 93 that fish and other aquatic life be protected is not met. In addition to these two basic problems, technical problems exist with each of the adopted standards or the approach to obtain the standard. Each of these problems is considered below.

BOD or Dissolved Oxygen. Allowing dissolved oxygen to be reduced to 2.0 mg/l is too low a level to protect aquatic life. The US EPA recommends a minimum of 5.0 mg/l, and the State of California uses a standard of not more than ten percent depression within the ZID. A level of 2.0 mg/l within the ZID would result in depression of DO to less than 70 percent of that outside the ZID.

A C_0 of 6.4 mg/l oxygen was used as a baseline condition. This is not the critical value for dissolved oxygen, since lower levels (4.8 and 5.7 mg/l) were recorded. If the BOD loading was exerted during a critical DO period of 4.8 mg/l, the resultant DO would be approximately 0.4 mg/l.

DO records for the outfall area are grossly inadequate, with data only available for October through early January. A complete annual record of dissolved oxygen is needed. Dissolved oxygen data was expressed in mg/l rather than the more commonly accepted terms of percent saturation. DO concentration is dependent on temperature and salinity. To compensate for these two variables, percentage saturation should be calculated. Recalculation of the DO values reported yielded values as high as 350 percent of saturation. Such high values are unusual and do not agree with data reported by Hanafy in his 1979 M. Sc. thesis. Percent saturation values were reported in 1979 work to range from 54 percent near the Kait Bey outfall to 147 percent near the Glym beach to the east.

Given that DO can naturally be below the EPA recommended level of 5.0, a percentage reduction standard appears to be appropriate.

Suspended Solids. As mentioned in the 1978 Master Plan, the proposed relationship between suspended solids and Secchi depth is tenuous. The

allowed C_r of 10 m would be a reduction of 75 to 80 percent if C_o were 40 to 50 m. As a comparison, the California Ocean Plan calls for no statistically significant ($P <= 0.05$) reduction outside the ZID.

Reduction of 75 to 80 percent within the ZID would undoubtedly result in significant reductions outside the ZID.

The background C_o of 0.1 mg/l would be a Secchi depth of approximately 125 m with the equation used. A Secchi depth of 40 m would indicate a suspended solids level of 0.34 mg/l.

Accumulated sediment impacts have been erroneously analyzed:

- The rate of accumulation can be related to impacts on biota (see Word, 1978, and Bascom, et al. 1978).

- The fact that no accumulation was found within 100 m of the existing outfall is no indication that there will be no impact at the new outfalls. The existing outfall is at a depth of 16 m in a sandy area. Both facts indicate a higher level of turbulence and less settling of particles than would occur at the designed outfalls' depth of 50 m, one in an area of clayey silt.

Nitrogen and Phosphorus. Data presented are insufficient to indicate which element is limiting or to realistically set C_o.

According to the report by Dr. Richard L. Haedrich to the 1978 Master Plan consultant, P is limiting with N:P ratios of 40:1 and P concentrations of 0.025 ug-at/l. The standards, however, are set on nitrogen, and the 1978 criteria claim that nitrogen is limiting. Additional data on nitrogen and phosphorus reported by Hanafy (1979) apparently confirm that nitrogen is not limiting.

Eutrophication in the marine environment plankton communities is not a serious problem, providing it occurs in open areas with good dilution and dispersion. The emphasis should be on light and suspended solids, not biostimulation.

Bacteria. A C_e of 1×10^9 was used, but values as high as 7.0×10^{11} were measured and a log mean of 3.4×10^9 was used. The log mean value was computed ignoring equal to or greater than signs, which will result in an underestimate. Additionally, only 14 measurements were made, which is minimal to determine any distribution of bacterial numbers. The 1×10^9 is optimistically low, based on the data, and a more conservative approach would have been to use 1×10^{11}. The 10^9 is several orders of magnitude higher than that expected in U.S. domestic wastes.

Duration of bioassays was inadequate to obtain realistic values of T_{90}. Additionally light, temperature, and salinity need to be measured during the test since these all alter the decay rate. These should be repeated with the intention of collecting data to calibrate the equation formulated by Mancinin (1978).

Marine Biology and Ecology

Limited field studies, including bacteriological, water and sediment quality, phytoplankton, zooplankton benthic sampling, a fouling organism study, and surveys of commercial fish were conducted during the work associated with the 1978 Master Plan.

Bacteria. Samples were collected in the existing Kait Bey outfall plume on September 5, September 28, and November 26, 1977. The sewage plume was marked by Rhodamine dye and a drogue (no drogue was used on September 5). Samples for salinity, dye concentration, total and fecal coliforms, and fecal streptococci were taken in the dye patch. Physical dilution was calculated from dye data, and decay and sedimentation were estimated from bacterial reduction in excess of physical dilution. A daytime T_{90} estimate of 1 hour resulted.

During June 26 and 27, 1978, bacterial reduction studies were conducted in polyethylene bags. Samples for total and fecal coliforms, enterococci, and fecal streptococci were taken every 2 hours for 8 to 20 hours, depending upon the tests. Tests were started at different times of the day. A calculated daytime T_{90} of 1.5 hours and nighttime of T_{90} of 40 hours resulted.

Samples for bacteria were taken at several places in the sewage system between September 29, 1977, and June 17, 1978. Log means resulting from a total of 14 observations were 9.6×10^9 total coliforms, 3.4×10^9 fecal coliforms, 7.9×10^{10} total enterococci, 4.2×10^{10} fecal enterococci.

Water Quality. Water quality was measured near the proposed outfalls on four occasions. Samples were collected in a Kemmerer sampler and transported for analysis. Variables measured and the range of values were pH (7.9-8.0), turbidity (1 to 10), BOD (1.0 mg/l), $KMnO_4$ (1 to 1.6 mg/l), albuminoid-N (0.04-0.27 mg/l), and PO_4-P (0.10-0.35 mg/l). In almost all cases, highest values were from a station 2 km northwest of the Eastern Harbor entrance. A Hydrolab TDO-2 dissolved oxygen-temperature meter was used for temperature and dissolved oxygen.

Dissolved oxygen minimums were recorded in August and maximums in winter, with an earlier maximum inshore (January) than offshore (March

through May). Higher and lower values were recorded inshore than offshore. Offshore minimums were 4.8 in August, 5.9 in September, and 5.7 in January. Ample oxygen for waste assimilation was concluded to be available.

Plankton. Plankton were sampled at three stations near the Kait Bey alignment, with Station 3 being northwest of the proposed diffuser. From August to September phytoplankton were sampled with a plankton net 0.25 m in diameter and 20 meshes per cm. From October onward, samples were collected with a Kemmerer bottle and fixed with formalin. Settled organisms were counted. Zooplankton were collected with a 0.52 m diameter, 20 holes/cm mesh net. Horizontal, vertical, and oblique tows were used. Plankton genera were subjectively ranked as abundant and most abundant, but cells/litre were listed in the raw data appendices.

Zooplankton showed a general rise in abundance in the fall (late November), and other small increases in February and April.

Benthos. Benthos was sampled using a hexagonal sled, 55 cm wide in the middle and 35 cm wide on each of the 6 sides. A coarse mesh net (2-cm squares) was used with the sled. One station was sampled in the proposed outfall area approximately 2.5 km to the northwest of the Kait Bey diffuser site. This station was sampled once (January 6, 1978).

In addition, two sediment cores were taken near the Kait Bey diffuser site and three sediment cores were taken along the Sidi Bishr outfall (original alignment), one very near the diffuser.

Benthos sampling resulted primarily in the capture of molluscs, although many were just shells. Light now penetrates to the bottom, as shown by algae collected, and fauna representative of both hard and soft substrates were present. Bottom conditions were reported as "rocky muddy."

The sediment samples show major differences between the two diffuser sites. At the Kait Bey diffuser site, the sediment is reported as coral gravel, coral, and sand 2 mm in size. At the Sidi Bishr outfall site, sediments were composed of grey-white silt and clay.

Fish. Fish were not directly sampled in the proposed outfall areas. Instead, a survey of commercially landed fish was conducted. The Eastern Harbor, Abu Qir Bay, and Maadiya were visited weekly and the abundance of each fish was estimated by counting the number of fish boxes. These fish are captured by a variety of methods. Thirty-seven families of fish were represented in the catch.

Evaluation of Marine Biology and Ecology Studies

The limited scope of oceanographic work was insufficient to permit final design of outfalls intended to safely disperse unchlorinated Alexandria sewage which had received only preliminary treatment. Technical problems identified with the marine biology and ecology studies are discussed hereinafter.

Bacteria. One to two litres of dye are insufficient to meet the requirements of a study like this. Typically, 30 to 40 litres would be a minimum for a dye patch in the plume, or a short-term injection into the wastewater discharge.

Position locations using sextant angles are off by ± 200 m, as shown by the Southern California Coastal Waters Research Program. More exact locating methods are needed.

Bacterial decay studies in the polyethylene bags suffer from severe limitations as follows:

- Not enough baseline information was collected. Temperature, salinity, and light intensity, all of which change seasonally and diurnally, affect decay rates. If such baseline data were available, a more generalized equation such as the one by Mancini (1978) could be used.

- Sewage was filtered through glass wool, thereby reducing the number of "particulate" bacteria. Bacteria attached to particles generally live longer than floating bacteria.

- As mentioned in Vol. IV of the Master Plan, the tests were too few and not of long enough duration.

Bacterial concentrations in sewage will vary diurnally. Sampling times were not stated, nor is there evidence of finding peak concentrations.

In calculating the log mean of bacterial samples, equal to or greater than symbols were ignored. This results in a log mean lower than it should be, since the next detectable limit would be roughly one order of magnitude higher than the level used in the computations.

Calculations of bacterial concentrations included prefiltered samples, which biased the mean downward.

An indication of the effects of calculating the log mean bacterial concentration noted previously is the odd order of log means, which indi-

cates total enterococci, fecal enterococci, total coliforms, fecal coliforms. The enterococci mean was approximately one order of magnitude more than the coliform means.

Water Quality. The Master Plan contains some conflicting data and omits some necessary information concerning water quality. Dates for water quality sampling on Figure 3-3 do not correspond to dates provided in Tables 3-40 and 3-41 (Vol. IV). Methods of analysis for nutrients were not reported.

In addition, the nutrient analyses that were done were atypical. Total phosphorus and total (or Kjeldahl) nitrogen are needed for calculating proper N:P ratios.

Turbidity was reported as "standard turbidity units." Either Jackson or nephelometry units are normally used. This leads to confusion over what units were actually used.

A complete annual record at monthly intervals or less is needed for dissolved oxygen.

Plankton. Data collected before October for phytoplankton are not representative since sampling was done with a plankton net. This data cannot be compared with later data. The phytoplankton community was undoubtedly highly underestimated during the August-September period due to use of a plankton net with a 350-micron mesh size. Ideally a smaller mesh should have been used.

Changes in the net tow method used prevent clear interpretation of zooplankton tow data.

Benthos. A sled (or drag) is not the instrument normally used in marine pollution studies. This method does not collect many of the infaunal organisms usually measured to assess marine pollution. A grab is needed for infaunal measurement, and the organisms are collected in a sieve 1 mm square or less in size. WWCG therefore cannot relate any of the data collected to the worldwide body of literature and knowledge on marine pollution studies. A single sample in the outfall diffuser area is completely inadequate to characterize existing conditions. More sediment cores, with a completely different set of variables measured (organic carbon, grain size, heavy metals, sulfide, etc.), are needed.

Fisheries Data. While the market survey was a good starting approach, more information is needed on life history habits and food habits to determine which fish species are most sensitive to pollution bioaccumulation and infaunal changes. In her review of the fisheries data presented in the 1978 Master Plan, Dr. Bishara concludes that sewage

discharges since 1950 (especially in the absence of the Nile flood since 1964) have resulted in changes in the composition of the commercial fish catch. Also, the relative increase in Sardinella catch in the Eastern Harbor area may be due to overfishing as the fish tend to be undersized. The absence of Mugil is of concern and sewage discharges may be one of the causes. According to Hanafy (1979), the coastal waters of Alexandria contain high concentrations of hydrogen sulfide, which may render these areas unsuitable as nursery grounds for a number of fish species.

Some scientific collection, coupled with tissue analysis for pollutant loads, is needed to establish predischarge conditions. Such information will be invaluable in correcting any losses in fisheries or health hazards in fish tissues.

Physical Oceanography

Studies undertaken in the work associated with the 1978 Master Plan included current studies, tide studies, wave height studies, wind studies, and density studies.

Currents. Currents off Alexandria were measured with Endeco current meters and drogues. Analysis of the current meter data suggests that the principal current directions were between 40 and 70 degrees, and between 230 and 260 degrees; that is, northeasterly parallel to the coast or southwesterly parallel to the coast. Except in late September and early October, speeds in the 40- to 70-degree direction were usually greater than speeds in the 230- and 260-degree direction. The nearshore current meters exhibited essentially no onshore or offshore current, except for very infrequent onshore currents at the lower meters. Also, speeds were generally greater in the upper meters than in the corresponding lower meters. The 20-, 50-, and 80-percentile speeds were analyzed for use in determining dilution.

Drogues were released at various sites and various depths off Alexandria. In general, the deeper drogues, which were less affected by wind action, traveled roughly parallel to the coast, corresponding to the predominant current directions shown from the current meter data. Drogues near the surface, however, traveled not only in paths parallel to the shore but in other directions as well.

Tides. Local tide data at Alexandria are determined relative to the zero-centimetre reference mark on the Alexandria Harbor tide gage. Mean sea level is approximately 44.1 cm above the reference line. The average annual variation in water level is approximately 40 cm, the average monthly variation is about 20 cm.

The main factors that affect sea level variations along the Mediterranean coast of Egypt include atmospheric pressure, wind, and steric (seasonal temperature and salinity variations) effects. Sea level at Alexandria tends to rise higher than normal with lower atmospheric pressures, north-northwesterly winds, and higher water temperatures. The data suggests a 10-year maximum of 97 cm, a 50-year maximum of 105 cm, and a 100-year maximum of 108 cm above the zero reference mark of the Alexandria Harbor tide gage.

Waves. Wave heights were not measured but were estimated using the Sverdrup-Munk-Bretschneider (S-M-B) method as described in the Shore Protection Manual. The estimate of the significant wave height was based on the frequency distribution of onshore wind speeds from November through April. The significant wave heights appear to converge toward an upper limit of 4.5 m for periods approaching 8.5 seconds. Comparison of these values with studies done for local gas field operations suggested a slight increase in the significant wave height was warranted. Consequently, the significant wave height was adopted to be 5 m and the significant wave period to be 8.5 seconds.

Winds. Long-term wind data (1956-1967) suggest that the dominant wind direction is onshore from the north to northwest with speeds in the range from 2.5 to 7.5 m/s (5-15 knots). Winds blow more strongly onshore in summer and autumn, and exhibit more variability in the winter and spring.

Short-term wind measurements taken from July 1977 to June 1978 show essentially the same pattern as described above; however, a greater percent of light (2-3 m/s) offshore winds occurred during this period, especially from November to May. These short-term wind data also suggest that night wind speeds are about 10 percent less than wind speeds during the day. However, no appreciable changes in wind direction occur between day and night.

Density Studies. Onsite salinity and temperature measurements were made with a Hydrolab Model TC-2 conductivity-temperature meter and a Beckman RS5-3 portable salinometer. Temperature and conductivity measurements were used to determine salinity. Salinity and temperature data were obtained primarily to determine seawater density and annual patterns in horizontal and vertical density gradients.

In summer, a pycnocline (strong density gradient) exists in the upper layers out to 5 km offshore and occasionally beyond. This pycnocline can be attributed to the occurrence of a thermocline and the presence of less saline surface waters. The decrease in salinity in the top 5 to 10 m is attributed to fresh water discharges from the Mex Canal and wastewater overflow points spreading out over the ocean surface.

Occasionally, however, salinities may be greater at the surface than below it. This is attributed to high rates of evaporation at the surface especially during the summer. The depth of the pycnocline changes with the seasons, moving upward during autumnal cooling and downward as the effect of surface heating penetrates the water column during spring and summer. In late summer, the pycnocline has been observed as deep as 40 m. At other times of the year there is a weak, nearly lineal, density gradient of zero to 0.3 sigma-t unit per 30 m below a 10-m depth. In general, areas within 5 km of the shore precede the areas offshore both in autumn cooling and spring warming.

Evaluation of Physical Oceanographic Studies

The physical oceanography studies undertaken were generally sufficient for conceptual design. For final design, however, additional data will be needed to supplement that already gathered. WWCG's comments on these studies are discussed below.

Current. WWCG has no comment on the quality of current meter and drogue data taken. Ideally, continuous current meter data for at least 1 year at all the current meter sites would be preferred; however, mechanical, logistical, and weather conditions may have dictated otherwise.

The surface drifter studies conducted by Gerges (1978) provide useful data on nearshore and offshore current patterns for 1977, which basically substantiate the results of the 1978 Master Plan. However, this data does reveal apparent differences in the movement patterns of coastal (<6 km) and offshore (10-20 km) water masses.

For worst-case calculations of dilution, EPA has recommended that current speeds in the lowest 10 percentile be used, not 20-percentile speeds. (The EPA regulation came out after publication of the Alexandria report).

WWCG has replotted the data to obtain the 10-percentile current speeds of 0.02 to 0.03 m/s. The worst-case 90 percentile based on nearshore upper current meter averages 0.15 m/s with an extreme value of 0.35 m/s for the Kait Bey alignment in February.

Tides. Although the tide data presented is of short duration, the data are adequate for preliminary design purposes. For more detailed design, typical tide records should be examined to determine the daily variation in water level. An attempt should be made to determine what percent each factor (steric, wind, atmospheric pressure) contributes to water level variations.

Waves. As a check on the design wave, the S-M-B method was also used but with slightly different input parameters. Waves are generated by winds blowing over the eastern Mediterranean Sea. The width of the Mediterranean Sea north of Alexandria is approximately 560 km. A wave-generating storm would probably blow as a gyre around the center of the eastern Mediterranean basin; thus the fetch contributing to the generation of waves off Alexandria would be approximately one-half the width of the Mediterranean or 280 km. Given a fetch of 280 km, 14 hours would be required to develop the greatest wave action. The 14 hours represent 0.16 percent of a year. The wind speed associated with 0.16 percent of the time during the winter months is 19 m/s. The S-M-B graph estimates a significant wave height of 5.1 m and a significant wave period of 9.0 seconds. These values are only slightly greater than the estimates presented in Vol. IV of the Master Plan.

Winds. The wind data collected and its analysis appears to be sufficient for the conceptual design of outfalls at Alexandria, Egypt.

Wind will create a surface current due to a momentum transfer across the air-sea interface. The speed of the surface current is about 3.6 percent of the wind speed measured 10 m above the sea surface. The thickness of the surface current is assumed to be no greater than the significant wave height generated by the wind (Ekman's Theory and empirical tests suggest a reduction of current speed with depth).

Density Studies. The instruments used indicate trends only within a general order of magnitude for temperature, salinity and thus, density. The temperature sensor is probably accurate to within 0.1 to 0.2 degrees C. However, the salinity is probably only accurate to 0.5 part per thousand. This yields a density accuracy of ± 0.6 sigma-t unit. These accuracies are considered sufficient for conceptual design of outfalls but are not the order of accuracy desired for final design.

A more thorough discussion of density gradients would have been helpful. Density profiles indicate that surface density varies between approximately 21 and 29 sigma-t units. Below 10 m, the variation is between 25 and 29 sigma-t units. Both outfall sites exhibit a relatively well mixed water column beginning in October with densities in the range of 26 to 26.5 sigma-t units and remaining homogeneous but increasing in density through the winter and ending with densities of 28 to 28.5 sigma-t units in March. In late March or early April, stratification of the water column commences and intensifies through the summer months. The water column appears to reach a maximum stratification in late August and/or early September. Also during the summer months, the overall density of the water column decreases.

The pycnocline is rarely deeper than 10 m. The wind has a great influence on the existence of a nearshore pycnocline. During an onshore

wind, the pycnocline is strongly developed. This suggests that onshore-wind-generated currents tend to hold the fresh water discharges together and close to shore. Conversely, with an offshore wind the pycnocline is either weaker or nonexistent. This suggests that fresh water discharges are dispersed laterally and/or mixed within the water column.

Effects of Nutrients on Ocean Productivity

A potential benefit from discharging nutrients into the ocean was claimed in the 1978 Master Plan. The report estimated that the annual nutrient load from the wastewater would provide 10 percent of the nutrient load from the past Nile floods and would therefore help replace some of the nutrients lost after completion of the Aswan High Dam.

Unfortunately, no real benefit from the wastewater nutrient load can be expected. The Nile floods discharged a large quantity of nutrients into the sea over a short period of time, enhancing algae blooms and leading to subsequent fish production. In comparison, wastewater would be discharged throughout the year, and the nutrients it contains would be continually diluted by large volumes of water. Any algae stimulated by these nutrients would also be diluted by water low in algae content. The result would be that any enhanced algae production that occurs would be too diluted to be converted into fish production.

PROPOSED WATER QUALITY STANDARDS

Rationale

Two important aspects must be considered when considering pollution of any body of water. The first is the nature and resilience of the receiving water; the second is the nature and persistence of the pollutants.

For the Mediterranean, water quality standards should be derived to protect both the sea in total and local areas of discharge. Murdoch and Onuf (1974) and Onuf and Murdoch (1974) provide a rationale that the whole Mediterranean Sea is not likely to be impacted from localized sources of pollution, such as domestic waste. This is due mainly to the sea's large size, relatively great depth, generally narrow continental shelf, well oxygenated deep water, low productivity, and turnover rate of approximately 80 years. Therefore, standards to protect the larger system are not needed.

Locally, coastal areas can be severely impacted by pollution, and water quality standards must be set to protect these areas. Due to the

generally narrow continental shelf, small areas are relatively more valuable and, except for shipping, provide the main beneficial use of the Mediterranean.

The other important aspect is the type of the pollutant. Persistent materials, particularly those that can accumulate through the food chain, are a more serious pollutant threat than degradable substances. Irreparable damage can result, and accumulation can directly affect human health, so stronger restrictions are needed. The impact of degradable pollutants can be reversed and is more restricted temporally and spatially. Therefore, more relaxed standards can be applied to these substances.

The following proposed water quality standards are set within the framework provided in Principles for Developing Coastal Water Quality Criteria (IMCO/FAO/UNESCO/WMO/WHO/IAEA/UN GESAMP 1976). Specific measures are based upon the MED POL Protocol for Prevention of Pollution of the Mediterranean by Pumping from Ships and Aircraft (1978), U.S. EPA Quality Criteria for Water 1976a), and procedures for impact assessment in the U.S. EPA Application for Modification of Requirements of Secondary Treatment (1976b) (40 C.F.R. Part 233, Subpart B, and Technical Support Document, 1979). The proposed water quality standards are formulated to protect public health and beneficial uses.

Public Health

Bacterial Pathogens. Fecal coliforms are not to exceed 100 MPN/100 ml in swimming areas. The lower basic criterion adopted in the 1978 criteria will be retained. A cause-effect relationship of this criterion and a disease rate of 1.2 percent (0.4 percent higher than the "background") has been derived for Alexandria. Given the normal variability in an epidemiological relationship, a 0.4-percent unit increase would not likely be detectable. In addition, such a standard is supportable by the Bilthoven Conference (1974) and is in the lower range adopted by numerous countries.

Due to the fact that treatment plant upsets and chlorination malfunctions are bound to occur, a risk analysis of the expected frequency and duration of system malfunctions should be combined with a more detailed and accurate study of bacterial decay, sedimentation, dilution, and onshore transport to estimate the risk of beach contamination due to treatment system failures.

Unchlorinated primary effluent would have bacterial concentrations on the order of 10^8/100 ml; raw sewage in Alexandria has about 10^{10}/100 ml. While the 1978 Master Plan estimates that a 10^9 concentration would be

reduced sufficiently by the time it reached shore, a more detailed analysis is advisable before final design.

Public health aspects related to toxicants in fish as a food source are considered in the following section, Beneficial Uses of Effluent. No shellfish fisheries have been included since none were reported in Alexandria.

Beneficial Uses of Effluent

Biostimulation. Standards to protect beneficial uses are established primarily to protect existing fisheries. No standard has been adopted with regard to light penetration or biostimulation. This is because any change in light penetration 10 km from shore will only be visible to fishermen, not to tourists. Therefore, no significant loss of beneficial use would result.

Visual biostimulation of open coastal waters, which might result from an input of nitrogen and phosphorus particularly with currents as fast as those found off Alexandria, also presents no major problem. The dispersion capabilities of open ocean areas are generally sufficient to disperse phytoplankton blooms before they can accumulate to noticeable levels. Additionally, blooms of smaller sizes would not be detrimental, and like light penetration, would only be observable to fishermen. Therefore, no water quality standards for nitrogen and phosphorus are proposed.

Dissolved Oxygen. Concentrations of DO are not to be reduced more than 10 percent below ambient levels beyond the zone of initial dilution (ZID).

As stated earlier, the 1978 criteria value of 2.0 mg/l DO within the ZID would definitely result in a large reduction of oxygen beyond the ZID. Additionally, the 2.0 mg/l is not sufficient to protect marine life. The EPA standard of 5.0 mg/l is impractical due to natural levels falling below this amount. A relative depression standard is therefore appropriate.

Suspended Solids. Normally, unless present in gross amounts, suspended solids primarily affect water clarity. As explained earlier, water clarity effects are not proposed as important criteria for Alexandria since no loss in beneficial uses would result. More important is the effect of sediment accumulation on the sea bed. An approximate estimate for the acceptable rate for sea bed sedimentation can be obtained from Word (1978), Bascom, et al. (1978), and McIntyre (1977). McIntyre found a level of 40 mg/cm^2/day (about 6 times background) to result in increased production without species diversity loss, but 117 mg/cm^2/day

(about 55 times background) results in a community not favorable to fish. Only "changed" conditions (modified community structure and slightly increased biomass) occurred off San Diego, with calculated sedimentation rates of 0.07 mg/cm^2/day (Herring and Abati 1978, Word 1978, Bascom, et al., 1978) to 0.10 mg/cm^2/day, depending upon the method of calculation. This method is very dependent upon the calculation of sedimentation rate, which has several assumptions. Sediments with high organic content can tolerate less loading of new organic material than inorganic sediment.

Table L-3 shows ranges of sediment changes that have occurred near southern California outfalls, based upon BOD_5 and percent volatile solids content. No loss in the benthic suitability to fish occurred until the area was "degraded." In general, changes in benthic food production on an areal basis of less than 10 percent would not be detectable in marine systems. The following standard is therefore proposed: Sediment accumulations are not to exceed normal rates of accumulation by a factor of 10 in any more than 5 percent of the fisheries grounds off Alexandria, Egypt.

Toxicants. In order to protect the fisheries and ultimately human health, toxicants must be restricted from entering biotic communities. While toxicity is very difficult to predict, receiving water toxicant standards have been established after considerable scientific review. The basic rationale for accepting these standards in the United States also applies to Egypt. Proposed toxicant standards are listed in Table L-4. These reflect receiving water limits set by the EPA. Where EPA has suggested a limit based upon onsite bioassay, California daily maximum limits have been adopted. These limits are somewhat more liberal than California daily average limits, but are in closer accord with EPA limits.

Table L-5 summarizes the water quality standards proposed by WWCG and used in the analysis of potential impacts of discharging chlorinated primary effluent to the sea through two outfalls.

Compliance with Proposed Water Quality Standards

The probability of the effluent meeting the proposed WWCG standards assumes the discharge of wastewater that has received primary treatment and chlorination. Necessary source control or elimination of industrial effluent toxicants is also assumed. The effluent is projected to have a BOD of 410 mg/l, suspended solids of 200 mg/l, fecal coliform count of 10,000/100 ml, ammonia nitrogen of 27 mg/l, and phosphorus of 8 mg/l.

Additionally the analysis is based on design peak discharges of 10.36 m^3/s and 7.00 m^3/s for the Sidi Bishr and Kait Bey outfalls, respectively. EPA recommends using the lowest 10-percentile current speed for calculating initial dilution. Off Alexandria, the lowest 10-percentile speeds were 0.02 to 0.03 m/s. For the calculation of initial dilution, WWCG applied both the Roberts model and the EPA simplified model. For the calculation of secondary dilution, the Brooks model was used.

Results of the WWCG worst-case analysis suggest that dilutions are strongly dependent on directional flow and that the 1978 Master Plan estimates were rather liberal for initial dilutions and conservative for secondary dilutions. The overall conclusion is that the estimates for obtaining a dilution of at least 1000 at the beaches of Alexandria are easily met. Longshore currents should provide minimum initial dilutions on the order of 350 and secondary dilutions sufficient to provide for a total dilution of 1000 within 1 km of the outfall discharges. Onshore currents, however, provide an initial dilution of only about 75, but a secondary dilution at the beach in excess of 60. This would provide a total dilution of 4500 at the beaches.

Public Health

Using an assumed effluent bacterial concentration of 10,000 MPN/100 ml, an initial dilution of 350 (during long shore currents) would reduce the concentration to 29 MPN/100 ml. This is below the proposed standard (100 MPN FC/100 ml) at the beach; however, an initial dilution of 75 (during onshore currents) would reduce the concentration only to 133 MPN/100 Ml. Far-field dilution benefits (estimated to be in excess of 60-fold) would therefore have to be relied upon to meet the proposed standard.

In the event of a prolonged treatment plant malfunction, resulting in discharges of raw or unchlorinated sewage, a significant probability exists that discharges at both outfalls would violate the proposed fecal coliform standard.

Beneficial Uses

Dissolved Oxygen. Dissolved oxygen impacts are calculated by two methods. The first assumes an immediate dissolved oxygen demand (IDOD) of two percent of the BOD. This relationship is based upon the BOD:IDOD ratio found in San Diego, San Francisco, and Oxnard, California, and is biased upward by exclusion of data where the IDOD is equal to 0. This is therefore a conservative, worst-case approach.

For Alexandria, using a BOD of 410 mg/l, an IDOD of 8.2 mg/l would result. A dilution of 75 would leave an IDOD of 0.11 mg/l, which is too

low to have any measurable effect after initial dilution, and meets the criteria of less than 10 percent reduction.

Due to possible error in the BOD:IDOD ratio, DO impacts were also calculated directly for BOD. This was done using the equation:

$$BOD_x = BOD \frac{(1-10^{-kx})}{(1-10^{-k5})}$$

with K = .17 day^{-1}, x = 1 hour, and the BOD at the end of initial dilution = 5.5 mg/l. (Use of this equation for x = 1 hour is also questionable, but will suffice as a rough estimator.) The resulting BOD would be 0.13 mg/l, an insignificant amount.

Suspended Solids. The accumulation rate of suspended solids was calculated using the methods of CH2M HILL, 1979, Application for Modification of the Requirements of Secondary Treatment, City of San Diego. This method was shown to agree well with both the Herring and Abati method, and with the benthic oxygen demand of marine sediments, which reflects accumulation rates. This method uses the following assumptions:

- V current = average net velocity, 4.0 cm/sec, perpendicular to the diffuser
- The following solids settling distribution:

Settling Velocity (cm/sec)	Settling Distribution Percentile
1.0	100 %
0.3	95
0.1	83
0.05	70
0.01	32
0.005	19

- Effluent suspended solids = 200 mg/l
- Settleable fraction of total suspended solids = 50 percent
- Percent carbon of settleable solids = 35 percent (McIntyre, 1977).

For the Kait Bey Outfall:

Diffuser length = 450 m

Zone of initial dilution (ZID) dimensions = 450 m x 45 m

Depth = 40 m with a surfacing plume

For the Sidi Bishr Outfall:

Diffuser length = 400 m

ZID dimension = 400 m x 53 m

Depth = 55 m

Plume height = 40 m (summer)
55 m (winter)

The following equations were used in calculating the solids deposition rates and areas:

- Settled solids (kg/day) = Total suspended solids (kg/day) x 0.5 x (settling distribution percentile)
- Area of deposition (m^2) = (settling time (sec) x current speed (cm/sec) x 2 x ZID length + ½ ZID width)
- Areal deposition rate = Settled solids/area of deposition

The deposition rates of solids and carbon and the deposition areas calculated are shown in Table L-6.

Productivity for the eastern Mediterranean basin, expressed in terms of carbon (C), is in the range of 30-70 gm C/m^2/year (Murdoch and Onuf, 1974). Assuming approximately 30 percent of this reaches the bottom (see McIntyre 1977), natural sedimentation rates of 9-21 gm C/m^2/year (.007 - .017 mg/cm^2/day) would be expected.

Background sedimentation rates could in fact be much higher. Primary productivity rates for the offshore region of Alexandria could be on the order of 120 gm C/m^2/year, according to Dr. Wahby.

Therefore, based upon a criterion of 10 x natural levels, rates of sedimentation in excess of .07 - .17 mg/m^2/day (average = .12 mg/cm^2/day) might reasonably be assumed to cause degradation. As can be seen from Table L-6, this rate is exceeded in a total area of the two outfalls of 28 to 33 km^2. If the fishing grounds are assumed to extend from Rashid to 50 km west of Alexandria, and are 100 m in depth, the fisheries area would be approximately 2,300 km^2. Five percent of this is 117 km^2,

which is more than the areas receiving high levels of sediments from the sewage. The areas receiving high levels represent approximately 1 percent of this area. According to the proposed standards, this is an acceptable level.

HYDRAULIC REVIEW

The outfall system must operate satisfactorily under all flows and sea conditions to reduce the chance of hydraulic failure, which could cause overflows across the beach. The hydraulics of the proposed outfalls were reviewed primarily to determine the hydraulic head required at the effluent pump station and the pressure in the outfall pipes. The minimum and maximum velocities were computed. We concur with the recommended Mannings friction factor of N=0.016.

A minimum velocity of 0.3 to 0.6 m/s must be maintained to move the effluent along the pipe without deposition of solids. The maximum velocity should be about 2 m/s. This velocity will flush out the deposition of sediments, while keeping the energy for pumping at a minimum. The energy requirements to transport the effluent along the outfall increase rapidly at the higher velocities, i.e., over approximately 2 m/s.

The pressure heads must be compatible with the pipe materials available for construction. The velocities and pressures in the proposed outfalls are much higher than normally found in ocean outfalls. The pressures at maximum flows are estimated to be over 58 m at the Kait Bey outfall and 41 m at the Sidi Bishr outfall. These pressures can be reduced to less than half by increasing the diameters of the pipes at Kait Bey from 1700 mm to 2130 mm and at Sidi Bishr from 2200 mm to 2600 mm. With these diameters, it is possible to maintain the minimum velocities during the initial years of operation and keep the maximum velocities less than 2 m/s for future peak flows.

The outfall pipe should be on a progressively downward slope from the pump station. High spots in the outfall line provide a place for air pockets to form, which may reduce the capacity of the pipe and trap hydrogen sulfide. The pump station should discharge near or below sea level so the pipe is always full of water. This will eliminate the need for an expensive headworks structure at the shoreline, to release air and dissipate excess energy. Hydrogen sulfide problems will need to be studied during final design. The larger diameter pipe requires more time for the effluent to move through the outfall. However, without air pockets and with primary treatment, the hydrogen sulfide problem should not be serious.

Manholes and removable endgates should be provided for maintenance.

CONSTRUCTION MATERIALS

Pipe

Using the outfall pipe diameters and the resulting pressure proposed in the Master Plan, only concrete-coated-and-lined steel pipes or concrete cylinder pipe would be satisfactory. Both of these will require thick concrete coatings for stability during and after construction. Fiberglass or plastic materials are too light for construction in the open sea.

By increasing the diameters to 2600 mm and 2130 mm, for Sidi Bishr and Kait Bey, respectively, the head can be reduced to about 21 m. This allows the outfall to be constructed of reinforced concrete pipes. This type of pipe has been commonly used for all large diameter outfall pipes in the past 20 years. The pipe is frequently manufactured in 4- to 5-m lengths, which weigh up to 18 and 26 tonnes, respectively. It should be possible to manufacture this pipe in Egypt, in a special casting yard using local cement and concrete aggregate. Quality control procedures would have to be strictly enforced to produce satisfactory pipe. With proper equipment, two or more lengths of pipe can be placed at one time.

Bell-and-spigot joints with single or double rubber "O" ring gaskets have been used in recent years on outfalls. A single "O" ring joint is usually more flexible than the double "O" ring joint and appears to be more compatible with the seabed conditions of alternating rock and soft soil. An outfall with a single "O" ring joint is generally easier to install, requires less time, and is considerably cheaper to build than one with a double "O" ring. The purpose of the double "O" ring is to allow pressure to be tested between the gaskets before the pipe is back-filled. Outfalls with single "O" ring gaskets can be tested in sections if necessary. Thrust ties across the joints should be considered during final design.

Armor Blocks

In depths less than 25 m, the pipe will be covered with medium size armor blocks (about 500 mm) to protect it from wave current and forces. Since suitable natural stone is not available in the Alexandria area, the protection will probably be manufactured from concrete. The shape of the manufactured armor may be in a cube or one of the several patented shapes presently used for port work in areas where natural stone is not available. Near shore, in areas of breaking waves, large armor blocks will be used to protect the pipe.

Bedding Stone and Cover Stone

Bedding stone of 75 to 100 mm minus gradation would be used to provide a smooth bed under the pipe and for backfill under the pipe. Cover stone of approximately 200 mm minus is reported to be available in Alexandria area, but it is expensive compared with concrete aggregate. This material will be used to cover the pipe in deeper water (below about 25 m). In shallower water, it will be used for an intermediate layer between the bedding stone and the armor stone.

ENVIRONMENTAL CONSIDERATIONS

The discharge of adequately treated and chlorinated domestic sewage through properly designed and constructed outfalls should not cause major adverse impacts to the local marine environment, nor will it cause significant beneficial impacts in the form of increased productivity. Some adverse impact due to shifts in species will unavoidably occur in the immediate area adjacent to the diffusers due to the deposition of solids. The total area thus affected has been computed to be up to 33 km^2 based on a criterion of impact occurring when deposition is 10 times or greater than natural conditions. This impact will, however, be very slight for the majority of the area since up to six times the natural rate has been found to increase productivity without loss of species diversity.

The potential response of benthic organisims residing near outfalls based on their feeding habits is listed in Table L-7. Fisheries information collected for the 1978 Master Plan indicates that a large portion of the fishery is pelagic Sardinella. These should be completely unaffected by the outfalls. Demersal fish may be more directly influenced, but no significant loss of suitable demersal habitat is anticipated since the expected species change in benthic organisms does not represent a loss of food resource to the fish, and may represent a food subsidy.

Another impact will be a slight local increase in phytoplankton blooms, which may be visible at times to fisherman. These are not expected to be visible from shore or of significant size.

Adverse impacts due to bacterial pollution or oxygen demand are not expected to occur.

Failure to limit the toxicants listed in Table L-4 to the allowable effluent concentrations shown could result in significant adverse impacts to the marine environment and potentially to humans who consume the affected fish. This potential impact can only be mitigated through

rigorous enforcement of appropriate sewer use regulations and industrial pretreatment if required.

Failure to maintain adequate primary treatment and effective disinfection would degrade the sea bed through solids deposition, ZID oxygen depletion, and, at times, bacterial contamination of the bathing beaches. Mitigating measures should include redundant facilities, particularly in respect to electrical power and chlorination equipment.

Major rupture of the outfalls, by whatever cause, could cause similar adverse impacts until such rupture is repaired. Mitigating measures should include marking signs to warn ships, prohibition of large ship anchorage and appropriate changes to marine charts, pilots and the like.

OUTFALL CONSTRUCTION AND COST ESTIMATES

Estimating Assumptions

The design, material selection, plans and specifications for an outfall must all be prepared with a thorough understanding of sea conditions, and the construction experience, equipment, and abilities of the available contractors. For this conceptual level cost estimate, we have assumed the location and profiles shown on Figures L-2 and L-3 and the following construction methods. Since no profile is available at the new Sidi Bishr outfall location, we have assumed it to be similar to the original Sidi Bishr outfall profile. We expect the contractor to build the outfall from a trestle built through the surf zone to a depth beyond the reefs, where the outfall can be continued from barge-mounted equipment. This trestle may be about 800 m long at Kait Bey and Sidi Bishr. From the trestle, the excavation, bedding, pipe placement, and backfill can be done from a stable platform above the surf.

Access to these trestles from land will be difficult because of the lack of space and traffic congestion. At Kait Bey, we have assumed that the outfall contractor will have the pump station site available to him for access. At the Sidi Bishr site, the contractor may elect to barge some of the materials to the end of the trestle to avoid the traffic.

Excavated materials will be side-cast. Rock excavation will be done by a combination of methods. The softer rock will probably be loosened by a heavy steel chisel and excavated by a large heavy clam bucket. Thin cuts in harder rock will be loosened by shaped charges. Deeper excavation may be accomplished by underwater drilling and blasting, or another setting of shaped charges. Through the surf zone, the trench would be supported by steel sheet piling. Through the surf zone and out to a depth of about 25 m most of the outfall will probably be built in a rock trench.

Work would be done from barges beyond the trestle. Transition areas between rock and soil would be excavated about 1 m below grade and back-filled with bedding stone. In some areas, bedding stone fill will be required to support the pipe above the surrounding seabed. In other areas, rock or soil excavation is required.

After the pipe bed is prepared, the pipe would be lowered into place using a "horse." This piece of equipment would support the pipe above the seabed while the joint is made up. It would also insulate the pipe from the movement of the floating equipment, which would move with wind and waves. The pipes would be placed to grade on sandbags and the bedding stone tremied under the pipe to at least the centerline. Cover stone would be placed over the pipes, which would be covered with armor blocks. The trench and medium armor block would not be used above a 25-m depth; above this, the pipe will be covered first with bedding stone and then with cover stone.

For the estimate, we assumed the contractor would establish a waterfront staging area, in either the Abu Qir area or the Mex Dekheila area, for casting the pipe and armor blocks, storing materials, and loading barges.

The "construction window" during the year is important for the contractor. He must estimate the number of days that construction will be stopped or interrupted by bad weather. After discussing the construction window with local people, and checking the Mediterranean Pilot, Volume 6 and the wind conditions reported in Volume 4, Marine Studies, we found a wide range of opinion. For this estimate we have assumed that the contractor's equipment can work in seas or swells of about 2 m and can stand by on position in seas or swells of about 4 m. Winds exceeding force 3 will affect the work primarily on summer afternoons and during the stormy months of December, January, and February. Thus, we estimate that work can progress with two shifts per day for as few as 150 days per year. For estimating purposes, 240 days per year were assumed.

Capital Costs

Capital costs for the two sea outfalls were estimated by three different methods. The first and most definitive method was an independent estimate made by a separate company that specializes in outfall design and construction. This estimate was made by H. V. Anderson Engineers and is based on end-of-1980 prices, materials and construction methods as proposed, and a maximum "construction window" of 240 workable days per year. The estimate assumes simultaneous construction of the two outfalls through the use of separate crews.

The second method employed (used as a check on the independent estimate) was to update detailed cost curves and information gathered from CH2M HILL's involvement in the recent San Francisco Southwest Ocean Outfall Project. This resulted in the curves shown on Figure L-4. This independent estimate closely agreed with the first one.

As a final check, the cost estimates presented in the 1978 Master Plan were updated and adjusted to reflect changed lengths, sizes, and costs (year) bases. Results obtained by this checking method also compared favorably with the independent estimate.

As discussed, one of the greatest uncertanties involved in estimating the costs of such large outfall projects as these is the "construction window." Any reduction in the assumed "construction window" will have a significant upward impact on costs. For this reason, WWCG strongly recommends immediately undertaking work to collect and analyze wind and wave data if the sea disposal option is selected.

Egyptian and US marine experts expressed a wide range of opinion as to the probable length of the construction window. In any event, contractors who submit tenders for this highly specialized construction will make their own estimates and these will likely contain some margin of safety, i.e., a basis of less than maximum possible. This expectation can be accommodated through the inclusion of contingency amounts in higher proportion to base construction cost estimates than would be the case for on-shore based construction. Also adding to the need for higher off-shore construction contingencies is the inability of field investigations and design to fully define sea floor and sub-floor conditions to the extent surface and subsurface conditions are definable prior to on-shore construction. This results in more contractor claims for "changed conditions" on off-shore outfall work. As a result of these considerations, we have used a 40 percent contingency on those portions of the outfalls constructed in the sea, in contrast to the 25 percent contingency employed on dry land based construction.

Table L-8 presents capital cost estimates for both the Kait Bey and Sidi Bishr outfalls. The land portion of the Sidi Bishr outfall is shown separately. Effluent pump stations are not included. Those are estimated in Appendix M for Kait Bey and Appendix J for Sidi Bishr.

Operation and Maintenance Costs

Once a properly designed and installed outfall is put into operation, it must be inspected and maintained like any other marine and hydraulic structure. The outfall and diffuser pipe, armor blocks, and cover stone should be inspected for damage caused by currents, wave forces, dragging

anchors, or trawlers. The diffusers should be inspected to see that the ports are not plugged and are discharging properly. Any damage to the armor blocks, bedding stone, etc., will need to be repaired.

Every 5 years or so, the endgate should be removed and the outfall flushed out to remove loose accumulations of grease, sediment, etc. If a heavy buildup of sediment or grease remains attached to the pipe, then it may be necessary to "pig" the line. The "pig" is a large brushing or scraping tool, which is pushed through the outfall pipe with water pressure. Normally, outfalls require very little maintenance and seldom require pigging.

Anticipated operation and maintenance costs for the two outfalls are shown in Table L-9. These costs are exclusive of treatment, sludge management, chlorination, and pumping costs which are shown in Appendix J. The cost estimate assumes each outfall will be inspected annually by hard-hat divers and the end gates cleaned every 5 years. The estimate also includes an annualized allowance of LE 130,000 per year to cover costs that might be incurred from a natural disaster or collision with a shipping vessel. Such an accident is unlikely, but possible. The allowance provided is treated as a "sinking fund" paid annually although the cost, if incurred, would not be annual.

ADDITIONAL STUDIES NEEDED

Based on our review of the marine studies, we anticipate that, before final design, the following additonal information will be needed. This list should be reconsidered in detail and authorized as early as possible if the sea disposal alternative is favored.

- Wind and wave data to establish "construction windows"--One full year of continuous data is essential; longer records would be statistically beneficial. The data could be collected at sea by installing wind and wave instruments on a spar buoy or in the surf zone or instruments could be mounted on a braced staff. Sufficient data may be available from studies funded by the UNDP-UNESCO.

- Density profiles--Density profiles for summer conditions are definitely required and records for 1 complete year are preferable, using more precise instrumentation. An array of T-C sensors plus random density profiles as deemed necessary should be considered at each location.

- Current data--Continuous current information for at least one summer and preferably for 1 year is required. Two metered arrays supplemented by drogue studies at each outfall location should be considered.

- Dye studies--A series of three dye drops should be made over several days time during the summer to determine vertical and horizontal eddy diffusion rates.

- Profiles--The bottom should be resurveyed before design and again before construction to identify any significant changes.

- Geotechnical study--Geotechnical studies along the outfall route, with special studies in areas to be trenched, will be required. These studies will include shallow drilling (to 10 m below the mudline) and laboratory testing of samples.

- Receiving water quality--Water column data on nutrients, chlorophyl, turbidity, light transmission, temperature, salinity, pH, and dissolved oxygen should be collected for 1 year.

- Sediment samples--Sediment samples should be collected for particle size analysis, percent volatile solids, BOD, and organic carbon. Settling traps should be used to measure settling rates and particle size distributions.

- Bacterial reduction--Measurements should be made of coliform bacteria death rate over time under ambient conditions.

- Benthic animals--Benthic animals should be collected by divers or by grab samples. Collection should be a minimum of four times per year for at least 1 year and at several stations around each outfall site. Samples should be sieved through 1-mm screens and analyzed by accepted scientific methods.

- Fish--Fish should be collected in the area by otter trawls. Samples should be analyzed for stomach content, chlorinated hydrocarbons, and trace metals.

- Library research--Detailed library research should be done to obtain available data on oceanography, marine biology, and water quality in the Alexandria area. This search should include universities and various UN and Egyptian agencies.

- Provide revised study level layouts and cost estimates based on the results of the above studies.

ABSTRACTS OF INFORMATION USED

This section contains abstracts of the information sources used to analyze the sea disposal alternatives.

1. Aleem, A.A. Effect of River Outflow Management on Marine Life. Marine Biology 15:200-208. 1972.

Deals with the effect of the Aswan High Dam. Shows before dam Mediterranean shelf fisheries (28,750 km^2) provided 23 percent of total Egypt fish catch. Mentions that the western sector (Alexandria to Sallum) was little affected by flood waters, and that demersal fish catch and trawling is limited in that area. Flood did affect Alexandria's phosphate cycle. Midsummer levels before flooding were 0.025 g-at P/l, then rose to 0.3 g-at/l. Same was true with nitrates with an increase from 1 ug-at N/l before flood to 3.5 g-at N/l afterwards. Reported blooms, mostly diatoms off Alexandria were 9 to 10 million cells per litre. Leading species were Skeletonema costatum, Chaetoceros socialis, C. decipens, C. curvisetus, Hemiaulus sinensis, Cerataulina bergonii and Thalassionema nitozschiodes. Nile bloom faded in December with a minimum of a few thousands cells per litre in July and August. A zooplankton bloom co-occurred with an increase from 2,000-4,000 individuals/m^3 in summer to 50,000-80,000 orgs/m^3 during the bloom. Estimated demersal fish standing crop off the Nile is 1 ton/km^2. After closure of the dam, pelagic fisheries decreased.

2. Becacos-Kontos, T., and R.C. Dugdale. Pollution in Greek Waters. Marine Pollution Bulletin 2:158-160. 1971.

Found high concentrations of phosphate, silicate and nitrate around the new sewage outfall at Keratsini in the Saronicos Gulf, with effects detectable for 20 km. Cites low Mediterranean nutrient background as factor in easy detection. Area is not open coastal and no current data is provided.

3. Ben-Tuvia, A. Revised List of the Mediterranean Fishes of Israel. Israel Journal of Zoology. 20:1-39. 1971.

Gives taxonomic revisions and new species. Points out problem of warmer temperatures accounting for only 284 species in eastern Mediterranean with a total of 540 species for the total system. Brief notes on catch methods, depth, and life habit are included with some species.

4. Ben-Tuvia, A. Man-Made Changes in the Eastern Mediterranean Sea and their Effects on the Fisheries Resources. Marine Biology 19:197-203. 1973.

Major influences are construction of the Suez canal in 1896, which allowed 30 species of Red Sea fishes to enter the Mediterranean, and construction of the Aswan High Dam in 1964. Lower winter tem-

peratures in the Mediterranean is the main deterrent to establishment of Red Sea migrants, as are food resources and the canal. Aswan High Dam affected Egytian sardinella fisheries but not Israel's, although average size of fish has decreased.

5. Camp, Dresser, and McKee, et. al. Alexandria Wastewater Master Plan Study. Volume IV, Marine Studies. 1978.

   Physical current data that is provided is sufficient for marine ecology determinations. Density profiles (temperature and salinity) are provided which is sufficient, but no other water quality data is sufficient. Sediment sample analysis is insufficient in the variables measured, number of samples taken, and periods when samples were taken to provide quantitative information needed in a biological assessment. The bacterial decay study was not performed correctly, making its use very limited. Benthic fauna collections provide only qualitative data and are insufficient to provide quantitative assessments, annual cycle information, or to be of use in monitoring impacts. Fish market surveys were used and provided a partial basis for fish community characterization, but project- and site-specific sampling is needed. More details are provided in Section I and II.

6. Camp Dresser and McKee et. al. Initial Environmental Impact Statement. Volume II, Final Technical Report. 1978.

   General description of marine ecology of the Mediterranean is adequate, but specific information for more localized coastal nearshore areas of Alexandria is inadequate. This report mentions studies underway by the University of Alexandria at the time of writing. No specific data on nutrients, water clarity, chlorophyll A, marine benthic fauna, zooplankton, phytoplankton and fish, which can be used to derive dynamic interactions between the components is available over an annual cycle.

7. El-Din, S.M. Sharaf, et. al. Hydrographic Structure and Circulation Patterns as a Measure for the Extent of Pollution Along Alexandria Coast. C.I.E.S.M. Work-shop on Pollution of the Mediterranean. 1978.

   Mentions study on pollutants (domestic sewage, industrial wastes and oil) on biota. This eport only has hydrographic data. Mentions collection of temperature salinity, dissolved oxygen, currents and other parameters nearly every month from May 1975 to December 1976. Mentions reduced salinity of surface lens near shore. Mentions generally adequate DO levels and transparency. Gives currents based on drogues. Useful paper, with lots of other data mentioned but not included.

8. El-Hehyawi, M. Lofti. New Data on the Distribution of Pollutants and Their Effects on Some Hyponeulston Constituents in the S.E. Mediterranean. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Collected tar balls along Alexandria Coast. Also collected water, zooplankton and polluted sediment samples. Water samples were analyzed for extractable hydrocarbons zooplankton and sediment were analyzed for As, Cd, Hg and Pb. Cite anoxic conditions in Abu Qir Bay and that animal communities are "affected." Collected fish eggs and says area from El-Sallum to the Nile Delta is a spawning ground for Cupeidae, Serranidae, Carangidae, Mullidae, Triglidae, Engraulidae and others. Polluted water of Abu Qir Bay had fewer fish eggs.

9. El-Sebae, A.H., and M. Abo-elamayem. A Survey to Determine Potential Pollution of the Mediterranean by Pesticides from the Egyptian Region. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Data on four samples of freshwater in Alexandria for lindane, heptachlor, p-p-DDE and o.p.-DDT is provided and indicates a potential problem due to accumulation of these pesticides in food chains. Concentrations of pesticides could be used as rough estimates of inputs via outfall.

10. El-Sharkawi, F. Coastal Water Quality Control. Study of the State of Pollution of Alexandria Beaches due to Sewage Discharge. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Mentions discharge of 200,000 m^3/day from present Kait Bey outfall, and 18 other minor outfalls. Presents average total coliform, streptococci, and percent fecal coliform for water samples taken from June through August along coastal area. Most show violation of proposed 100 FC/100 ml standard. Describes each outfall. Mentions study being carried out by High Institute of Public Health, University of Alexandria. Suggests immediate closure of beaches to protect public health.

11. Fowler, S.W., et al. The Role of Zooplankton Fecal Pellets in Transporting PCBs from the Upper Mixed Layer to the Benthos. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Provides background data on one input of PCB's.

12. Gerges, M.A. Trajectories and Speeds of Surface Currents Near the Egyptian Mediterranean Coast as Deduced from the Movements of

Surface Drifters. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Reports on results of 2-year (1976-1977) release of 2664 . .dhead surface drifters from El-Mex to Rosetta at 57 stations. Provides trajectories from 1977 data for each month. Very useful.

13. Groupe EPOPEM. Impact de L'Effluent Ubain de Cortiou sur les Populations Phytoplanetonique Heritigues. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Based on a spring and autumn cruise, cites "perturbed" phytoplankton (especially Cyanophyceae and Eugleno-phyceae) within 2 km of outfall but, farther away, system "even seems normal". Also states no phytoplankton bloom seen, just change in species composition. Concerns the Marseille outfall. Cites lower plankton numbers very near outfall due to "dilution" (shows very low initial dilution).

14. Groupe EPOPEM. Effet Global de la Pollution d'un Emissaire Urbain (Marseille-Cortiou) sur les Populations Zooplanetonique d'une Zone Neritique. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Cites 4 zones: 1) Very polluted near outfall (500 m, Secchi = 2 m; 10,000 mg C/l; 500-700 individuals/m^3); 2) near outfall (2 m) with 6 m visibility, 1000 mg C/l, and high zooplankton (3000 individuals/m^3); 3) intermediate 14-15m visibility 100 mg C/l, 800 individuals/m^3; and, 4) normal area of 15 m visibility, 400 mg C/l. Shows effects of outfall with very little initial dilution.

15. Mujeriego, R., et al. Marine Water Pollution Control. Recreational Coastal Water Quality Effects on Public Health. C.I.E.S.M. Workshop on Mediterranean Pollution. 1978.

Found total coliform MPN of 10-800 MPN/100 ml related with morbidity rates of 50 percent, and 550-3000 MPN/100 ml related to 80 percent. Morbidity rates for skin, ear, eye and nose infections.

16. Murdoch, W.W. and C.P. Onuf. The Mediterranean as a System Part I-Large Ecosystems. International Journal of Environmental Studies. 5:275-284. 1974.

A general paper on the Mediterranean as a whole system. Gives overall circulation, topography, and oceanographic data. Included are ranges of primary productivities of 30-70 gm C/m^2/year in the eastern basin. Puts forth argument that total system can withstand pollution and total system is not under threat.

17. Onuf, C.P. and W.W. Murdoch. The Mediterranean as a System Part II - Small Ecosystems. International Journal of Environmental Studies 6:29-34. 1974.

General paper with a good overview of the coastal, river and multiple-use aspects of the Mediterranean. Gives rationale that since the coastal waters are unproductive, communicate freely with the open sea and have small continental shelf, they can take a considerable load of pollutants. Gives tourism, fisheries, industrial development, mineral extraction, and shipping as major multiple uses.

18. Stirn, J. Modifications of Some Mediterranean Communities Due to Marine Pollution. Thalassia Jugoslavica 7(1):401-413. 1971.

Used modern methods (10-cm cores by divers, dredges) for macro- and meiobenthos. Study areas were Gulf of Koper, N. Adriatic, and Lake of Tunis, So. Mediterranean. Found Ulva lactuca, Enteromorpha intestinalis, nematodes, oligochaetes (Peloscolex,) polychaetes (Capitella capitata, Eteone picta and Scololepis fuliginosa) and copepods (Harpateticus littoralis) in Gulf of Koper and Ulva lactuca, Enteromorpha intestinalis, Hyrdrobia acuta (snail), nematodes, amphipod (Corophium acuta) polychaetes (Capitella capitata), Podarka pollida and harpachoid Tisbe furcata) only parethisis on one area serpulid worm Spirorbis corrugatus in Lake of Tunis, both on soft bottoms. Hard areas had Merceriella enigmatica, Hydroides unicinata, H. norvegica, Balanus amphitrite, etc. Provides compilation of pollutant tolerant species that might inhabit areas near outfalls near Alexandria. A well-done survey, which provides a good example for work in Alexandria.

19. United National Environment Programme. Mediterranean Action Plan and the Final Act of the Conference of Plenipotentiaries of the Coastal States of the Mediterranean Region for the Protection of the Mediterranean Sea. 1978.

Egypt signed this plan, which includes: (1) Convention for the Protection of the Mediterranean Sea against Pollution; (2) Protocol for the Protection of the Mediterranean Sea from Dumping by Ships and Aircraft; (3) Protocol Concerning Cooperation in Combating Pollution of the Mediterranean Sea by Oil and Other Harmful Substances in Cases of Emergency. In this document, the signing parties agreed in principle to protect the Mediterranean from pollution, including land-based pollution, but did not agree to any specific standards or procedures relevant to land-based pollution.

In the section on discharges from ships and aircraft, they placed toxic and persistent organohalogen compounds and their precursors,

sistent plastics and synthetics, crude oil and hydrocarbons, and radioactive wastes on a "black list." Arsenic, lead, copper, zinc, beryllium, chromium, nickel, vanadium, selenium, antimony and related compounds, cyanides, fluorides, other pesticides and their byproducts, potentially hazardous synthetic organics, and dangerous strengths and quantities of acids and alkalines were placed on a permissible list if discharged with care. Both lists were stated specially not to apply to wastes such as sewage sludge and dredge spoils.

The lists show a concern for persistent and food-chain accumulable waste products, which are parallel to the priority of concerns used by the Environmental Protection Agency in the United States.

20. United Nations Environmental Programme. Final Act of the Kuwait Regional Conference of the Plenipotentiaries on the Protection and Development of the Marine Environment and the Coastal Areas. 1978.

An agreement of principles, not specific scientific/engineering criteria.

21. Wahby, S.D. Pollution by Petroleum Hydrocarbons Along the Alexandria Coast. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Collected floating tar balls. Oil and tar pollution originated from tankers, the 'SUMED' pipeline, accidental spills in the Western Harbor, and Alamein oil field. Provides some hydrographic, meteorologic and oceanogrphic information. Good for background and other pollution sources.

22. Zavodnik, D., and N. Zavodnik. The Benthos of Rijeka Bay - Subject to Stress of Pollution. C.I.E.S.M. Workshop on Pollution of the Mediterranean. 1978.

Used modern methods (scuba transects and grabs) but approached the subject on a biocoenotic basis, based mainly on height relative to tide and bottom type. For sandy sublittoral areas, get Cymodocea nodosa and Echinocardium cordatum; for rocky areas get Bunodlactis verruscosa, Rocellaria dubia, Balanophyllia italica, Cystoseira discors, and Sargassum vulgare. Pure sands get C. nodosa at 150-200 gm wt/m^2. Deeper silty sediments get Turritella communis, Cultrensis adriaticus, Nephthys hystricis, Marphysa Kinbergi, Jaxea nocturna, and Oestergrenia digitata. Pollution effects are

restricted to littoral areas due to surface plumes. Similar species, or ecological homologues, would be expected of Alexandria.

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2. Abdel Moneim, A. A. Eutrophication of Lake Maryout. M. Sc. Thesis, Faculty of Science, Alexandria University. 1977.

3. Abou el Naga, W. M. The Occurrence and Distribution of Some Trace Metals in the Mediterranean Waters off Alexandria and Their Effect on the Water Productivity. M. Sc. Thesis, Faculty of Science, Alexandria. 1979.

4. Ahmed, F. M. E. Currents and Water Masses of the Coastal Area from Abu Qir to Agamy. M. Sc. Thesis, Faculty of Science, Alexandria University. 1979.

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8. El Samra, M. A. Physical and Chemical Studies on Lake Idku and Abu Qir Bay and the Mixed Water Between Them. M.Sc. Thesis, Faculty of Science, Alexandria University. 1973.

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13. Hassan, H. M. The Hydrography of the Mediterranean Waters Along the Egyptian Coast. M. Sc. Thesis, Alexandria University. 1969.

14. Sabra, A. F. M. Wind, Currents and Sea Level Variations Over the Continental Shelf off Alexandria Coast. M. Sc. Thesis, Faculty of Science, Alexandria University. 1979.

15. Samaan, A. A. Fisheries Investigations of Sardine and Other Pelagic Fish Along the Mediterranean Coast from Rashid to El-Sallum. Technical Report No. (2/3). Academy of Scientific Research and Technology, 114 pp. 1979.

16. Samaan, A. A. Primary Production of Lake Maryout. Ph. D. Thesis, Faculty of Science, Alexandria University. 1966.

17. Sultan, H. A. Preliminary Investigation on the Primary Production of Marine Phytoplankton at the Egyptian Mediterranean Coast around Alexandria Region. Thesis submitted to the Faculty of Science, Alexandria University, 237 pp. 1975.

18. Wahby, S. D. and Abdel Moneim. The Problem of Phosphorus in the Eutrophic Lake Maryout. Estuarine and Coastal Marine Science. 1979.

19. Wahby S. D., et.al. Chemical Characteristics of Lake Maryout. Estuarine and Coastal Marine Science. 7:17-28. 1978.

Table L-1
OUTFALL CHARACTERISTICS PROPOSED BY
1978 MASTER PLAN (MODIFIED)

Parameter	Units	Kait Bey	Sidi Bishr
Average dry weather flow	m^3/s	4.54	6.42
Peak instantaneous flow	m^3/s	6.68	9.30
Velocity, average/peak	m/s	2.00/2.94	1.69/2.37
Inside diameter	mm	1,700	2,200
Length (from shore)	m	8,000	10,000
Added length of diffuser	m	50	00
Total length from P.S.	m	8,450	12,600
Diffuser ports diameter	mm	50	50
Diffuser depth	m	40	55
Friction factor (Mannings "N")	--	0.016	0.016
TDH at pump station (peak)	m	58	41
Initial dilution	--	50	50
Subsequent dilution	--	7	7
Total dilution (at beaches)	--	1,000	1,000
Construction material	--	Mortar-lined-and-coated steel pipe	
1977 construction cost[a] (Land and sea portions + diffuser)	LE	40,000,000	46,000,000
1980 construction cost[a] (Land and sea portions + diffuser)	LE	52,000,000	60,000,000

[a] Vol II, Master Plan

[b] Escalated by ENR Index Ratio, 1980:1977

Table L-2
SUMMARY OF 1978 WATER QUALITY CRITERIA

Constituent (S)[a]	Adopted Background Value, C_o	Adopted Ambient Criteria, C_r	Conditions	Sidi Bishr Outfall: Typical Effluent Value, C_e	Sidi Bishr Outfall: Required Dilution, S[a]	Kait Bey Outfall: Typical Effluent Value, C_r[b]	Kait Bey Outfall: Required Dilution, S[a]
Fecal Coliforms per 100 ml	30 to 40	--		10^9	10^3 dilution	10^9	10^3 dilution
					10^3 dilution		10^3 dilution
BOD	1 mg/l	4.4 mg/l	Initial dilution	480 mg/l	141	510 mg/l	150
	Nil	1.4 mg/l over background	Far-field dilution	480 mg/l	343	510 mg/l	364
COD	Nil	1.4 over background	Far-field dilution	960 mg/l	686	1,020 mg/l	730
SS	0.1 mg/l	1 mg/l	Far-field dilution	600/mg/l	667	536/mg/l	595
N	0.0014 mg/l	0.028 mg/l	Far-field dilution	31 mg/l	1,165	24 mg/l	902
Adopted design dilution			Initial		150		150
			Far-field		1,000		1,000

Source: Alexandria Wastewater Master Plan Studies. Volume IV, Marine Studies October 1978, pages 4-27

a

$$S = \frac{C_e - C_o}{C_r - C_o}$$

b Assuming preliminary treatment only, without chlorination.

Table L-3
RELATIVE CHANGES IN SEDIMENT ORGANIC MATTER AND INFAUNA RESPONSES

Benthic Infauna Conditions	Range of BOD Change	Range of % Volatile Solids Change
Control Number	100%	100%
Changed	137-383%	133-207%
Degraded	1,482-2,693%	297-737%

Table L-4
RECOMMENDED WATER QUALITY STANDARDS FOR TOXICANTS

Toxicant	Recommended Safe Levels After Initial Dilution (ug/l)	Bioaccumulation Potential	Resultant Effluent Concentration (mg/l) 150:1 Dilution
Arsenic	50[a]	Negligible	7.5
Cadmium	5[a]	Moderate	.75
Chromium	100[a]	Negligible	13.0
Copper	20[b]	Negligible	3.0
Lead	32[b]	Moderate	4.8
Mercury	0.1[a]	Severe	0.015
Nickel	100	Negligible	1.5
Silver	1.8[b]	Negligible	0.27
Zinc	80[b]	Negligible	12.0
Cyanide	5[a]	None	0.75
Phenols	1[a]	Negligible	0.15
Total Identifiable Hydrocarbons	4[b]	Moderate	0.6
Ammonia	2,400[b]	None	360.0

[a]Quality Criteria for Water, U.S. EPA 1976

[b]Maximum receiving water limits set by the California State Water Resources Control Board. Water Quality Control Plan for Ocean Waters of California, 1978.

Table L-5

WATER QUALITY STANDARDS PROPOSED BY WWCG (1981)

Public Health	Fecal coliforms not to exceed 100 MPN/100 ml at the recreational beaches
Beneficial Uses	
Dissolved Oxygen	Dissolved oxygen not to be reduced by more than 10% beyond the zone of initial dilution.
Suspended Solids	Sediment accumulation not to exceed normal (natural) rates by a factor of 10 in any more than 5% of the fisheries grounds off Alexandria
Toxicants	

Toxicant	Recommended Safe Levels[a] (ug/l)	Resultant Effluent Standard Concentration (mg/l, 150:1 dilution)
Arsenic	50[b]	7.5
Cadmium	5[b]	0.75
Chromium	100[b]	13.0
Copper	20[c]	3.0
Lead	32[c]	4.8
Mercury	0.1	0.015
Nickel	100[b]	1.5
Silver	1.8[c]	0.27
Zinc	80[c]	12.0
Cyanide	5[b]	0.75
Phenols	1[b]	0.15
Total Identifiable Hydrocarbons	4[c]	0.6
Ammonia	2,400[c]	360.0

[a]After initial dilution.

[b]Based upon Quality Criteria for Water, U.S. EPA 1976

[c]Based on daily maximum receiving water limits set by the State Water Resources Control Board. Water Quality Control Plan for Ocean Waters of California, 1978.

Table L-6
SEABED SETTLEABLE SOLIDS ACCUMULATIONS

Kait Bey Outfall			Sidi Bishr Outfall					
			Summer			Winter		
Area (m^2)	Deposition Rate $mg/cm^2/day$	Deposition Rate $mg\ C/cm^2/day$	Area (m^2)	Deposition Rate $mg/cm^2/day$	Deposition Rate $mg\ C/cm^2/day$	Area (m^2)	Deposition Rate $mg/m^2/day$	Deposition Rate $mg\ C/cm^2/day$
$5x10^5$	.40	.14	$4.5x10^5$	.62	.22	$6.1x10^5$	.46	.16
$9.6x10^5$	.50	.18	$8.5x10^5$	.79	.28	$1.2x10^6$	.57	.20
$1.4x10^6$	.36	.13	$1.3x10^6$	.57	.20	$1.8x10^6$	.41	.14
$1.2x10^7$	.13	.05	$1.0x10^7$	.21	.07	$1.4x10^7$	.15	.05
$1.4x10^7$	.04	.01	$1.3x10^7$	.06	.02	$1.8x10^7$	.04	.01

Total Area Receiving More Than .12 $mg/cm^2/day$

Kait Bey Outfall:
15 km^2

Sidi Bishr Outfall:
Summer: 13 km^2
Winter: 18 km^2

Table L-7

BENTHIC ORGANISMS NEAR OUTFALLS AND POTENTIAL RESPONSES TO EFFLUENTS

Organism	Feeding Mode	Substrate	Possible Response to Effluent[a]
Porifera (sponge)			
Hippospougia	Filter feeder		Favor
Communis	Filter feeder		Favor
Other sp.	Filter feeder		Favor
Coelenterata			
Gerardia sp.	Filter & suspended particle feeding		Favor
Polycaheta			
Glycera	Predator	Silt-sand	Tolerant
Hermione	Commensal with bivalves		Tolerant
Crustacea			
Dromia perseuata	Scavenger		Tolerant
Actaea rutopunctata	Scavenger		Tolerant
Mollusca			
Chitons			
Chiton olivaceus	Herbivore		Negative
Gastropod			
Turitella sp.	Filter		Tolerant
Siliquaria anguina	Filter		Tolerant
Bittium sp.	Predator		Tolerant
Rissoa costata	Herbivore		Negative
Persicula	Predator	Sand	Tolerant
Aporrhais pespelicann	Scavenger	Gravelly mud	Tolerant
Vermetus gigas	Filter & suspension	Encrusting	Favor
Natica sp.	Predator		Tolerant
Opisthobranch			
Bullaria striata	Predator	Silt-sand	Tolerant
Scaphopoda			
Dentalium	Direct-deposit feeder	Sandy	Tolerant

[a]The possible responses to wastewater are based primarily on the feeding modes of organisms and do not reflect predictions based on actual effluent exposure.

Table L-7 (continued)

Organism	Feeding Mode	Substrate	Possible Response to Effluent[a]
Bivalves			
Chlamys	Filter feeder	Nestling	Tolerant
Arca	Filter feeder		--
Posinia	Filter feeder	Sandy (in)	--
Venus verrusoca	Filter feeder	Sandy (in)	Tolerant
Cordium	Filter & suspension	Silty,	Tolerant
Cyrtodaria	Filter feeder	Sandy	Negative
Spisula	Filter feeder	Sandy	Negative
Modiolous barbatus	Filter feeder	Nestling & encrusting	Tolerant
Lima sulcata	Filter feeder	Nestling	Tolerant
Tellina	Suface deposits	Silty	Favor
Bryozoa			
Retipora cellulosa	Filter feeder	Encrusting solid substrate	Favor
Echinodermata			
Echnaster	Predator	Variable/mobile	Negative
Echinoid	Omnivorous & herbivorous	Variable/mobile	Negative
Amphiura chiajei	Surface deposit feeder	Variable/mobile	Tolerant
Ascidia			
Botryllus	Filter & suspension	Attached/hard substrate	Tolerant

[a]The possible responses to wastewater are based primarily on the feeding modes of organisms and do not reflect predictions based on actual effluent exposure.

Table L-8
ESTIMATED CAPITAL COSTS FOR OUTFALLS
(LE, end-of-1980 prices)

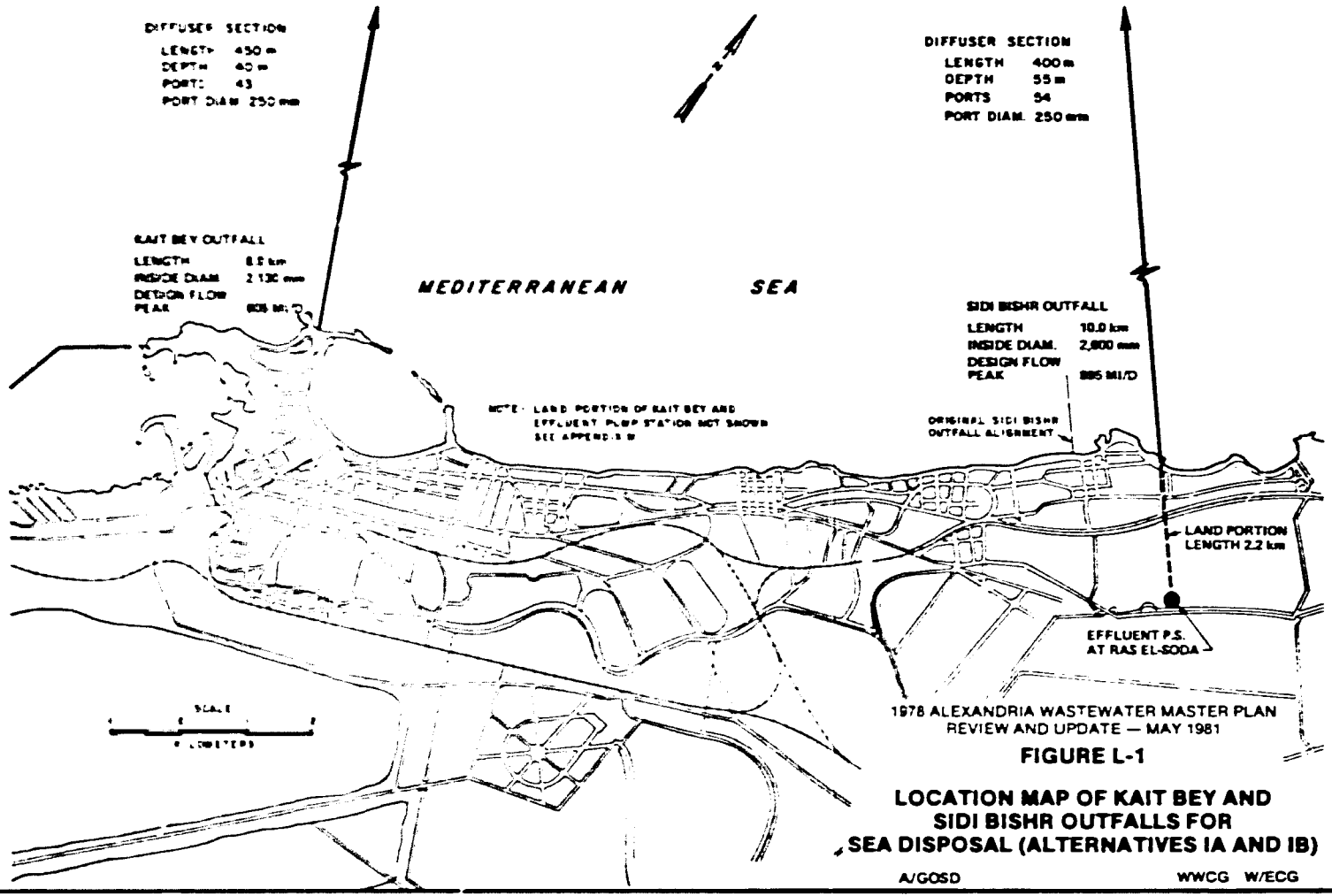
Item	Kait Bey	Sidi Bishr
Sea portion	36,829,000	45,932,000
Land portion	Not incl. See App. M	5,000,000
Subtotal	36,829,000	50,932,000
Contingencies on sea portion @ 40%	14,732,000	18,373,000
Contingencies on land portion @ 25%	Not incl. See App. M	1,250,00
Subtotal	51,561,000	70,555,000
Engineering, legal and administrative @ 15%	7,734,000	10,583,000
Total Est. Cost	59,295,000	81,138,000

Table L-9
ESTIMATED ANNUAL MAINTENANCE/INSPECTION COSTS
FOR THE KAIT BEY AND SIDI BISHR OUTFALLS

Item	Initial year		Year 1990		Year 2000	
	Quantity	LE Cost	Quantity	LE Cost	Quantity	LE Cost
Annual Inspection						
Mandays	160		160		160	
Cost		16,000		16,000		16,000
Annualized Cost of Major Repair Event[a]	N/A	130,000	N/A	130,000	N/A	130,000
Endgate/Diffuser Cleaning[b] Cost		16,000		16,000		16,000
Total Estimated Cost		162,000		162,000		162,000

[a]Based on the probability that one major accident costing LE 2 million could occur during the first 15 years of outfall operation.

[b]Based on cleaning once in 5 years

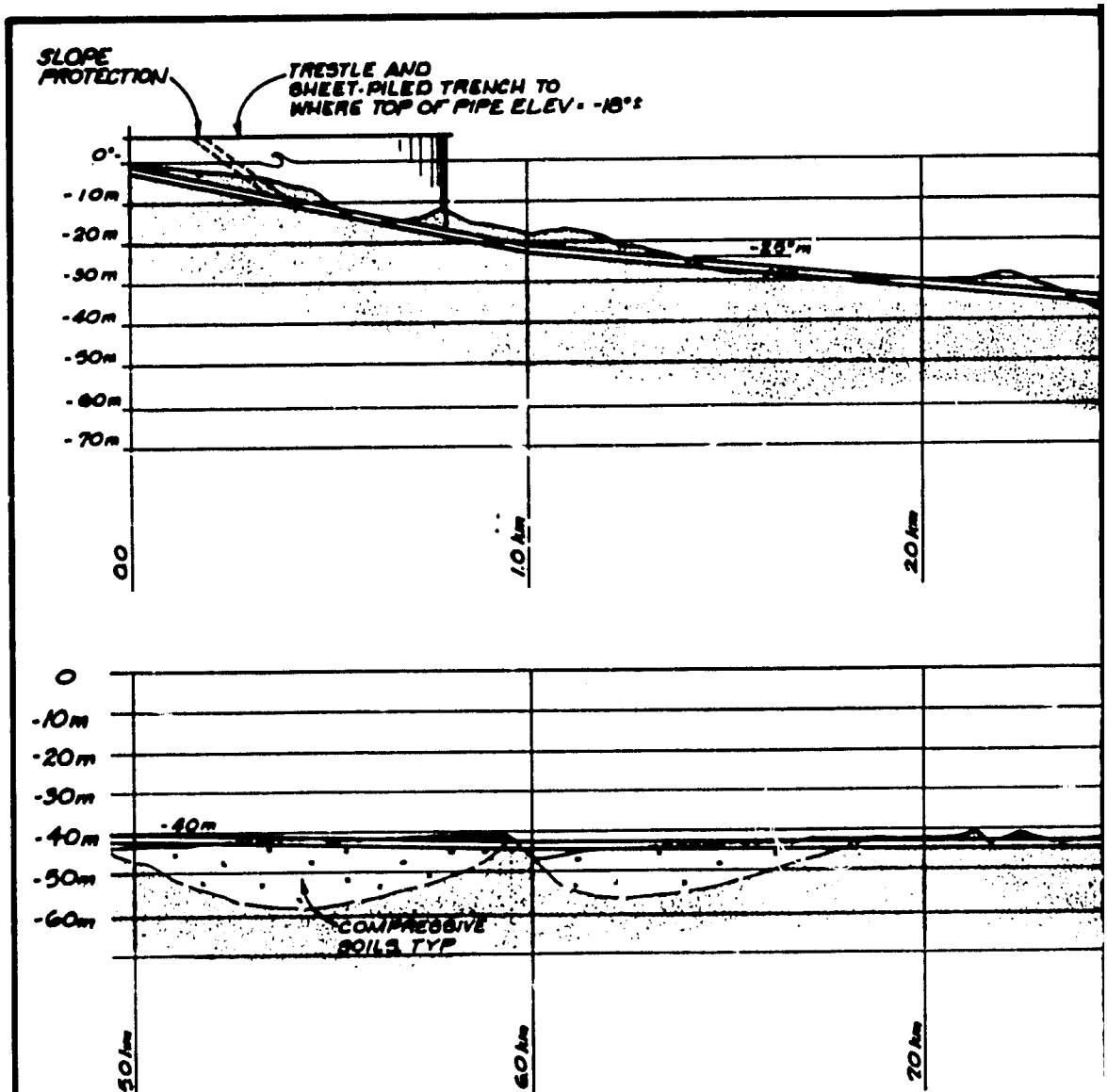


1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE L-1

LOCATION MAP OF KAIT BEY AND SIDI BISHR OUTFALLS FOR SEA DISPOSAL (ALTERNATIVES IA AND IB)

A/GOSD WWCG W/ECG


SLOPE PROTECTION
TRESTLE AND SHEET-PILED TRENCH TO WHERE TOP OF PIPE ELEV = -18°±
0°
-10m
-20m
-30m
-40m
-50m
-60m
-70m
-26°m
0.0
1.0 km
2.0 km
0
-10m
-20m
-30m
-40m
-40m
-50m
-60m
COMPRESSIVE SOILS TYP
5.0 km
6.0 km
7.0 km

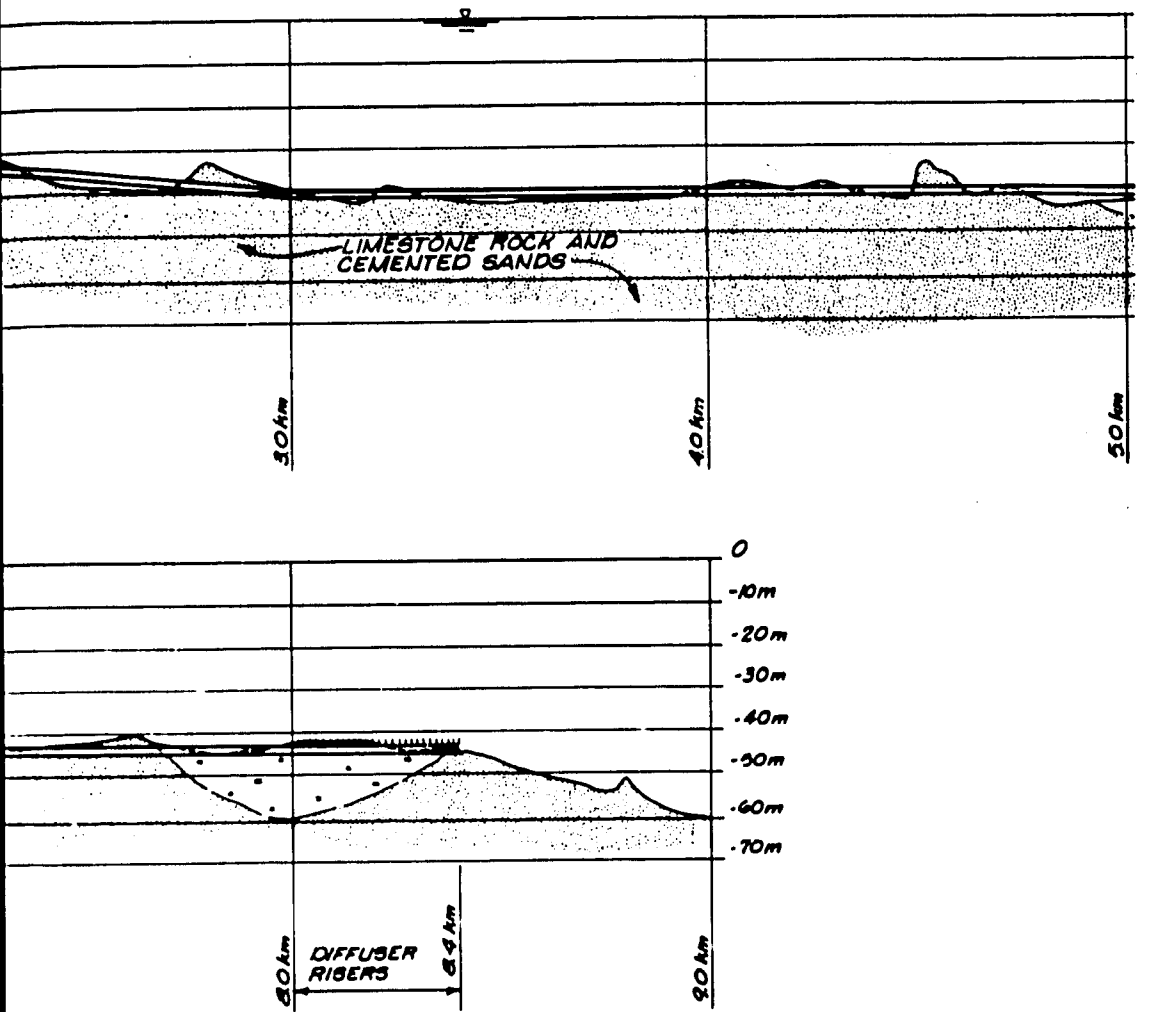


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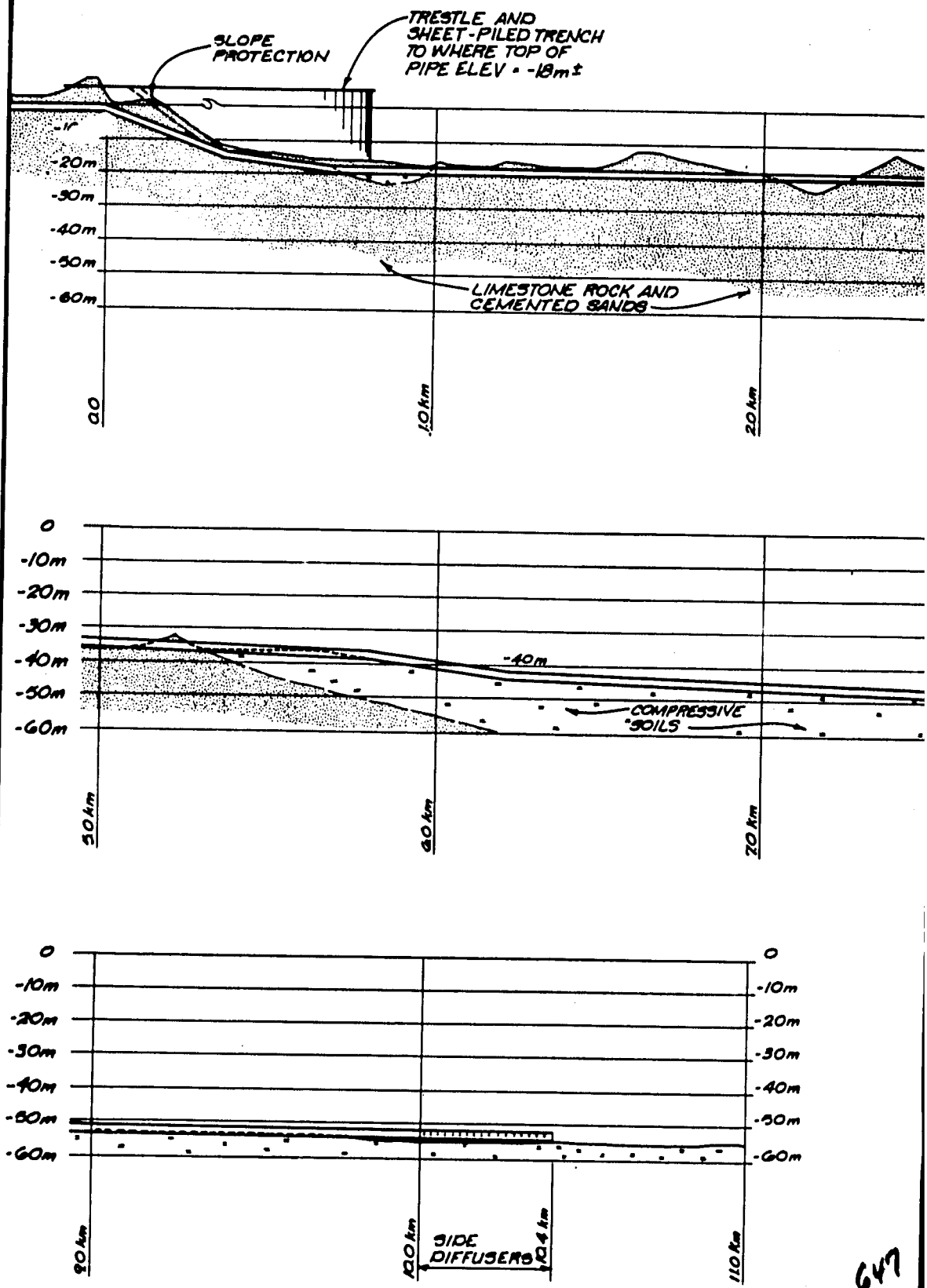
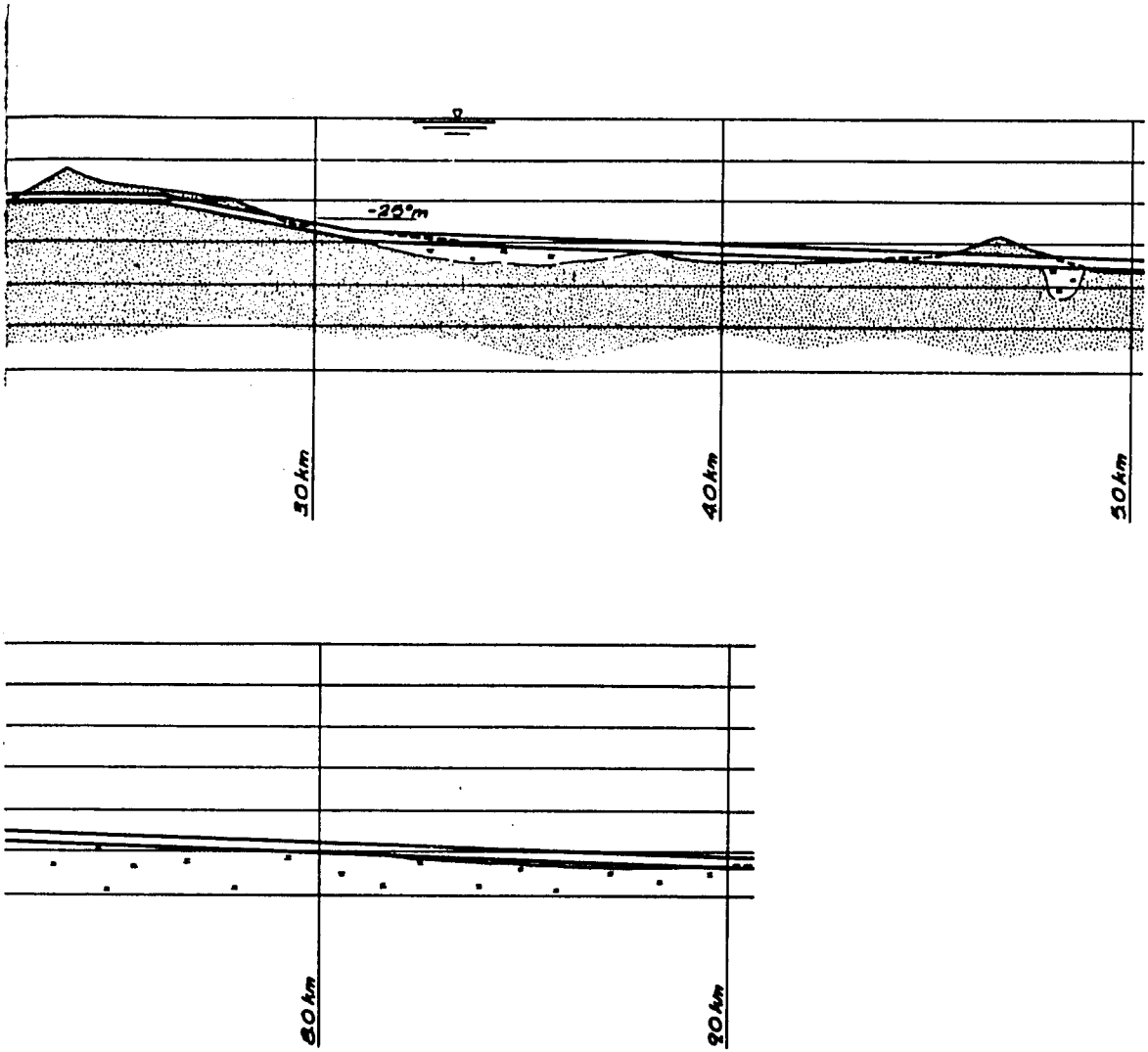
FIGURE L-2

KAIT BEY OUTFALL PROFILE

A/G08D WWCG W/ECG


SLOPE PROTECTION
TRESTLE AND SHEET-PILED TRENCH TO WHERE TOP OF PIPE ELEV = -18m ±
-11
-20m
-30m
-40m
-50m
-60m
LIMESTONE ROCK AND CEMENTED SANDS
0.0
10 km
20 km
0
-10m
-20m
-30m
-40m
-50m
-60m
-40m
COMPRESSIVE SOILS
30 km
40 km
70 km
0
-10m
-20m
-30m
-40m
-50m
-60m
90 km
100 km
SIDE DIFFUSERS
104 km
110 km



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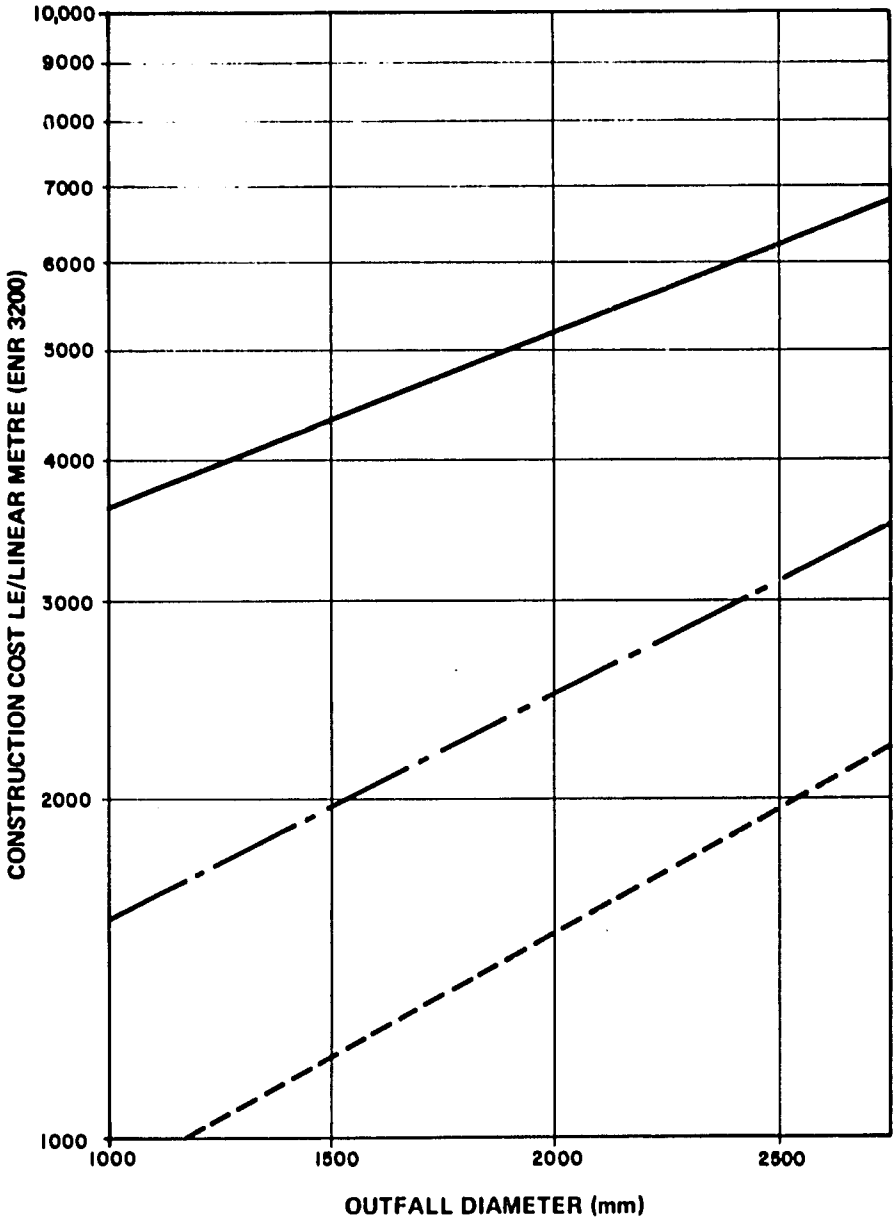
REVIEW AND UPDATE — MAY 1981

FIGURE L-3

SIDI BISHR OUTFALL PROFILE

A/G06D WWCG W/ECG



EXPENSIVE
MODERATELY PRICED
INEXPENSIVE

SOURCE: WALUS JOURNAL WPCF. VOL. 51, NO. 5, 1979

1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE L-4

COMPARATIVE COSTS FOR OUTFALL CONSTRUCTION UNDER VARYING CONDITIONS

A/GOSD WWCG W/ECG

APPENDIX M

CONVEYANCE SYSTEMS

This appendix discusses alternative systems of collecting and conveying both untreated and treated wastewater to proposed sites for treatment and/or disposal. Three systems are proposed: one system for each of the three major alternatives of treatment and disposal.

Conveyance from areas that are both sewered and unsewered at the present time are included. The conveyance systems have been developed using the general criteria listed below.

- All existing sewers and those presently under construction that are part of the conveyance system are to be included.
- All lines required to intercept sanitary sewage flows from pump stations to be abandoned are to be included.
- Major gravity collectors, pressure sewers, and pump stations required to convey flow from unsewered areas are to be included.
- Collection within unsewered areas is not included.
- Proposed interim improvements are considered as existing facilities.
- Existing facilities are used where possible and practical.
- Facilities are sized to convey peak dry weather wastewater flows.
- Gravity sewers are sized to handle projected year 2030 flows.
- Pressure sewers are sized to ultimately handle projected year 2030 flows. A comparison of flow conditions for both year 2000 and year 2030 was made. Pipes sized for year 2030 are used if conditions warrant.
- Pump station structures are sized to accommodate projected year 2030 flows. Initial equipment is planned for pumping year 2000 projected flows.

Much of the existing collection system uses combined sewers to convey both wastewater and stormwater to the major collectors and pump

stations. Excess stormwater flows are diverted from the present conveyance system by gravity overflows and separate facilities at pump stations, through drains to Lake Maryout and to the Mediterranean Sea.

Separate stormwater and wastewater systems are desirable. It is beyond the scope of this review to study possible methods of separating these flows within the present system. Further study of this problem may be addressed during development of the basis of design report for components of the adopted conveyance system. For the purposes of this study, we have assumed that the combined system will continue in use.

The area encompassed by the study has been divided into the following general zones for planning the conveyance facilities:

- Inner west and central (western and central sewerage zones)
- Inner east and outer east (eastern sewerage zone)
- Nouzha
- Desert

Because of the inherent differences among the treatment/disposal alternatives, each requires a combination of three conveyance systems. Some of the alternative conveyance systems are similar with the only difference being the final destination of the wastewater. In all three treatment/disposal alternatives, the conveyance plan for the Nouzha area is the same.

The unsewered area of Abu Qir is outside the study area. Some sewage (principally domestic) from Abu Qir will, at least initially, probably be conveyed to the existing system for treatment. Because of this, we have included a year 2000 allowance of 52 Ml/d from Abu Qir in planning the conveyance alternatives. A treatment/disposal system choice for the sewage generated within the study area is not significantly impacted by the inclusion or exclusion of Abu Qir domestic sewage flows. Costs for conveying the flow from the Abu Qir area to the eastern terminus of the conveyance system studied in this report have not been included. The cost of capacity for the flow is included in the conveyance system within the study area.

Flows from projected development in the Sadat City and Manshiya El-Bahareyah areas have been included in planning the treatment/disposal and conveyance alternatives. Costs have been included to convey these flows to the system. Costs for collection within these areas are not included.

Several alternative plans for conveyance were explored for each zone. Only the recommended conveyance system for each alternative is presented in this appendix.

A discussion of existing collection systems is found in Appendix H.

DESCRIPTION OF ALTERNATIVES

The three treatment/disposal alternatives discussed in the report require eight alternative conveyance systems within the general zones listed above.

The basic conveyance system associated with each treatment/disposal alternative is listed below:

Alternative IB

- Wastewater from the inner western and central zones will be conveyed to the West Treatment Plant, receive primary treatment, and then be conveyed to the new Kait Bey outfall for sea disposal (Figure M-1).
- Wastewater from the inner and outer eastern zones and from Abu Qir will be conveyed to the Ras El-Soda Treatment Plant, receive primary treatment, and then be conveyed to the new Sidi Bishr outfall for sea disposal (Figure M-4).
- Wastewater from Nouzha will be conveyed to the East Treatment Plant, receive secondary treatment, and be discharged to the Qala Drain (Figure M-7).

Alternative II

- Wastewater from the inner western and central zones will be conveyed to the West Treatment Plant, receive secondary treatment, and then be discharged to the Umum Drain (Figure M-2).
- Wastewater from the inner and outer eastern zones and from Abu Qir will be conveyed to the Ras El-Soda Treatment Plant, receive secondary treatment, and then be discharged to local drains (Figure M-5).
- Wastewater from Nouzha will be conveyed to the East Treatment Plant, receive secondary treatment, and be discharged to the Qala Drain (Figure M-7).

Alternatives IIIA and IIIB

- Wastewater from the inner western, central, inner and outer eastern zones, and from Abu Qir will be conveyed to the East Site for treatment and land application (Figures M-3, M-6, and M-8).

- Wastewater from Nouzha will be conveyed to the East Treatment Plant, receive secondary treatment, and be discharged to the Qala Drain (Figure M-7).

Inventories of gravity lines, pressure lines, and pump stations proposed for each alternative are contained in Tables M-1 through M-9.

BASIS OF EVALUATION

The recommended conveyance systems for the three treatment/disposal alternatives were selected from several systems considered. The merits of each proposed system were compared using the following elements as a basis for evaluation:

- Costs: capital construction costs and long-term operation and maintenance costs (see Appendix B)

- Design: adherence to standards set for design and construction (see Appendix A)

- Flexibility: ability to meet present conditions and also allow for new conditions, anticipated and unanticipated

- Local conditions: local construction practices and availability of materials for both initial construction and lifetime operation and maintenance; consideration of geological and geographical conditions affecting construction and maintenance of facilities

- Desirability: consideration of factors that affect public acceptance and systems operations; environmental factors such as routing of pipelines and placement of pump stations and their effect on present and future land use, both permanent and temporary; engineering factors such as accepted practices and "standards of the industry," e.g., gravity sewers are generally more desirable than force mains

POWER SUPPLY TO PUMP STATIONS

Each pump station will have two independent sources of electrical power. These two services may be either two independent and reliable utility services, or a single utility service together with a dedicated onsite power generation system. The choice between these two alternatives for power supply redundancy must be made individually for each pump station. For purposes of review and update of the 1978 Master Plan, however, we assumed that the larger pump stations' total connected load of 750 kW and greater will be served by two reliable and independent utility feeds. The smaller pump stations may be served by a single utility feed and an onsite power generation system where this is determined to be cost-effective. A more specific development of power supply criteria and support thereof follows.

Where pump stations are equipped with onsite engine-driven power generation systems, approximately 20 to 30 minutes will elapse from primary source power failure to resumption of normal pump station operation. Therefore, only outages of 30 minutes or longer are of concern when choosing between an onsite power generation system or, alternatively, a second power company feed for power supply redundancy. Records from the Alexandria Company for Electric Distribution (hereafter referred to as the power company) show only moderate power supply reliability at the intermediate (10 kV) voltage. Detailed documentation from the power company is not available, however.

In the absence of power company statistics, the existing A/GOSD pump station records were reviewed. These records show that at any single point, the 10.5-kV power supply is interrupted for 30 minutes or longer approximately three times per month. The power company's 66-kV power supply system is, as would be expected, much more reliable. Single service outages of 30 minutes or more can only be expected approximately once each 6 months. In the event that power supply redundancy is provided by two independent 66-kV power feeds, the expected frequency of pump station outages exceeding 30 minutes would be acceptably low. That is, the frequency and duration of outages with two independent 66-kV power feeds would be within or below that range expected when an onsite power generation system is provided as the redundant power supply. Standby power generation systems would be expected to suffer many, if not more, of the operational and maintenance problems that detract from the power company's reliability.

Power supply reliability from the power company, along with the high cost and high maintenance requirements for onsite generation, virtually eliminates the engine-generator option for the larger pump stations. As such, large pump stations, where individual motors are 300 kW or larger

and the total load is greater than 750 kW, are planned to include the following power supply features:

- Two incoming subtransmission voltage power lines, each from a different substation served by a 66-kV feed. In the case of the large East, West, and Desert pump stations under Alternatives IIIA and IIIB, the power feed to each is planned to be via two 66-kV overhead power lines on separate poles brought to the sites from the power company's main 66-kV loop around Alexandria. Separate power company substations, located on different segments of the loop, are proposed as power feed origins.

- A double-ended substation located at the pump station. This substation is planned to consist of: (a) two power transformers (each of which could carry the entire pump station load) to change the subtransmission voltage to the pump station utilization voltage; and (b) switchgear to isolate either source.

- Pump station utilization voltages are planned to be 4.16-kV and 480-V, three-phase U.S.A. standard power, and 240-V, single-phase U.S.A. standard power. This selection allows optimum switchgear, transformers, protective devices, and motors.

In the cases where individual motors are less than 300 kW, total load is less than 750 kW, and 66-kV power is not readily available, a single source from the power company and an onsite generation system is planned. These smaller stations are proposed to utilize 380-V, three-phase Egyptian standard power.

INNER WESTERN AND CENTRAL ZONES

Alternative IB

A new western zone collector sewer, varying from 1200 mm to 1700 mm in diameter, will collect domestic and industrial wastewater from the inner western zone, which presently flows into Lake Maryout from discharges into Mohsein Pasha Drain and Gabbary Drain (see Figure M-1). The collector route will extend approximately 4 km west, from near Gheit El-Enab main pump station to the West Treatment Plant. This collector will eliminate the present need for six pump stations; the Gheit El-Enab main and auxiliary, Mohsein Pasha, the old and new Furn El-Gueraya, and the No. 1 West pump stations will be abandoned. The existing Industries pump station will be rebuilt and will pump industrial wastewater to the West plant through a newly constructed pressure line 914 mm in diameter not yet in service. Flows from the Mohsein Pasha pump station will be conveyed to the new collector in existing sewers.

A new gravity sewer, 800 mm in diameter, will collect wastewater from the No. 3 West pump station; this flow is currently pumped into a ditch tributary to Lake Maryout. This sewer will travel south from the pump station and east approximately 500 m to the West plant, eliminating the need for the No. 3 West pump station, which will be abandoned. The No. 2 West pump station will be rebuilt and will pump wastewater to the new sewer through approximately 1.2 km of new pressure line 550 m in diameter.

Flow from the central zone will be collected in a new Port Est collector, varying from 1100 mm to 1200 mm in diameter. Its upstream terminus will be near the Kait Bey pump station. It will extend about 2 km in a southerly direction to a new Central pump station to be built in the vicinity of Mohamed Ali Square. The existing Port Est collector will be abandoned. A new collector, 1800 mm in diameter, will intercept flow from the existing Interceptor General at El Sabaa Banat Street and convey it 400 m to the new Central pump station. West of the point of interception, the existing Interceptor General and the existing pump station at Kait Bey will be abandoned. Wastewater collected at the new central pump station will be conveyed through 5.1 km of pressure line 1200 mm in diameter to the West plant.

Alternative IB recommends disposing of the primary treated wastewater at sea. The effluent from the West plant will be pumped through 5.1 km of pressure line 2130 mm in diameter back to the Central pump station. It will then be pumped through 2 km of pressure line 2130 mm in diameter to the Kait Bey sea outfall (see Appendix L).

Alternative II

Conveying wastewater from the inner western and central zones to the West plant will be exactly as described for Alternative IB (see Figure M-2). Conveying the treated effluent for disposal in the Umum Drain will be as described in Appendix J.

Alternative III

The existing Industries pump station will be rebuilt and the industrial wastewater conveyed to the West Treatment Plant for pretreatment. (See Figure M-3 and Appendices J and F.) These flows will be conveyed through an unused existing pressure line 914 mm in diameter that has been constructed from the Industries pump station to the West plant. A new gravity sewer ranging in diameter from 800 mm to 1900 mm will collect wastewater from the inner western zone and will include the pretreated industrial effluent from the West plant. Untreated

wastewater presently flows into Lake Maryout from discharges at the Nos. 3 and 1 West pump stations, and the old and new Furn El-Gueraya, Gheit El-Enab main and auxiliary, and Mohsein Pasha pump stations. The new collector will originate at the No. 3 West pump station and extend 4.4 km east to the new West pump station located south of the Gheit El-Enab main pump station. This collector will eliminate the need for seven existing pump stations, which will be abandoned. Wastewater intercepted at the Mohsein Pasha pump station will be conveyed to the new collector in existing sewers. The No. 2 West pump station will be rebuilt and will pump wastewater to the new collector at the No. 3 West pump station.

Wastewater flow from the central zone will be collected in the new Port Est collector and existing Collector General and conveyed to the new central pump station exactly as described for Alternative IB. It will then be conveyed to the new West pump station through 3.9 km of new pressure sewer 1200 mm in diameter.

All wastewater from the inner west and central zones will be conveyed from the West pump station to the Desert pump station. (See DESERT ZONE.)

INNER EASTERN AND OUTER EASTERN ZONES

Stormwater conveyance in the eastern zones will not be affected by the treatment/disposal alternative selected. Several existing overflow points along the shore of the Mediterranean Sea will remain for excess stormwater flows. Provisions for pumping excess flows to the sea may be required at the Nos. 1 East, 3 East, 4 East, and 6 East pump stations, and at the Mamoura pump station. The Montazah pump station and the Nos. 2 East and 5 East pump stations may need to be retained for the sole purpose of conveying stormwater.

An overflow for excess stormwater flows will be provided in the Smouha area that will discharge, by gravity, to the Qala Drain.

A stormwater pressure main 914 mm in diameter is presently being constructed from the No. 11 East pump station to a subdrain in the Qala Drain system. WWCG recommends that this line be retained for excess stormwater flows as a part of a new pump station to be built on the site of the No. 11 East pump station. Gravity sewers conveying flow from the Nos. 7, 8, 9, and 10 East pump stations should be sized to convey both stormwater and wastewater to the new pump station. The size of these gravity sewers cannot be determined without further study. No costs for increasing the sizes of the gravity sewers or for providing stormwater pumping facilities are included.

Alternative IB

A new collector will be constructed to convey wastewater from the Smouha area (see Figure M-4). It will begin north of the existing Smouha pump station and will intercept flows from the existing Hadara, Ibrahimia south, and Aacha subsystems. This new collector will originate as a gravity sewer 1300 mm in diameter and will proceed 1.3 km east along Mohamed Ali Street. It will intercept flows from the existing Sidi Gaber and Abu Qir collectors, increase in diameter to 1600 mm, and continue 1.5 km south to a new Smouha pump station located near the existing siphon under the Mahmoudiya Canal. The existing Smouha pump station will be abandoned. The new Smouha pump station will also collect flows from a section of the existing Hagar El-Nawatiya collector. The flows from these two collectors will be pumped 9.5 km to the Ras El-Soda Treatment Plant utilizing approximately 3.2 km of an existing pressure line 1220 mm in diameter, which presently conveys wastewater from the No. 11 East pump station to the East plant. If flows develop as projected, a parallel pressure line 1065 mm in diameter will be necessary before year 2030 to provide the needed capacity.

A new collector will be constructed to collect the flows from the existing upper Hagar El-Nawatiya collector and to intercept flows entering existing Nos. 9 and 10 East pump stations. The collector, 800 mm in diameter, will originate west of the No. 9 East pump station and proceed north to where it will intercept the flows from the No. 9 East pump station. The collector will then increase to 900 mm in diameter and proceed east to where it will intercept flows from the No. 10 East pump station. With a further increase in diameter to 1100 mm, the collector will continue to the No. 11 East pump station. Nos. 9, 10, and 11 East pump stations will no longer be used to pump sewage. The new Abu Soliman pump station will be built at the site of the No. 11 East pump station.

The Siouf Keblia area will be served by a new collector that will convey the flows to the new Abu Soliman pump station. The collector will start in a street approximately 150 m west of the No. 7 East pump station. Flows from the pump station will be conveyed to the collector in a gravity line 500 mm in diameter. The collector, 700 mm in diameter, will proceed south 500 m and join an interceptor 450 mm in diameter from the No. 8 East pump station. The collector will increase to 900 mm in diameter and continue approximately 550 m to where a collector 1400 mm in diameter from the Manshiyah El-Bahareyah area will join it. Here the collector will increase to 1600 mm in diameter, and will continue west approximately 670 m to Abu Soliman Street. It will then increase to 1700 mm in diameter and proceed south 390 m to the Abu Soliman pump station. Nos. 7 and 8 East pump stations will be abandoned. Flows from the Mahmoudiya Keblia area south of the Mahmoudiya Canal will be con-

veyed through 640 m of gravity sewer 1000 mm in diameter to the Abu Soliman pump station. Wastewater will be pumped from the Abu Soliman pump station to the Ras El-Soda plant through 3.1 km of pressure line 2000 mm in diameter.

Conveyance of flows within the Ras El-Soda area will be as described in the 1979 Design Report for the Ras El-Soda Wastewater System. Three new gravity collectors will convey wastewater to the Ras El-Soda plant.

The Mohandes collector will originate in the vicinity of the existing Montazah pump station and will carry the flow from that pump station, Mamoura pump station, No. 1 East pump station, and Abu Qir. The Montazah pump station will be used for stormwater only. The collector will traverse nearly 3 km and will vary in diameter from 1100 mm to 1550 mm. Flow from the Hospitals pump station will be conveyed through 600 m of gravity line and will enter the Mohandes collector at a point approximately 2 km upstream from where the collector ends at the Ras El-Soda plant. The Hospitals pump station will be abandoned.

A new collector in Street 45 is presently under construction. The new sewer varies in diameter from 700 mm to 1800 mm. A new extension to the Street 45 collector 700 mm in diameter will intercept the flows from the No. 2 East pump station, which will be used only for stormwater flows. This new collector will terminate at the Ras El-Soda plant.

The Cairo Street collector will originate at the intersection of Cairo Street and Malek Hefny Street. It will intercept flow from the Nos. 4 and 6 East pump stations and convey it approximately 930 m to the Ras El-Soda plant. The collector will vary in diameter from 700 mm to 1200 mm. A pressure line, approximately 0.8 km long and 400 mm in diameter, will be constructed from the No. 4 East pump station to the new collector. A new pressure line, 350 mm in diameter, will convey wastewater 1.4 km from the No. 6 East pump station to the new collector.

Flows from the future Sadat City will be pumped to the Ras El-Soda plant through a pressure line to be constructed when flows warrant it. Interim flows from the area may be discharged to the Ras El-Soda collection system. A pump station and 5 km of pressure line 1000 mm in diameter from Sadat City have been included in the cost estimates.

A new collector will be constructed along the Corniche. It will originate at the Glym pump station and proceed east 4.1 km to the No. 3 East pump station. Its size will vary from 850 mm to 1200 mm in diameter. It will intercept flows from Glym, Sarwat, Sidi Bishr, and the No. 5 East pump stations. These pump stations will be abandoned, except for the No. 5 East pump station (which will be retained for stormwater).

The No. 3 East pump station will be rebuilt and will convey the collected wastewater to the Ras El-Soda plant through approximately 3.3 km of pressure line 1000 mm in diameter.

Effluent from the Ras El-Soda Treatment Plant will be pumped to the Sidi Bishr sea outfall (see Appendix L).

Alternative II

Conveyance of wastewater in the inner and outer eastern zones to the Ras El-Soda plant will be exactly as described for Alternative IB (see Figure M-5). The effluent from the secondary treatment process will be discharged to the Abu Qir drainage system by way of existing local drainage collectors (see Appendix J).

Alternative III

Conveyance of wastewater in the Smouha area will be by the new Mohamed Ali Street collector as described for Alternative IB (see Figure M-6). Flows will be pumped from the new Smouha pump station to the new East pump station on the present site of the No. 11 East pump station, which will be abandoned. Conveyance to the East pump station will be through 3.9 km of pressure line 1220 mm in diameter. Approximately 3.2 km of the existing pressure line 1220 mm in diameter will be utilized. A parallel pressure line 1065 mm in diameter will be required to carry year 2030 projected peak flows.

Flows from the upper Hagar El-Nawatiya collector and the existing Nos. 9 and 10 East pump stations will be conveyed to the new East pump station. The conveyance system will be the same as described for Alternative IB. The Nos. 9, 10, and 11 East pump stations will be abandoned.

The configuration of the conveyance system from the Siouf Keblia area will be generally the same as that described for Alternative IB. In this alternative, the collector will begin near where Mustafa Kamel Street crosses the Montazah Canal and will intercept an existing siphon under the canal. The diameter of the collector will start at 1700 mm and will successively increase to 2500 mm. These increases in size will allow this line to collect flows from the Ras El-Soda system, Abu Qir, and Sadat City. The length of the collector in this alternative is 3.7 km. The collector will terminate at the new East pump station.

Wastewater from the Mahmoudiya Keblia and the Manshiya El-Bahareyah areas will be conveyed to the East pump station in collectors as described for Alternative IB.

Wastewater will be collected along the Corniche as described for Alternative IB and will be conveyed to the No. 3 East pump station. Existing pump stations at Glym, Sarwat, and Sidi Bishr, will be abandoned. The No. 5 East pump station will be retained for stormwater. The No. 3 East pump station is to be rebuilt and the flows conveyed 3.7 km to an interim relief sewer in Abu Soliman Street, which will serve the No. 11 East pump station (see Appendix H). The interim sewer will be used as a collector and will require a larger pipe if this alternative is selected. The flow in this collector will be conveyed south to the new East pump station. The cost of increasing the size of the relief sewer is not included.

Conveying wastewater in the Ras El-Soda area will be similar to the system described for Alternative IB. Flows will be conveyed to a new Ras El-Soda pump station located as described in the 1979 Design Report for the Ras El-Soda Wastewater System. Flows will be conveyed approximately 2 km from the new pump station to the Siouf Keblia collector pressure line, 1600 mm in diameter.

Interim flows from Sadat City will be discharged to the Ras El-Soda system until direct conveyance to the Siouf Keblia collector is necessary.

The flows from the Nos. 4 and 6 East pump stations will not be conveyed to the Ras El-Soda system. They will continue to be discharged to the Abu Soliman collector in the existing pressure lines.

All wastewater from the inner and outer east zone will be conveyed from the East pump station to the Desert pump station (see DESERT ZONE).

NOUZHA ZONE

WWCG recommends that separate stormwater and wastewater systems be planned for the Nouzha area. No provision for stormwater flows is included in the sewage conveyance facilities recommended for Nouzha. The same wastewater conveyance system will be used for all three alternatives (see Figure M-7).

Wastewater from the unsewered area west of the Alexandria airport will be collected in a new gravity sewer 750 mm in diameter and conveyed 0.85 km to a new pump station at Fisherman's Village (Ezbet El Saydeen). This flow will then be conveyed 1.7 km north in a pressure line 500 mm in diameter to the starting point of a new collector. The new collector will vary from 900 to 1100 mm in diameter and will extend east approximately 1.9 km to a new main pump station built to serve the Nouzha area.

Flow from the area north of Alexandria Airport will flow 430 m through a collector 650 mm in diameter to the Nouzha collector. Flow from the eastern Nouzha area will be conveyed to the Nouzha pump station through a new gravity collector 800 mm in diameter that will extend west approximately 0.5 km. Flow from the Nouzha pump station will be conveyed to the East Treatment Plant through 0.9 km of pressure line 950 mm diameter. Secondary treated effluent will be discharged to Lake Maryout by way of Smouha Drain in all alternative plans (see Appendix J).

DESERT ZONE

No system to convey wastewater to the Desert zone is needed for Alternatives IB and II.

For Alternatives IIIA and IIIB, flow will leave the West pump station in two pressure lines, both 1450 mm in diameter (see Figures M-3, M-6, and M-8). These pipelines will extend approximately 2.5 km south across Lake Maryout to where they will cross the Alexandria-Cairo Desert Road. They will continue another 2.5 km south across the lake and then enter an area of farmland reclaimed from the lake. From there, they will traverse farmland approximately 4 km south and then join the pipelines from the East pump station.

Flow from the East pump station will be conveyed in two pressure lines, both 1750 mm in diameter. The lines will extend south from the pump station approximately 9 km, crossing the Mahmoudiya Canal, the Alexandria-Cairo Delta Railroad, Delta Road, and the drinking water canal, and passing by the east side of the Hydrodrome. They will then travel southwest approximately 5 km and join the two pipelines from the West pump station. All of this route will be across low-lying farmland, some reclaimed from Lake Maryout.

The four pipelines will be routed southwest approximately 9 km through reclaimed farmland to the intersection with the Noubaria Canal, where the route will turn and extend south for approximately 15 km to the Desert pump station. The pipelines will cross the Noubaria Canal approximately 2 km before reaching the Desert pump station.

At the Desert pump station, the flows from the West and East pump stations will be combined and pumped to the land application site in three pressure lines 1850 mm in diameter. These pipelines will extend approximately 44 km to the discharge point for the land application. A fourth pipeline will need to be added to the conveyance system before year 2030 if the flows increase as projected.

The pumping and conveyance facilities at the Desert pump station will be designed to carry maximum day flows. Storage will be provided at the

Desert pump station for peak flows. Wastewater stored during peak periods will be returned to the system and conveyed to the land application site as the peak flows subside to rates below the maximum day design flow.

The storage for flow equalization would be provided in six reinforced concrete tanks, each 40 m in diameter with a 14-m side water depth, and equipped with a conical-shaped bottom. The wastewater would enter and exit the tanks at the low center points of the bottoms. Flow would be delivered to the tanks by variable capacity pumps designed to divert flows influent to the Desert pump station in excess of 1255 Ml/day. Discharge from the tanks would be by gravity through flow control gates to the pump station inlet, maintaining pump station flows slightly under 1255 Ml/day until the equalization tankage is empty. The equalization tank contents would be mixed by diffused air to minimize sedimentation, with associated tank cleaning requirements, and odor release. High-pressure washdown water would be provided to assist in daily tank cleaning. Final design would provide for the addition of tank covers and exhaust air scrubbing for additional odor control when and if needed in the future.

EVALUATION

Flexibility

The conveyance system alternatives in this appendix were developed on a conceptual basis. Since the conveyance will be refined in the final design, the flexibility to adapt the conceptual design was considered in the proposed conveyance systems.

Flexibility was considered in recommending the sizing of pump stations, pressure lines, and in some cases, gravity lines. In general, it is more economical to construct a gravity sewer to its ultimate size. If, due to limitations on the availability of large-diameter pipe, two pipes are necessary to carry the projected design flow, construction could be staged. Therefore, where projections of future flows allowed, staged construction of the pump stations was planned. They will be designed to allow equipment to be added as needed to meet flow requirements as they develop. The same reasoning was applied to some of the pressure sewers. If initial flows would not be sufficient to maintain recommended velocities in a line sized to accommodate projected design flows, a smaller initial line was recommended with provisions for a larger line when needed.

Each of the conveyance systems presented have opportunities for flexibility and no single alternative presents a great advantage over the others in this regard.

Operations

From the standpoint of operation and maintenance of a conveyance system, pump stations are the highest single item of cost. Costs connected with the operation of the pump stations, including the cost of power, can vary depending upon size and complexity of the individual stations. Alternatives IIIA and IIIB with 16 pump stations and a total year 2000 pumping capacity of 3975 Ml/d are highest in this category. Operation and maintenance costs for pump stations in Alternatives IIIA and IIIB are 264 percent higher than for Alternative IB and 286 percent higher than Alternative II. Alternatives IB and II each have 14 pump stations with total year 2000 pumping capacities of 2133 Ml/d and 1528 Ml/day, respectively.

Gravity sewers have the second highest unit costs for operation and maintenance of a conveyance system. Alternatives IIIA and IIIB have the largest amount of gravity lines, 31.4 km, and these are, in several cases, larger than those in the other two alternatives. Projected year 2000 operation and maintenance costs for gravity sewers in Alternatives IIIA and IIIB are 19 percent higher than for Alternatives IB and II. Alternatives IB and II each have the same amount and sizes of gravity sewer.

Pressure sewers have the lowest per unit cost for operation and maintenance of the system. In Alternatives IB and II, pressure lines make up 50 to 60 percent of the total piping in the system. In Alternatives IIIA and IIIB, pressure lines make up approximately 90 percent of the system. This is due to the large amount of pressure line required to convey wastewater to the land application site in the desert. Operation and maintenance costs for year 2000 for the pressure lines in Alternatives IIIA and IIIB are 380 percent higher than for Alternative IB and 590 percent higher than for Alternative II.

In evaluating the effects of the factors discussed above, the system to convey flows to the desert in Alternatives IIIA and IIIB is responsible for the largest difference between the alternatives. Total year 2000 operation and maintenance costs for Alternatives IB and II are 39 percent and 32 percent, respectively, of those for Alternatives IIIA and IIIB. Operation and maintenance costs for each alternative are listed in Tables M-13, M-14, and M-15.

Construction

Construction of a conveyance system as outlined in this appendix will be a major undertaking. Alternatives IB and II contain 65 km and 72 km of

pipe, respectively. Alternatives IIIA and IIIB have 279 km of pipe. The alternatives propose utilizing from 14 to 16 pump stations having individual pumping capacities for the year 2000 varying from 14 Ml/day to 1275 Ml/day. Constructibility of the proposed system for each alternative has been carefully considered. Information in Appendices A and B is used as a guide in proposing the design of the conveyance facilities.

For cost estimating purposes, the use of vitrified clay pipe for gravity sewers up to 600 mm in diameter was assumed. The use of this pipe would be subject to the comments made in Appendix A. Gravity sewers between 600 mm and 2000 mm in diameter were assumed to be made of glass reinforced plastic (GRP) pipe. There is a question as to the availability of GRP pipe in sizes above 1600 mm. We have been informed that GRP pipe up to and including 2000 mm in diameter will be manufactured in Egypt and will be available within the next 2 years. Imported pipe in these sizes is available and could be used, if local pipe is not available, without a great impact on the estimated cost of the conveyance system.

Imported ductile iron pipe is proposed for pressure sewers up to and including 1220 mm in diameter. Above this size, imported concrete cylinder pipe is proposed in all but two cases. In Alternative IB, reinforced concrete pressure pipe is proposed for the 2130-mm pressure line from the West plant, just as was proposed for the Kait Bey sea outfall (see Appendix L).

In Alternatives IIIA and IIIB, concrete cylinder pipe is proposed for the pressure pipes 1450 mm, 1750 mm, and 1850 mm in diameter from the West and East pump stations to the land application site. WWCG has received indications from a US-based pipe manufacturer that it would be feasible to build a pipe manufacturing plant in Egypt to produce concrete cylinder pipe in the proposed quantities. This manufacturer currently has concrete cylinder pipe manufacturing facilities in Saudi Arabia.

Water is expected to be a construction problem in deep excavations and in the lower areas of the city. Costs for dewatering by well-point systems and by sump pumping and the use of both wood and steel sheeting have been incorporated into the preparation of the construction cost estimates.

Construction of the facilities in all zones except the desert zone are similar in all the alternatives. Alternative IB incorporates installation of large pressure pipe to the sea outfall, but construction methods will be similar to those used in other pipe installations within the western and central zones.

Construction of the conveyance facility to the desert presents different construction conditions. The initial 5 km of pipeline will traverse

Lake Maryout after leaving the West pump station. Our estimate assumes that the two pipelines 1450 mm in diameter will be elevated where they cross the lake, supported on a system of concrete cradles and pilings.

The initial 14 km of the pipeline route from the East pump station, and 4 km of the pipeline from the West pump station, traverse an area that has been reclaimed from Lake Maryout and is now used mostly as agricultural land. This area, identified in the 1978 Master Plan as Zone 4, consists of subsoils of layers of clays. We assume that pile supports will be required through some of this area and that conventional cut-and-cover methods, with special attention given to pipe foundation and bedding, may also be used. Pile-supported sections will be elevated on cradle supports, as proposed for the lake crossing. Extensive dewatering and special soil stabilization and pipe foundation treatment is anticipated on the sections of buried pipeline.

The next 24 km of pipeline (two lines 1450 mm in diameter and two lines 1750 mm in diameter) will traverse more of the low-lying, reclaimed farmland and enter the desert plateau west of the Noubaria Canal. Construction of the pipeline across this Zone 4 area is expected to require some elevated, pile-supported sections and to allow some buried sections with special treatment, as proposed for the preceding section of line.

The final 44 km of pipeline from the Desert pump station to the land application site is across both developed and undeveloped desert land lying above elevation +9 metres. This area, classified as Zone 5 in the 1978 Master Plan, is expected to offer no special construction problems. In some areas, dewatering and temporary re-routing of irrigation and drainage channels will undoubtedly be required. The Desert pump station is located in the Zone 5 area, where conventional construction is anticipated.

The pipeline will cross two major canals, a major road, and various minor canals and roads along its route. The Noubaria Canal, the El Nasser Canal, and the Alexandria-Cairo Desert Road will require specially constructed crossings. Minor road and canal crossings are anticipated to require routine construction.

Desirability

The relative desirabilities of the conveyance system alternatives primarily depends on their environmental impacts. A detailed analysis is beyond the scope of this study, since proposed systems are conceptual in nature and pipeline routes approximate. Anticipated long- and short-term impacts are identifiable and are discussed.

The effluent pipelines from the West and Ras El-Soda Treatment Plants to the sea outfalls are the most important difference between the land-based portions of the conveyance system of Alternative IB and the other alternatives. Construction of the pressure line from the West Treatment Plant will result in major short-term adverse impacts. The proposed route follows existing streets through the central city. Approximately 1 km of the route passes through an area of high density commercial development. Near the Kait Bey outfall the roadway is narrow and congested. Construction activity will further aggravate existing transportation problems and may result in losses to abutting businesses. We do not anticipate removing any buildings along the route, nor do we believe the likelihood of finding archaeological resources is significant.

The adverse environmental impacts associated with the Alternative II conveyance system will be of the same nature, but will be less severe than those discussed for Alternative IB.

The desert conveyance system creates the major impacts of Alternatives IIIA and IIIB. Approximately 91 km of pipeline are required to route the flows to the land application site. Proposed right-of-way widths vary from 15 to 30 m, and are anticipated to require more than 90 feddans of land.

Construction of the proposed section of elevated pipeline from the West pump station across Lake Maryout is anticipated to have a short-term impact on water quality and fishing. The impact of crossing the main basin is expected to be negligible, since the area to be traversed is primarily reed grass and the value of existing fisheries is low. It is anticipated that fish landings will be reduced while the section of the pipeline crossing the fishery lagoon south of the Alexandria-Cairo Desert Road is under construction.

Disruption of agricultural production will be unavoidable in areas of the pipeline corridor occupied by farmland. The disruption is anticipated to be short-term; most of the land will return to production after construction of the pipeline.

The route does not traverse any known tombs shown on the 1949 Survey of Egypt topographic maps. A reconnaissance level archaeological survey to locate sites of possible historic or archaeological significance is recommended prior to final route selection for the pipeline.

The proposed pipeline route crosses the drinking water canal, the Noubaria Canal, and the El Nasser Canal. Leakage or failure of the pipeline at these crossings, especially at the drinking water canal, would have serious adverse impact. Special facilities and construction are recommended to minimize the possibility of failure.

Potential odor impacts at the Desert pump station can be minimized through proper design and a location away from present and proposed settlements.

Substantial energy requirements to operate the conveyance system to the land application site are unavoidable and will have a long-term impact.

Some beneficial impacts may result from constructing maintenance access roads in undeveloped areas. There is also the possibility that the route selection and the crossing of existing installations could be coordinated with future transportation needs.

COSTS

Capital construction costs for the alternative conveyance systems are presented in Tables M-10, M-11, and M-12. Operation and maintenance costs are presented in Tables M-13, M-14, and M-15. Table M-16 is a summary of the total cost of each of the alternatives.

Capital costs for the conveyance alternatives are estimated in accordance with the information and guidelines contained in Appendix B. Appendix B also contains the basis for the pump stations' operation and maintenance costs.

Operation and maintenance costs for pressure sewers are based upon a percentage of the capital cost for the system. Operation and maintenance of the system of pressure lines, exclusive of those in the desert zone, is calculated at 0.10 percent of the capital cost by computing labor costs at 0.03 percent, and materials and equipment costs at 0.07 percent. These percentages are based upon records of similar operating systems and information published by the US Environmental Protection Agency (EPA).

Operation and maintenance of the pressure sewers in the desert zone is expected to be a lower percentage of the capital cost than for the other pressure sewers within the system. Operation and maintenance of these lines is calculated at 0.06 percent of the capital construction cost (not including right-of-way costs), with labor at 0.02 percent and material and equipment at 0.04 percent.

Operation and maintenance costs for gravity lines in the conveyance systems are based upon the same sources of information as for the pressure lines. Operation and maintenance costs are calculated at 0.30 percent of the capital cost: 0.20 percent for labor and 0.10 percent for materials and equipment.

Labor is assumed to cost LE 6.50 per manday.

Costs for rights-of-way have not been included for the conveyance systems in the western, central, eastern, and Nouzha zones. We anticipate that most of the lines will be constructed in existing streets and easements. Any lines for which rights-of-way may have to be acquired will most likely be the same for each alternative. Any necessary right-of-way acquisition in these zones should have little impact on the calculated capital costs for the system.

Right-of-way costs have been calculated for the desert conveyance system. The system has a total route length of 91 km, with a large percentage traversing tillable land with no existing streets. Right-of-way costs are calculated for all portions of the desert conveyance system except the section crossing Lake Maryout. Costs are calculated at LE 10,000 per feddan of permanent right-of-way required. No allowances are made for property severance or penalties for disruption of present use such as crop production.

Table M-1
LENGTH OF GRAVITY CONVEYANCE LINES FOR ALTERNATIVE IB, SEA DISPOSAL
(km)

	Pipeline Diameter (mm)[a]							
	400	500	600	700	800	900	1000	1100
Inner West/Central								
West Zone					0.50			
Port Est								0.90
Coll. General								
Inner/Outer East								
Mohamed Ali								
Corniche						1.25		1.10
Cairo St.				0.89				0.46
Street 45		0.07		0.23				
Mohandes	0.30	0.30						0.47
Hagar El-Nawatiya					1.09	0.38		0.58
Slouf Kebila		0.35		0.50		0.55	0.64	
Nouzha								
Fishermen's Village					0.85			
Nouzha				0.43	0.50	0.60		1.30
Total	0.30	0.72		2.05	2.94	2.78	0.64	4.81

	Pipeline Diameter (mm)[a]						
	1200	1300	1400	1500	1600	1700	1800
Inner West/Central							
West Zone	1.00			1.20	0.73	1.10	
Port Est.	1.10						
Coll. General							0.40
Inner/Outer East							
Mohamed Ali		1.30			1.50		
Corniche	1.78						
Cairo St.	0.76						
Street 45							
Mohandes		0.51	0.87		1.12		
Hagar El-Nawatiya							
Slouf Kebila		0.31	0.90		0.67	0.39	
Nouzha							
Fishermen's Village							
Nouzha							
Total	4.64	2.12	1.77	1.20	4.02	1.49	0.40

[a]For convenience, line sizes are listed in 100-mm increments only. Planned half sizes are placed in the next higher category.

Table M-2

LENGTH OF GRAVITY CONVEYANCE LINES FOR ALTERNATIVE II, SECONDARY TREATMENT

(km)

	Pipeline Diameter (mm)[a]							
	400	500	600	700	800	900	1000	1100
Inner West/Central								
West Zone					0.50			
Port Est								0.90
Coll. General								
Inner/Outer East								
Mohamed Ali						1.25		1.10
Corniche				0.89				0.46
Cairo St.		0.07		0.23				
Street 45								0.47
Mohandes	0.30	0.30			1.09	0.38		0.98
Hagar El-Nawatiya						0.55	0.64	
Slouf Kebila		0.35		0.50				
Nouzha								
Fisherman's Village					0.85			
Nouzha				0.43	0.50	0.60		1.30
Total	0.30	0.72		2.05	2.94	2.78	0.64	4.81

	Pipeline Diameter (mm)[a]						
	1200	1300	1400	1500	1600	1700	1800
Inner West/Central							
West Zone	1.00			1.20	0.73	1.10	
Port Est.	1.10						
Coll. General							0.40
Inner/Outer East							
Mohamed Ali		1.30			1.50		
Corniche	1.78						
Cairo St.	0.76						
Street 45							
Mohandes		0.51	0.87		1.12		
Hagar El-Nawatiya							
Slouf Kebila		0.31	0.90		0.67	0.39	
Nouzha							
Fisherman's Village							
Nouzha							
Total	4.64	2.12	1.77	1.20	4.02	1.49	0.40

[a]For convenience, line sizes are listed in 100-mm increments only. Planned half sizes are placed in the next higher category.

Table M-3

LENGTH OF GRAVITY CONVEYANCE LINES FOR ALTERNATIVE III, LAND APPLICATION

(km)

	Pipeline Diameter (mm)[a]										
	400	500	600	700	800	900	1000	1100	1200	1300	1400
Inner West/Central											
West Zone					0.50						
Port Est.								0.90	1.10		
Coll. General											
Inner/Outer East											
Mohamed Ali										1.30	
Corniche						1.25		1.10	1.78		
Cairo St.		0.29	0.48	0.12			0.46	0.76			
Street 45		0.07		0.23							
Mohandes	0.30	0.30						0.47		0.51	0.87
Hagar El-Nawatiya					1.09	0.38		0.58			
Slouf Kebila		0.35					0.64			0.31	0.90
Nouzha											
Fisherman's Village					0.85						
Nouzha				0.43	0.50	0.60		1.30			
Total	0.30	1.01	0.48	0.78	2.94	2.23	1.10	5.11	2.88	2.12	1.77

	Pipeline Diameter (mm)[a]										
	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500
Inner West/Central											
West Zone	1.10		0.73	1.20	1.00						
Port Est.											
Coll. General				0.40							
Inner/Outer East											
Mohamed Ali		1.50									
Corniche											
Cairo St.											
Street 45											
Mohandes		1.12									
Hagar El-Nawatiya											
Slouf Kebila			1.46	0.14		1.05		0.67			0.39
Nouzha											
Fisherman's Village											
Nouzha											
Total	1.10	2.62	2.19	1.74	1.00	1.05		0.67			0.39

[a] For convenience, line sizes are listed in 100-mm increments only. Planned half sizes are placed in the next higher category.

Table M-4
LENGTH OF PRESSURE CONVEYANCE LINES FOR ALTERNATIVE IB, SEA DISPOSAL
(km)

	Pipeline Diameter (mm)												
	350	400	500	550	950	1000	1200	1450	1600	1750	1850	2000	2130
Inner West/Central													
P.S. 2 West				1.2									
Central P.S. to West STP							5.1						
West STP to Central P.S.													5.1
Central P.S. to Sea Outfall													2.0
Inner/Outer East													
Nemoura P.S.						1.7							
P.S. 1 East				1.6									
Sadat City P.S.						5.0							
P.S. 3 East						3.3							
P.S. 4 East		0.8											
P.S. 6 East	1.4												
Saouha P.S							7.1						
Abu Soliman P.S.												5.7	
Nouzha													
Fisherman's Village P.S.			1.7										
Nouzha P.S.	—	—	—	—	0.9	—	—					—	—
Total	1.4	0.8	1.7	2.8	0.9	10.0	12.2					5.7	7.1

Table M-5

LENGTH OF PRESSURE CONVEYANCE LINES FOR ALTERNATIVE II, SECONDARY TREATMENT

(km)

	Pipeline Diameter (mm)												
	350	400	500	550	950	1000	1200	1450	1600	1750	1850	2000	2130
Inner West/Central													
P.S. 2 West				1.2									
Central P.S. to West STP							5.1						
Inner/Outer East													
Mamoura P.S.						1.7							
P.S. 1 East				1.6									
Sadat City P.S.						5.0							
P.S. 3 East						3.3							
P.S. 4 East		0.8											
P.S. 6 East	1.4												
Sawuha P.S							7.1						
Abu Soliman P.S.												5.7	
Nouzha													
Fisherman's Village P.S.			1.7										
Nouzha P.S.	—	—	—	—	0.9	—	—					—	
Total	1.4	0.8	1.7	2.8	0.9	10.0	12.2					5.7	

Table M-6

LENGTH OF PRESSURE CONVEYANCE LINES FOR ALTERNATIVE III, LAND APPLICATION

(km)

	Pipeline Diameter (mm)												
	350	400	500	550	950	1000	1200	1450	1600	1750	1850	2000	2130
Inner West/Central													
P.S. 2 West				1.2									
Central P.S. to West P.S.							3.9						
Inner/Outer East													
Mamoura P.S.						1.7							
P.S. 1 East				1.6									
Sadat City P.S.						6.3							
P.S. 3 East						3.8							
Smouha P.S							1.4						
Ras El-Soda P.S.									1.9				
Nouzha													
Fisherman's Village P.S.			1.7										
Nouzha					0.9								
Desert[a]													
West P.S. (two lines)								66.0					
East P.S. (two lines)										76.0			
Desert P.S. (three lines)											132.0		
Total			1.7	2.8	0.9	11.8	5.3	66.0	1.9	76.0	132.0		

[a]Lengths of Desert zone lines are total length of parallel lines shown.

Table M-7
PUMP STATIONS FOR ALTERNATIVE IB, SEA DISPOSAL

Name	Type[a]	Capacity (Ml/day) Year 2000 Avg.	Year 2000 Peak	Year 2030 Avg.	Year 2030 Peak
Inner West/Central					
P.S. 2 West	R	16	27	22	37
Industries P.S.	R	42	67	46	74
Central P.S.[b]	N	a.196	292	241	354
		b.395	605	490	740
Inner/Outer East					
Mamoura P.S.	R	43	74	66	110
P.S. 1 East	R	11	23	15	30
P.S. 3 East	R	52	88	69	114
P.S. 4 East	R	7	14	9	19
P.S. 6 East	R	6	14	7	16
Sadat City P.S.	N	60	99	74	120
Smouha P.S.	N	146	227	185	282
Abu Sollman P.S.	N	211	324	328	485
Ras El-Soda P.S.	N	136	209	184	276
Nouzha					
Fisherman's Village P.S.	N	0	0	15	30
Nouzha P.S.	N	45	70	77	125

[a]Legend: N = new construction
R = rebuild existing

[b]Central pump station is a dual pump station in this alternative
a. Pumps central zone flows to West STP
b. Pumps West plant effluent to sea outfall

Table M-8

PUMP STATIONS FOR ALTERNATIVE II, SECONDARY TREATMENT

Name	Type[a]	Capacity (Ml/day) Year 2000 Avg.	Year 2000 Peak	Year 2030 Avg.	Year 2030 Peak
Inner West/Central					
P.S. 2 West	R	16	27	22	37
Industries P.S.	R	42	67	46	74
Central P.S.	N	196	292	241	354
Inner/Outer East					
Mamoura P.S.	R	43	74	66	110
P.S. 1 East	R	11	23	15	30
P.S. 3 East	R	52	88	69	114
P.S. 4 East	R	7	14	9	19
P.S. 6 East	R	6	14	7	16
Sadat City P.S.	N	60	99	74	120
Smouha P.S.	N	146	227	185	282
Abu Soliman P.S.	N	211	324	328	485
Ras El-Soda P.S.	N	136	209	184	276
Nouzha					
Fisherman's Village P.S.	N	0	0	15	30
Nouzha P.S.	N	45	70	77	125

[a]Legend: N = new construction
R = rebuild existing

Table M-9
PUMP STATIONS FOR ALTERNATIVE III, LAND APPLICATION

Name	Task[a]	Capacity (Ml/day) Year 2000 Avg.	Year 2000 Peak	Year 2030 Avg.	Year 2030 Peak
Inner West/Central					
P.S. 2 West	R	16	27	22	37
Industries P.S.	R	42	67	46	74
Central P.S.	N	196	292	241	354
West P.S.	N	395	605	490	710
Inner/Outer East					
Mamoura P.S.	R	43	74	66	110
P.S. 1 East	R	11	23	15	30
P.S. 3 East	R	52	88	69	114
P.S. 4 East	R	7	14	9	19
P.S. 6 East	R	12	25	16	31
Sadat City P.S.	N	60	99	74	120
Smouha P.S.	N	146	227	185	282
Ras El-Soda P.S.	N	125	194	170	256
East P.S.	N	585	895	825	1,168
Nouzha					
Fisherman's Village P.S.	N	0	0	15	30
Nouzha P.S.	N	45	70	77	125
Desert					
Desert P.S.	N	980	1,275[b]	1,300	1,700[b]

[a] Legend: N = new construction
R = rebuild existing

[b] Peak pumping capacity for Desert pump station is maximum day flow. Storage is provided for peak hour flows.

Table M-10
ALTERNATIVE 1B CONVEYANCE SYSTEM
CAPITAL CONSTRUCTION COSTS
(1000 LE)

Inner West/Central			
Pump stations	12,382		
Gravity lines	8,119		
Pressure lines			20,603
Subtotal	20,501		20,603
Inner/Outer East			
Pump stations	20,517		
Gravity lines	20,561		
Pressure lines			26,966
Subtotal	41,078		26,966
Nouzha			
Pump stations	2,258		
Gravity lines	2,282		
Pressure lines			1,182
Subtotal	4,540		1,182
Alternative 1B Subtotal	66,119		48,751
Contingency[a]	16,530		9,750
Subtotal	82,649		58,501
Engineering, Legal and Administration[b]	12,397		8,775
Subtotal	95,046		67,276
Alternative 1B Total		162,322	

[a]Contingency for pump stations and gravity lines is 25%; contingency for pressure lines is 20%.

[b]Engineering, legal, and administration is 15%.

Table M-11
ALTERNATIVE II CONVEYANCE SYSTEM
CAPITAL CONSTRUCTION COSTS
(1000 LE)

Inner West/Central			
Pump stations	7,244		
Gravity lines	8,119		
Pressure lines			9,562
Subtotal	15,363		9,562
Inner/Outer East			
Pump stations	20,517		
Gravity lines	20,561		
Pressure lines			26,965
Subtotal	41,078		26,965
Nouzha			
Pump stations	2,258		
Gravity lines	2,282		
Pressure lines			1,182
Subtotal	4,540		1,182
Alternative II Subtotal	60,981		33,709
Contingency[a]	15,245		6,742
Subtotal	76,226		40,451
Engineering, Legal and Administration[b]	11,434		6,068
Subtotal	87,660		46,519
Alternative II Total		134,179	

[a] Contingency for pump stations and gravity lines is 25%; contingency for pressure lines is 20%.

[b] Engineering, legal, and administration is 13%.

Table M-12
ALTERNATIVE III CONVEYANCE SYSTEM
CAPITAL CONSTRUCTION COSTS
(1000 LE)

Inner West/Central			
Pump stations	16,145		
Gravity lines	9,589		
Pressure lines			4,387
Subtotal	25,734		4,387
Inner/Outer East			
Pump stations	27,632		
Gravity lines	24,174		
Pressure lines			14,056
Subtotal	51,806		14,056
Nouzha			
Pump stations	2,258		
Gravity lines	2,282		
Pressure lines			1,182
Subtotal	4,540		1,182
Desert			
Pump station	19,930		
Pressure lines			336,215
Subtotal	19,930		336,215
Alternative III Subtotal	102,010		355,840
Contingency	25,503		71,168
Subtotal	127,513		427,008
Engineering, Legal and Administration[b]	19,127		64,051
Subtotal	146,640		491,059
Subtotal		637,699	
Right-of-way		2,530	
Alternative III Total		640,229	

[a]Contingency for pump stations and gravity lines is 25%; contingency for pressure lines is 20%.

[b]Engineering, legal, and administration is 15%.

Table H-13
ALTERNATIVE IB CONVEYANCE SYSTEM
O&M COSTS

	Costs (10^3 LE)		
	1980	1990	2000
Inner West/Central			
Pump Stations			
Labor (9,231 mandays)	60.0	60.0	60.0
Material (complete)	246.3	246.3	246.3
Power[a]	230.2	245.5	276.0
Pressure Lines			
Labor (1,908 mandays)	12.4	12.4	12.4
Material (complete)	24.7	24.7	24.7
Gravity Lines			
Labor (4,692 mandays)	30.5	30.5	30.5
Material (complete)	20.3	20.3	20.3
Subtotal	624.4	639.7	670.2
Inner/Outer East			
Pump Stations			
Labor (28,615 mandays)	186.0	186.0	186.0
Material (complete)	293.1	293.1	293.1
Power[b]	206.1	279.2	315.9
Pressure Lines			
Labor (2,492 mandays)	16.2	16.2	16.2
Material (complete)	32.4	32.4	32.4
Gravity Lines			
Labor (11,862 mandays)	77.1	77.1	77.1
Material (complete)	51.4	51.4	51.4
Subtotal	862.3	935.4	972.1
Nouzha			
Pump Stations			
Labor (2,769 mandays)		18.0	18.0
Material (complete)		29.2	29.2
Power[c]		24.2	28.0
Pressure Lines			
Labor (108 mandays)		0.7	0.7
Material (complete)		1.4	1.4
Gravity Lines			
Labor (1,323 mandays)		8.6	8.6
Material (complete)		5.7	5.7
Subtotal		87.8	91.6
Total O&M Cost	1,486.7	1,662.9	1,733.9

[a]Power costs are based on the following estimated consumption:
1980--19.6 million kWh
1990--22.4 million kWh
2000--28.0 million kWh

[b]Power costs are based on the following estimated consumption:
1980--1[illegible] million kWh
1990--23.2 million kWh
2000--28.9 million kWh

[c]Power costs are based on the following estimated consumption:
1990--1.6 million kWh
2000--1.9 million kWh

Table M-14
ALTERNATIVE II CONVEYANCE SYSTEM
O&M COSTS

	Costs (10^3 LE)		
	1980	1900	2000
Inner West/Central			
Pump Stations			
Labor (9,231 mandays)	60.0	60.0	60.0
Material (complete)	137.3	137.3	137.3
Power[a]	83.3	102.8	116.2
Pressure Lines			
Labor (508 mandays)	3.3	3.3	3.3
Material (complete)	6.7	6.7	6.7
Gravity Lines			
Labor (4,692 mandays)	30.5	30.5	30.5
Material (complete)	20.3	20.3	20.3
Subtotal	341.4	360.9	374.3
Inner/Outer East			
Pump Stations			
Labor (28,615 mandays)	186.0	186.0	186.0
Material (complete)	293.1	293.1	293.1
Power[b]	206.1	279.2	315.9
Pressure Lines			
Labor (2,492 mandays)	16.2	16.2	16.2
Material (complete)	32.4	32.4	32.4
Gravity Lines			
Labor (11,862 mandays)	77.1	77.1	77.1
Material (complete)	51.4	51.4	51.4
Subtotal	862.3	935.4	972.1
Nouzha			
Pump Stations			
Labor (2,769 mandays)		18.0	18.0
Material (complete)		29.2	29.2
Power[c]		24.2	28.0
Pressure Lines			
Labor (108 mandays)		0.7	0.7
Material (complete)		1.4	1.4
Gravity Lines			
Labor (1,323 mandays)		8.6	8.6
Material (complete)		5.7	5.7
Subtotal		87.8	91.6
Total O&M Cost	1,203.7	1,384.1	1,438.0

[a]Power costs are based on the following estimated consumption:
1980--5.1 million kWh
1990--8.8 million kWh
2000--11.0 million kWh

[b]Power costs are based on the following estimated consumption:
1980--11.6 million kWh
1990--23.2 million kWh
2000--28.9 million kWh

[c]Power costs are based on the following estimated consumption:
1990--1.6 million kWh
2000--1.9 million kWh

Table M-15
ALTERNATIVE III CONVEYANCE SYSTEM
O&M COSTS

	Costs (10^3 LE)		
	1980	1900	2000
Inner West/Central			
Pump Stations			
Labor (13,231 mandays)	86.0	86.0	86.0
Material (complete)	324.3	324.3	324.3
Power[a]	349.2	[illegible]	499.1
Pressure Lines			
Labor (400 mandays)	2.6	2.6	2.6
Material (complete)	5.3	5.3	5.3
Gravity Lines			
Labor (5,985 mandays)	38.9	38.9	30.9
Material (complete)	26.0	26.0	26.0
Subtotal	832.3	908.9	982.2
Inner/Outer East			
Pump Stations			
Labor (32,615 mandays)	212.0	212.0	212.0
Material (complete)	628.7	628.7	628.7
Power[b]	337.9	616.4	793.6
Pressure Lines			
Labor (1,292 mandays)	8.4	8.4	8.4
Material (complete)	16.9	16.9	16.9
Gravity Lines			
Labor (13,954 mandays)	90.7	90.7	90.7
Material (complete)	60.4	60.4	60.4
Subtotal	1,355.0	1,633.5	1,810.7
Nouzha			
Pump Stations			
Labor (2,769 mandays)		18.0	18.0
Material (complete)		29.2	29.2
Power[c]		24.2	28.0
Pressure Lines			
Labor (108 mandays)		0.7	0.7
Material (complete)		1.4	1.4
Gravity Lines			
Labor (1,323 mandays)		8.6	8.6
Material (complete)		5.7	5.7
Subtotal		87.8	91.6

[a]Power costs are based on the following estimated consumption:
1980--35.2 million kwh
1990--45.6 million kwh
2000--54.6 million kwh

[b]Power costs are based on the following estimated consumption:
1980--28.9 million kwh
1990--61.8 million kwh
2000--83.4 million kwh

[c]Power costs are based on the following estimated consumption:
1990--1.6 million kwh
2000--1.9 million kwh

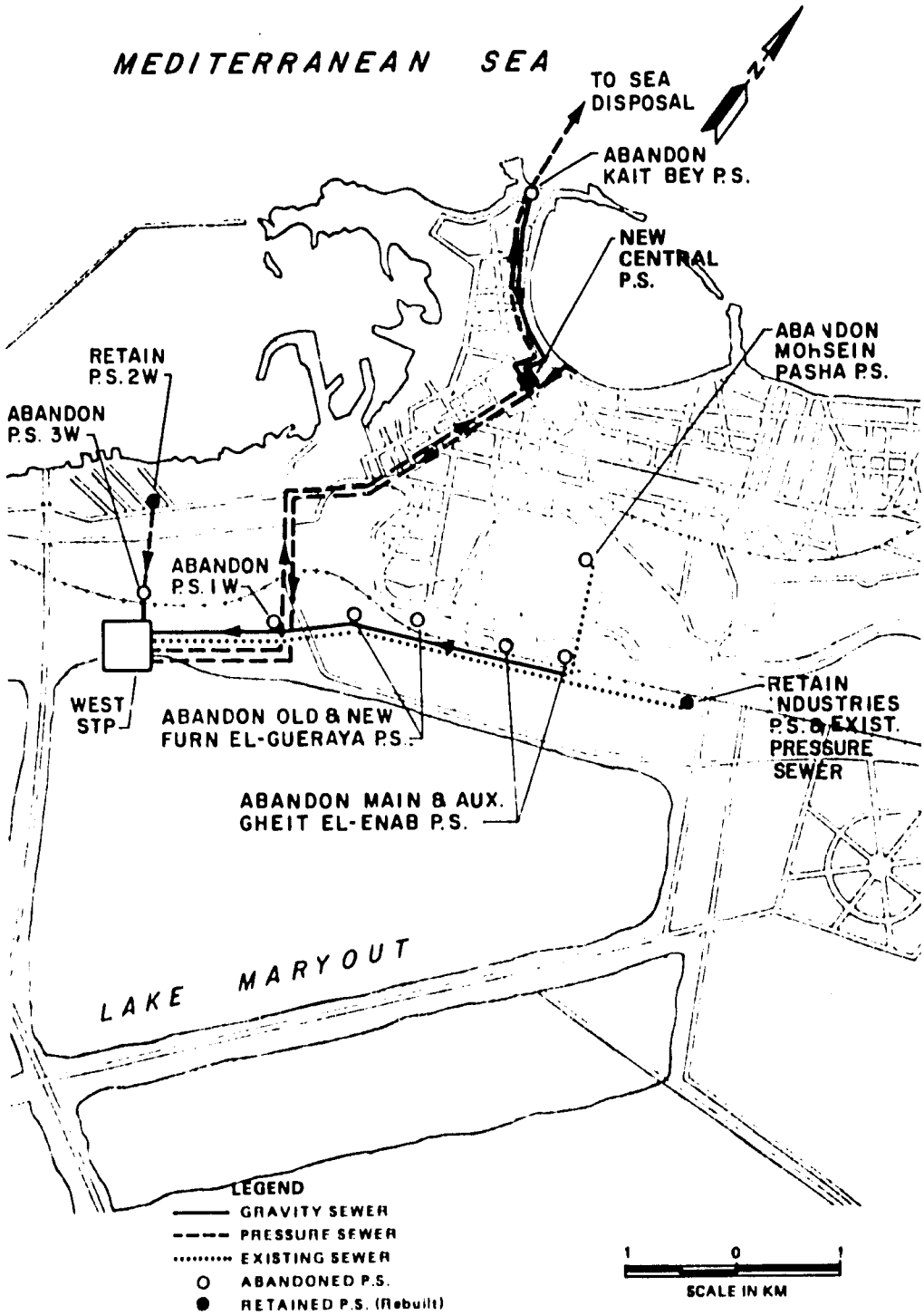
Table M-15 (continued)
ALTERNATIVE III CONVEYANCE SYSTEM
O&M COSTS

	Costs (10^3 LE)		
	1980	1900	2000
Desert			
Pump Stations			
Labor (4,000 mandays)	26.0	26.0	26.0
Material (complete)	310.0	310.0	310.0
Power[d]	426.0	687.0	885.0
Pressure Lines			
Labor (23,846 mandays)	155.0	155.0	155.0
Material (complete)	230.0	230.0	230.0
Subtotal	1,147.0	1,408.0	1,606.0
Total O&M Cost	3,334.3	4,038.2	4,[illegible]90.5

[d]Power costs are based on the following estimated consumption:
1980--51.2 million kWh
1990--83.1 million kWh
2000--107.5 million kWh

Table M-16
SUMMARY OF TOTAL CONVEYANCE SYSTEM COSTS
(1000 LE)

Alternative IB	
Pump stations	50,538
Pressure lines	67,276
Gravity lines	44,508
Total capital cost	162,322
O&M cost (year 2000)	1,734
Alternative II	
Pump stations	43,152
Pressure lines	46,519
Gravity lines	44,508
Total capital cost	134,179
O&M cost (year 2000)	1,438
Alternatives IIIA and IIIB	
Pump stations	94,825
Pressure lines	491,059
Gravity lines	51,815
Total capital cost	637,699
Right-of-way cost	2,530
O&M cost (year 2000)	4,491



1978 ALEXANDRIA WASTEWATER MASTER PLAN
REVIEW AND UPDATE — MAY 1981

FIGURE M-1

ALTERNATIVE IB
INNER WEST/CENTRAL ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

637

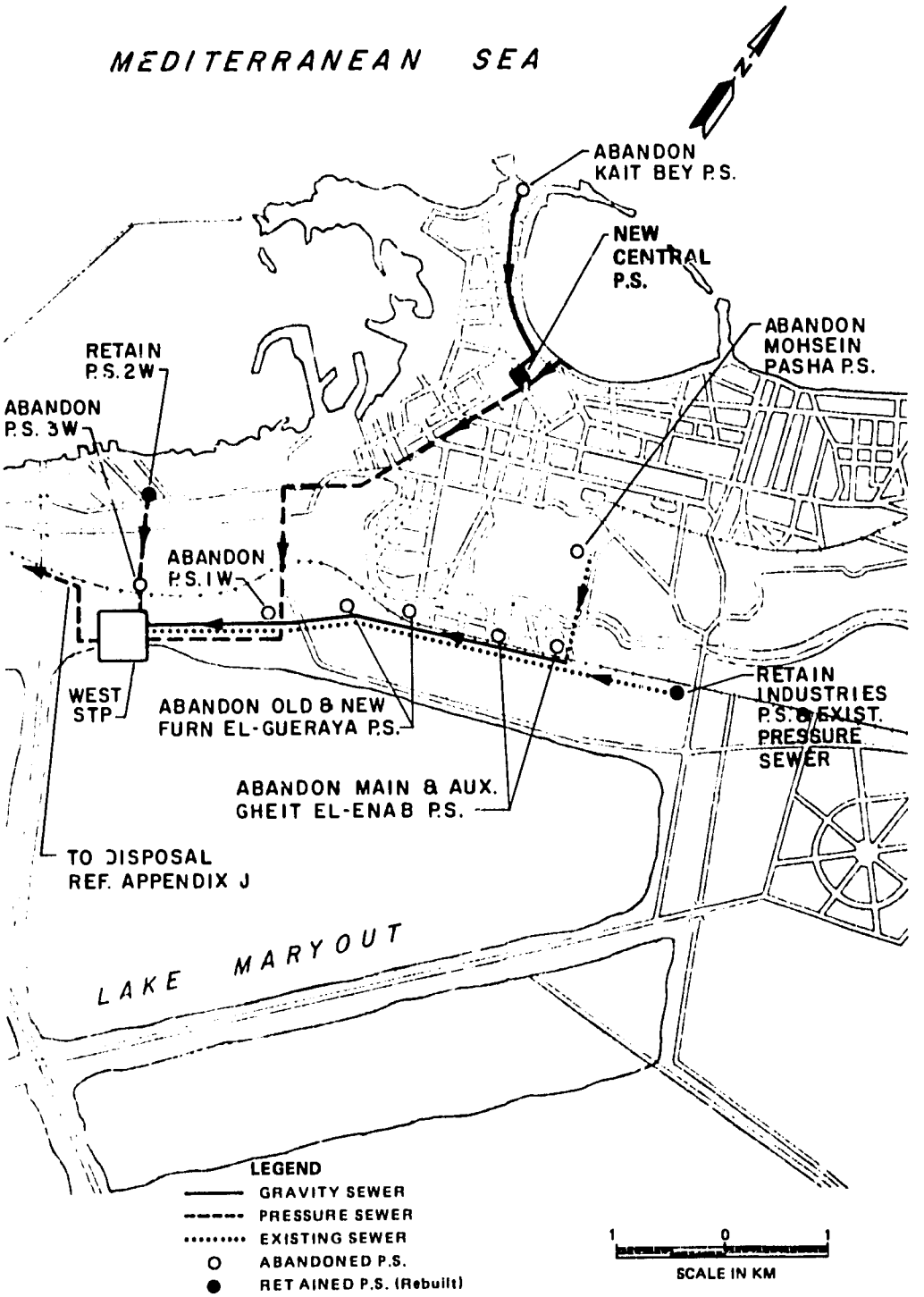


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FIGURE M-2

ALTERNATIVE II
INNER WEST/CENTRAL ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

688

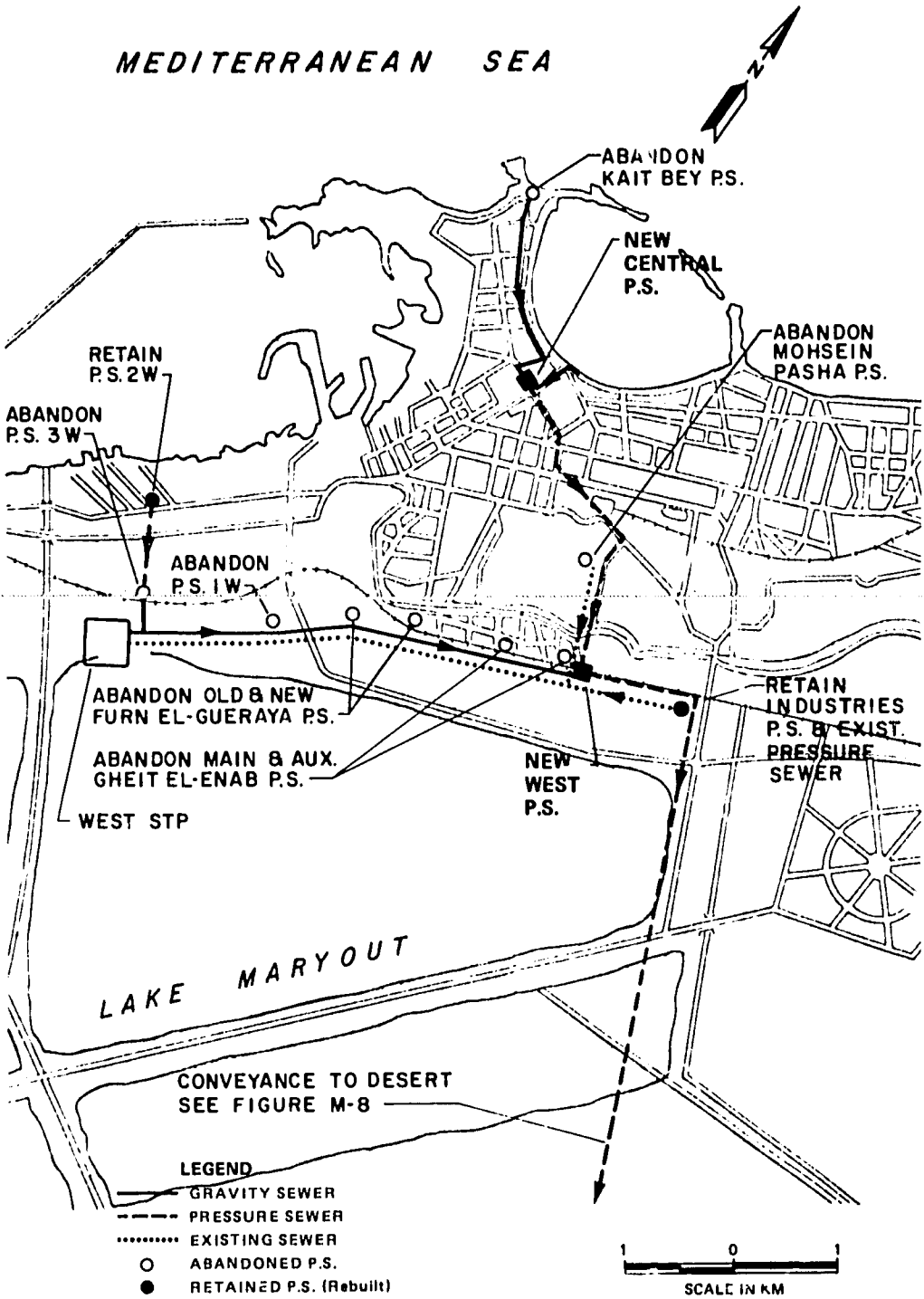

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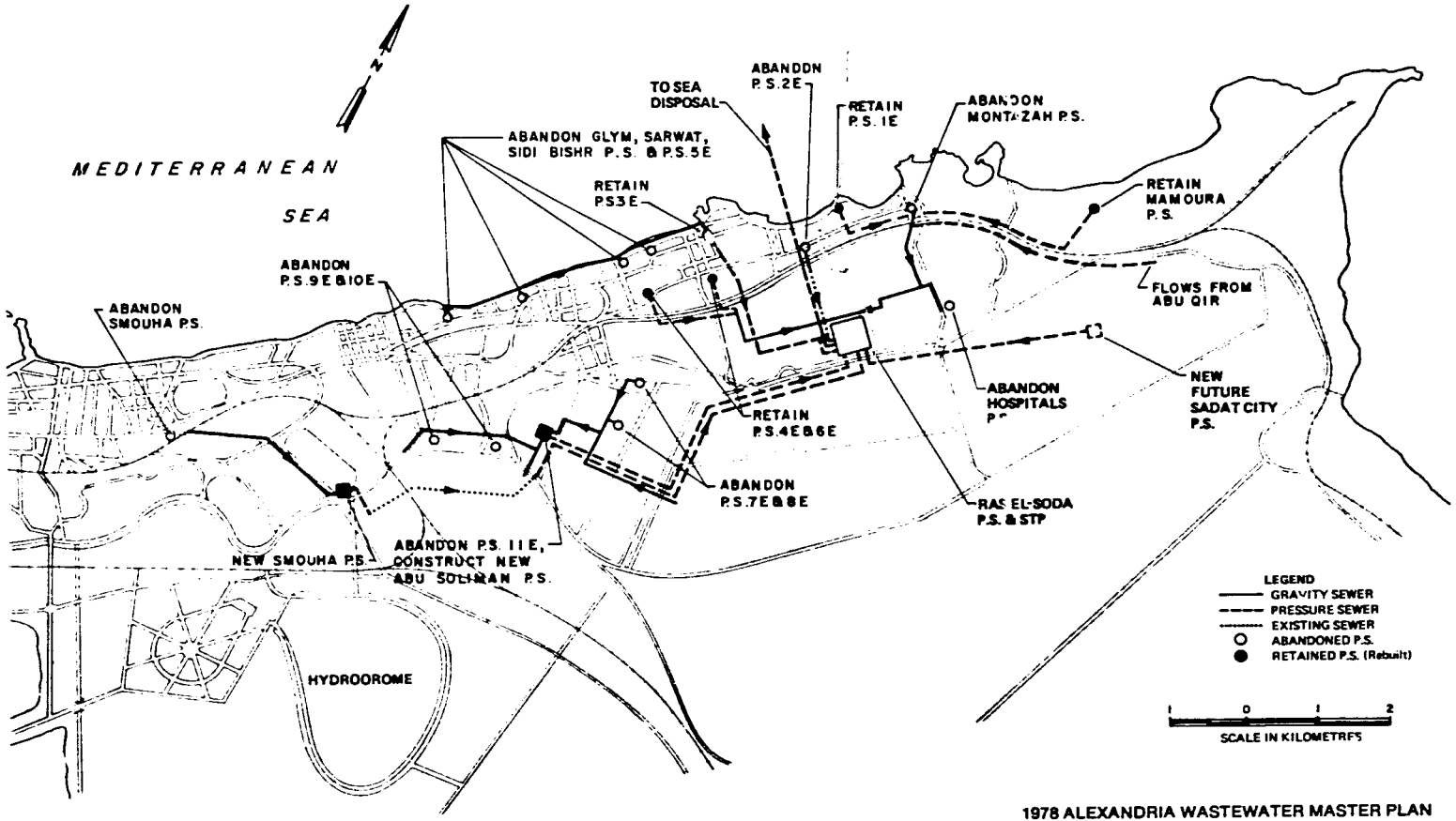
FIGURE M-3

ALTERNATIVE III
INNER WEST/CENTRAL ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

689



NOTE:
PUMP STATION NOS. 2E, 5E, MONTAZAH AND THE SIDI GABER PUMP STATION WILL BE RETAINED FOR STORMWATER PUMPING.

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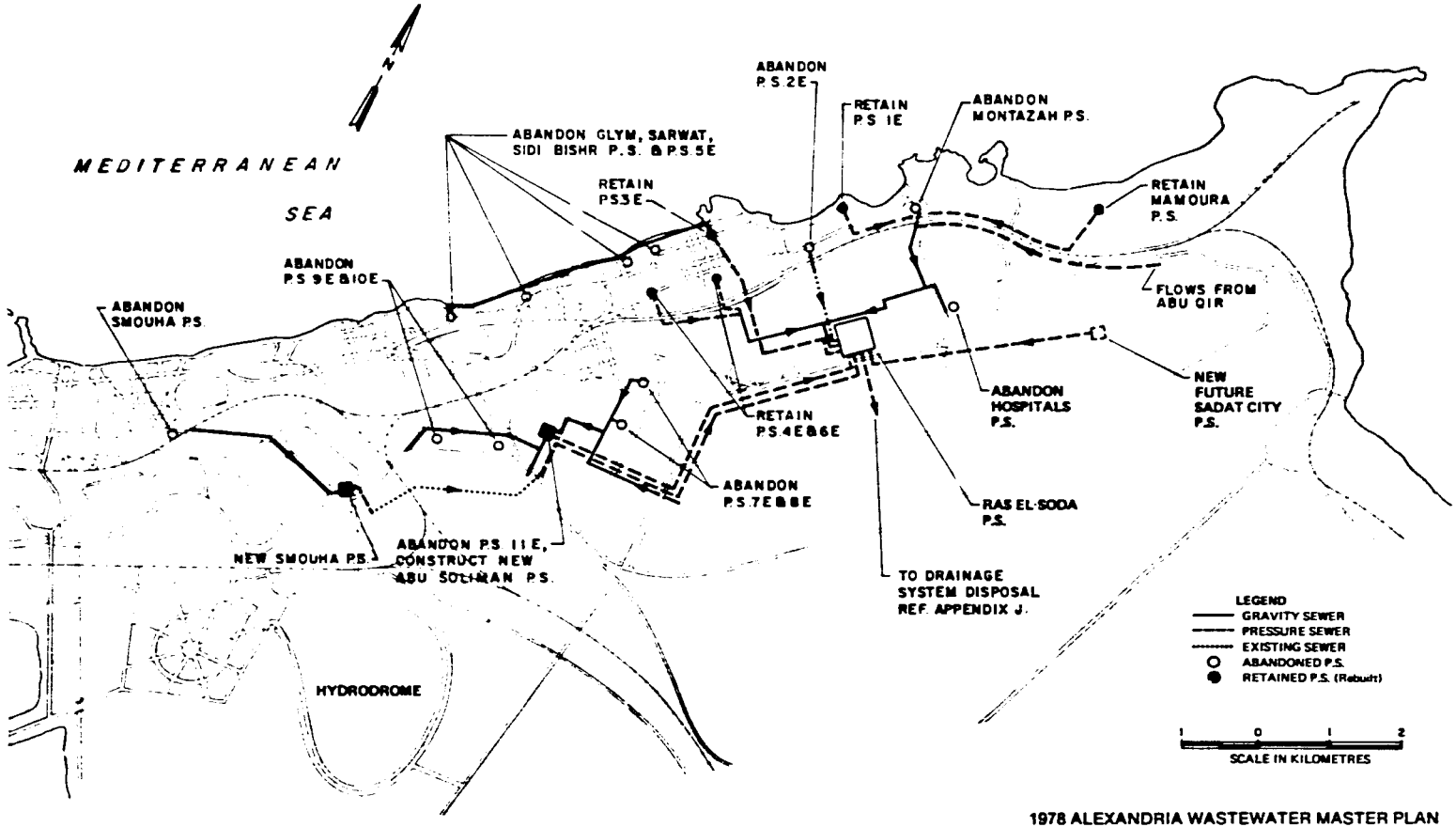
FIGURE M-4

ALTERNATIVE IB
INNER/OUTER EAST ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG



NOTE:
PUMP STATION NOS. 2E, 5E, MONTAZAH AND THE SIDI GABER PUMP STATION WILL BE RETAINED FOR STORMWATER PUMPING.

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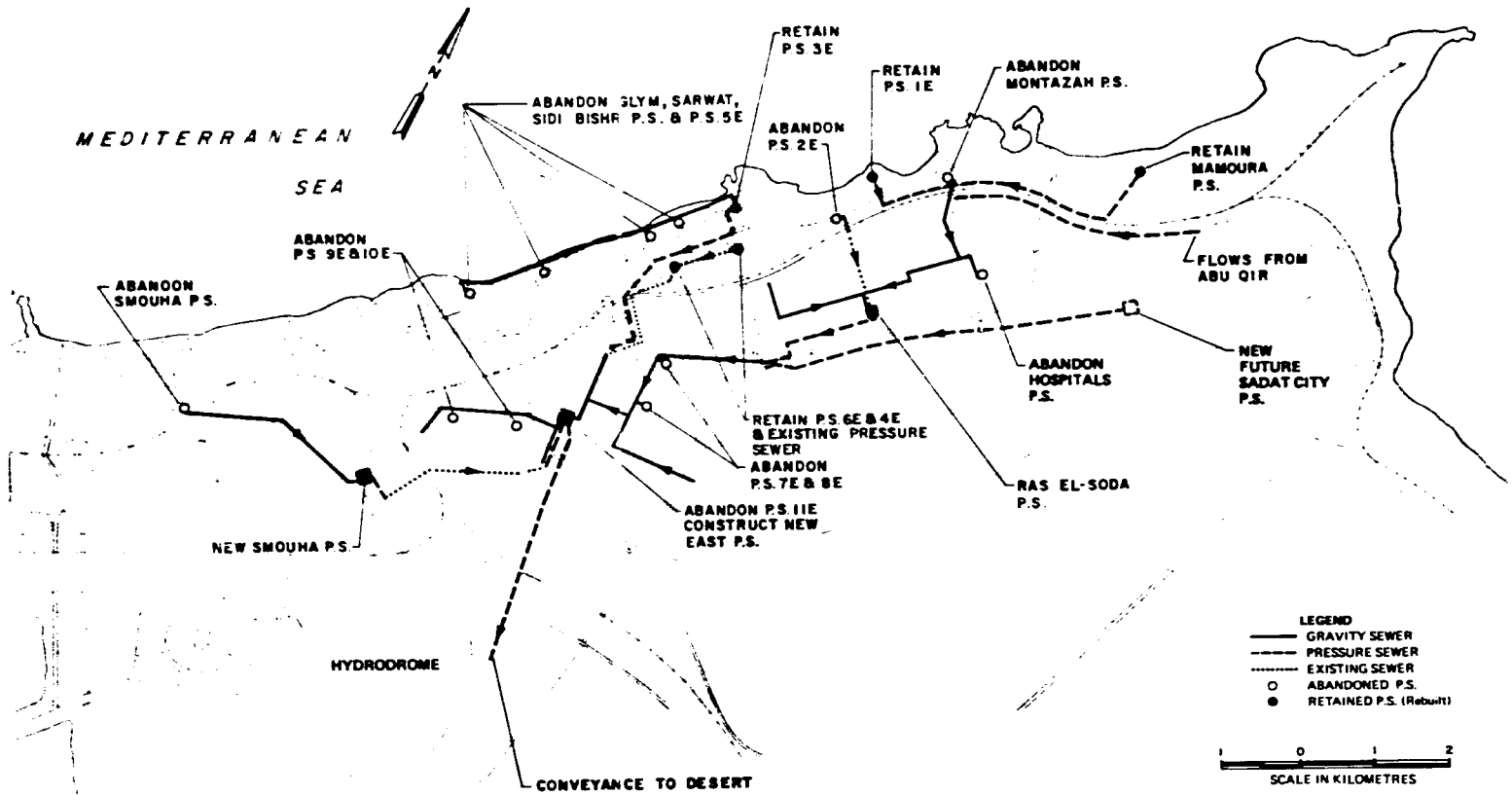
REVIEW AND UPDATE — MAY 1981

FIGURE M-5

ALTERNATIVE II
INNER/OUTER EAST ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG



NOTE:
PUMP STATION NOS. 2E, 5E, MONTAZAH AND THE SIDI GABER PUMP STATION WILL BE RETAINED FOR STORMWATER PUMPING.

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FIGURE M-6

ALTERNATIVE III
INNER/OUTER EASTERN ZONES
CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

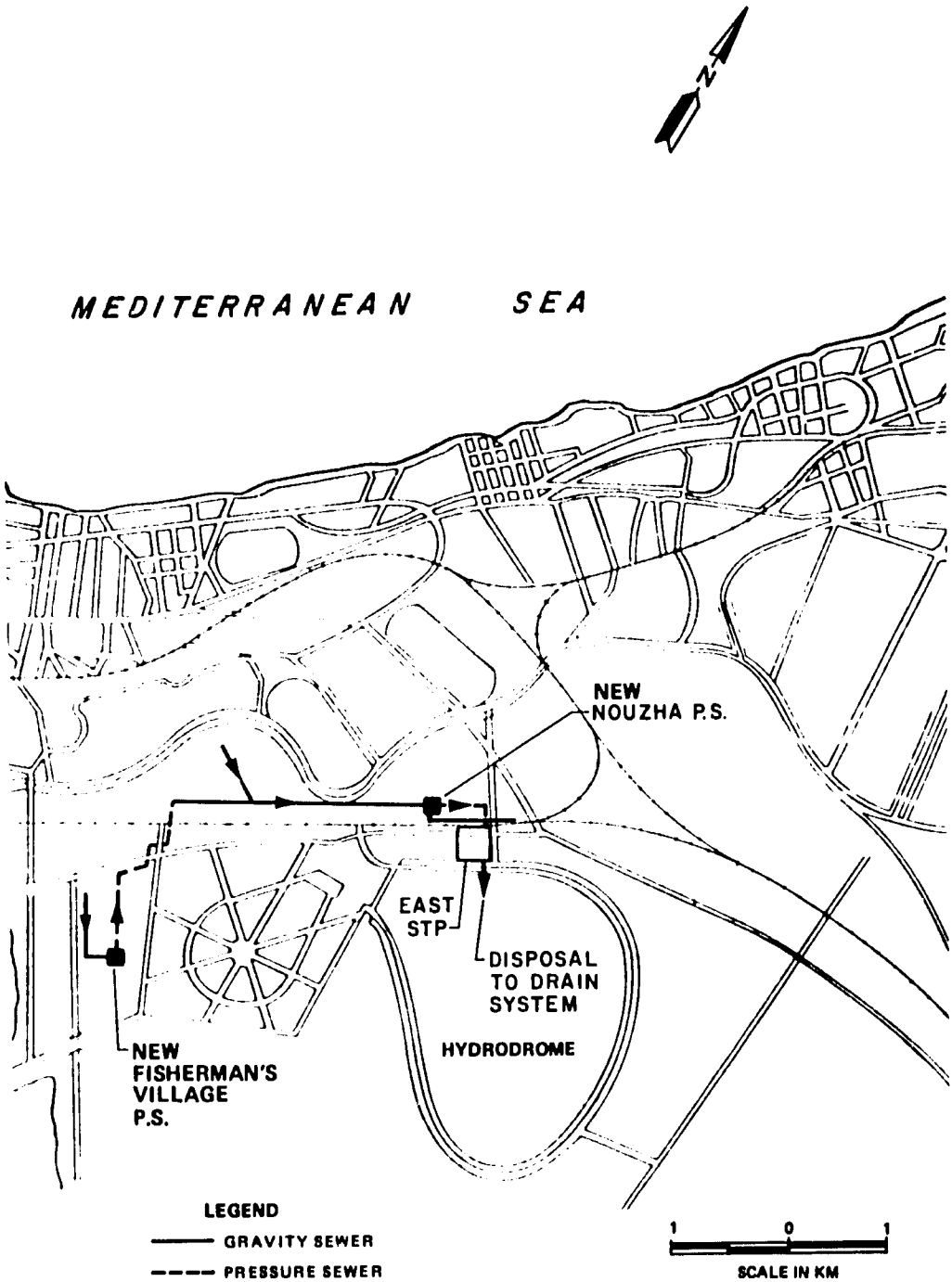

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FIGURE M-7

ALTERNATIVES IB, II, III
NOUZHA ZONE CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

693

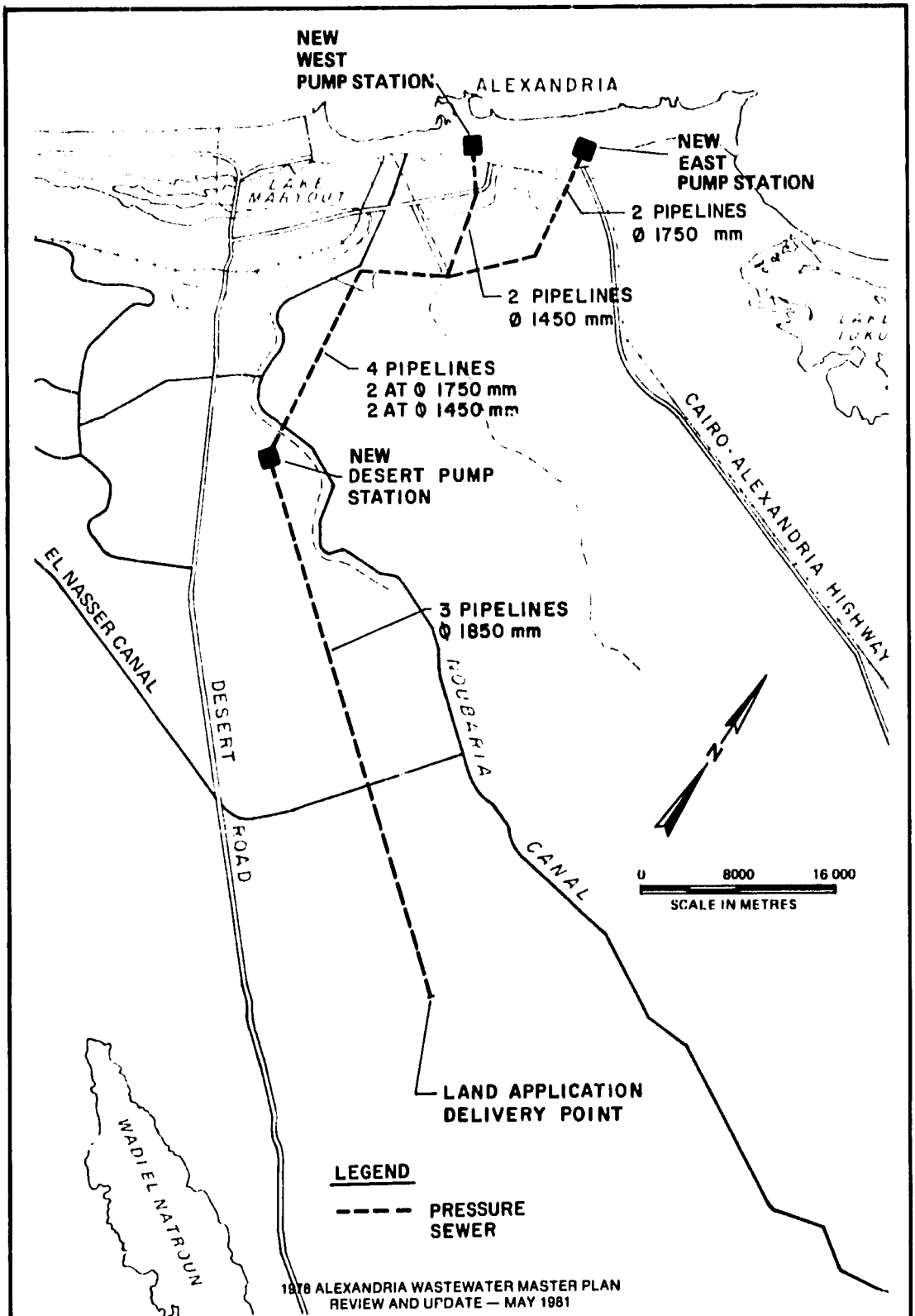

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FIGURE M-8

ALTERNATIVES IIIA AND IIIB
DESERT ZONE CONVEYANCE FACILITIES

A/GOSD WWCG W/ECG

694

APPENDIX N

ECONOMIC ANALYSIS

INTRODUCTION

An economic analysis was conducted to compare the three most attractive methods of wastewater disposal on the basis of cost-effectiveness. These three alternatives include: Alternative IB, Sea Disposal; Alternative IIIA, Land Application (wastewater treatment by anaerobic lagoons followed by rapid infiltration into a ground-water storage system); and Alternative IIIB, Land Application (wastewater treatment by anaerobic lagoons only). Under Alternative IIIA, the water quality would impose no restrictions on the types of crops to be grown. With Alternative IIIB, water quality would restrict the crops grown to feed and forage.

This appendix presents both the conclusions of the economic analysis and documentation for these conclusions. The analysis and conclusions are also summarized in Chapter 6 of the Report, Volume 1.

GENERAL

As discussed in Appendices I, J, K, and L, all three alternatives mentioned above are technically feasible and can provide the necessary protection to the public health and the environment. Unlike the sea disposal alternative, which provides only benefits associated with wastewater collection, treatment, and disposal, the land application alternatives would also provide direct monetary benefits from irrigation. Therefore, the project benefits are substantially higher for the land application alternatives than for the sea disposal alternative. However, the project costs are also substantially higher.

The economic analysis focused on whether the additional project costs for either Alternative IIIA or IIIB appear warranted in view of the additional benefits.

One method of analyzing direct monetary aspects of projects that exhibit differing costs and benefits is to determine the internal rate of return (IRR) generated by the projects. Simply stated, the IRR is the interest rate at which the present worth of the future flow of monetary benefits is equal to the present worth of the future flow of monetary costs. The IRR is therefore numerically equal to the present worth discount rate at which the ratio of benefits to costs would be 1.0. The intangible and indirect benefits also are important considerations. Using the wastewater for irrigation would help achieve the national goal of food

self-sufficiency, create about 1100 farm jobs, and result in some decentralization of the population through development of small villages needed to accommodate the farming and treatment operations. The final decision on whether the additional expenditures are warranted should be made on the basis of the availability of financing and potential opportunities foregone. If capital is limited, consideration should be given to whether society could gain greater benefits by investing available funds in other types of projects.

STUDY APPROACH

The purpose of the economic analysis was to provide Egyptian and USAID officials with a basis for deciding which wastewater disposal alternative would provide the greatest net benefit to the economy of Egypt as a whole. Therefore, opportunity costs and prices (often referred to as "shadow prices") were used in measuring benefits and costs. These shadow prices refect the real cost of production inputs and outputs to the national economy with the influence of market distortions removed. Examples of market distortions that were removed include artificially low prices for some crops caused by government controls, and subsidized prices for some farm production items such as fertilizers, fuel, and improved seeds. Energy costs in the economic analysis were based on marginal rates, which reflect the full cost of developing new sources of supply.

The economic analysis was based on a useful life for project facilities of 50 years. However, the structures were sized for the year 2000 design flow capacity.

Tangible benefits and costs were measured in monetary terms for each alternative. However, intangible benefits and costs also were important in comparing the alternatives. Therefore, they are described in narrative terms.

Data used in the analysis were obtained by a review of previous similar studies, published and unpublished information available from Egyptian government agencies, interviews with knowledgeable professionals, site investigations by the study team, and engineering cost estimates. The data are regarded as adequate for the purpose of updating the Master Plan. However, more detailed investigations will be required to confirm the economic feasibility of the selected alternative.

SEA DISPOSAL ALTERNATIVE

Project Benefits

A detailed description of Sea Disposal Alternative IB is presented in Appendix J. In summary, the plan includes treatment facilities at the West plant, East plant, and Ras El-Soda plant.

Project benefits are measured on the basis of the "with and without" principal. Future conditions with the project are compared with conditions anticipated to occur without the project. Net improvements are benefits attributable to the project.

For the most part, wastewater is now discharged untreated into area waterbodies, including the Mediterranean Sea and Lake Maryout. The exception has been brief periods when the East Treatment Plant has operated. The major discharge points are the Kait Bey outfall on the Mediterranean Sea, the Tabia pump station discharge to Abu Qir Bay, and local drains into the Western Harbor. Additionally, numerous local overflows are located along the Mediterranean shore. Finally, wastewater discharge into Lake Maryout is eventually released into the Western Harbor by the Mex pump station.

Future conditions without improvements in the wastewater facilities were considered as part of the "Environmental Data Base Review" presented in Appendix I. In summary, population, land uses, and economic development of the Alexandria region are expected to change dramatically even if improvements are not made to the existing wastewater facilities. Current projections indicate that the population will increase approximately 84 percent by the year 2000. A variety of residential, commercial, and industrial developments are expected to accommodate many of the future demands for housing and employment. Total wastewater flows in the Alexandria region (including the outer areas) are expected to increase from 480 Ml/day (sewered) in 1981 to about 1476 Ml/day in the year 2000.

The adverse environmental impacts associated with taking no action to improve the wastewater facilities of Alexandria are regarded as unacceptable. The public health consequence of increased population densities in nonsewered areas and increases in the discharges of untreated wastewater flows would be severe. The risks of contacting contaminated water would increase dramatically. In addition, the worsening of the beach pollution could have a significant adverse impact on the important tourism economy of the Alexandria region.

No attempt was made to measure the monetary benefits of the wastewater system improvements for Sea Disposal Alternative IB, primarily because

the public health and other nonmonetary benefits are of overriding importance. Furthermore, the monetary benefits from sea disposal of the effluent would also be achieved by the land application alternatives. Consequently, they are of no significance in the economic comparision of alternatives. Estimates of the incremental monetary benefits for the land application alternatives are estimated in a subsequent section of this appendix and are considered in the comparison of alternatives.

Sea Disposal Alternative IB would resolve the overall problem of wastewater collection, treatment, and disposal in Alexandria. The primary benefits would be in protecting the public health and general environment. These are discussed in Appendices J and L. Other related benefits include increased tourism, improved labor productivity because of better health, and local employment generated by facilities. Some benefits also would result from using dewatered solids as a fertilizer/soil conditioner on farms.

Project Costs

Project costs are for construction, operation, maintenance, and replacement of facilities. A schedule of these costs over the 50-year period of analysis is presented in Table N-1. The costs are based on price levels at the end of 1980. They represent economic costs excluding taxes and duties. The operation and maintenance costs in Table N-1 differ from those reported in other appendices and sections of the report. This is because marginal rather than average costs have been used in computing electric energy costs. The marginal rates reflect the full cost of developing new sources of energy supply.

Project Feasibility

Sea Disposal Alternative IB is feasible from a technical viewpoint and would provide the necessary protection to public health and the environment. The economic feasibility depends upon how this alternative compares with other available alternatives in terms of benefits and costs. This comparison of alternatives is presented in a subsequent section of this appendix.

LAND APPLICATION ALTERNATIVES

Project Benefits

Detailed descriptions of Land Application Alternatives IIIA and IIIB are presented in Appendix K. In summary, the land application alternatives consist of conveying wastewater to the desert, providing pretreatment, and using it to irrigate farm crops. The difference between Alternatives IIIA and IIIB lies in the level of pretreatment and the cropping pattern.

In addition to providing the same project benefits as discussed for the sea disposal alternative, the land application alternatives will provide tangible monetary benefits as a direct result of farming operations. These benefits from land application of the reclaimed water have been measured on the basis of a farm budget analysis. Direct irrigation benefits have been defined as the return to water (net farm income) that results from irrigation with the reclaimed water. The return to water is calculated as a residual by estimating gross crop income and deducting crop production costs (including a return to farm labor), returns on investment (6 percent), and return to management (1 percent of gross income).

The return to water was calculated for representative crops expected to be grown in the irrigation service area. The anticipated cropping pattern was then used as a basis for calculating the weighted average benefit on an annual basis.

Cropping Pattern. Consideration was given to available markets, anticipated prices and yields, costs of production and net income, soil and drainage conditions, and wastewater quality in projecting the cropping patterns for lands irrigated under Alternatives IIIA and IIIB. For purposes of this study, a cropping pattern consisting mainly of vegetables, grain, and beans was assumed for Alternative IIIA. During the first 2 years of project operation, the land will be reclaimed by using a crop rotation including legumes and grains. In subsequent years, vegetables and grain would be the principal crops. Many other high value crops also could be grown and should be evaluated prior to design. These include timber, fiber, sugar, and oil crops.

For Alternative IIIB, the cropping pattern would be limited to forage, fiber, and seed crops primarily because of public health considerations in irrigating with primary level effluent. The cropping pattern assumed in this study consists of equal areas of water-tolerant grasses and alfalfa.

Markets. Markets are adequate to handle the crop production that would occur from either land application alternative. Egypt has been transformed from a net exporter of agricultural products to a large importer of many basic foods as a result of a rapidly growing population and a relativey low rate of growth in agricultural production. In the 3-year period 1977-79, imports of wheat, corn, beans, lentils, vegetable oil, sugar, red meat, and poultry meat made up 42 percent of the total supply of the nine basic food groups on a quantity basis (Ref. 1). For other crops normally exported, such as rice and oranges, increased domestic demands have restricted export availabilities.

Crop Yields. The yields at full production for some of the crops likely to be grown under Alternative IIIA and IIIB are presented in Table N-2.

Crop yields in the first year of irrigation will amount to only 60 percent of those attained at full production. Full production will be attained over a 5-year development period.

Yields are the same as those reported in Table K-27 of Appendix K, Land Application of Wastewater. Subsequent to the preparation of Appendix K, the yields were reviewed and verified to be reasonable on the basis of additional information provided by Mr. Yehia Mohieldin, Undersecretary for Economic Affairs, Ministry of Agriculture.

Prices. The prices of many farm products and inputs are influenced by government controls and other interventions. Government programs are designed to provide basic foods at subsidized prices affordable by the poorest Egyptians (Ref. 2). The government also has established food security as a priority national goal. Egypt is concerned with food security because its agricultural production falls far short of its needs and because rising world commodity prices place a burden on its balance of payments (Ref. 3).

Most international lending institutions, such as USAID and the World Bank, require the use of shadow prices in the economic analysis of projects whenever government controls, subsidies, taxes, and other interventions cause significant distortions in market prices. Shadow prices are measures of the "true" values of project inputs and outputs with the influence of market distortions removed.

The prices of the inputs and outputs for each of the Alexandria wastewater management alternatives were reviewed to determine whether market prices were sufficiently distorted to warrant using shadow prices. This review determined that the prices of many farm products are significantly distorted. For example, the market price of wheat is substantially less than the free market price. Furthermore, the market prices of some production inputs such as fertilizer are kept at levels to encourage their use as a means of increasing production.

Shadow prices for some crops can be computed on the basis of projections prepared by the Commodities and Export Projections Division of the World Bank (Ref. 7). Some of the data for the shadow price analysis were obtained from a World Bank economist who recently conducted an appraisal of a new lands irrigation project in a nearby area (Ref. 8). Data also were obtained from numerous Egyptian and U.S. government sources, and from studies by other consultants (Ref. 1, 2, 3, 4, 5, and 6).

Comparisons of market prices with shadow prices for several of the project outputs and inputs used in the economic comparison of the project alternatives are presented in Table N-3. They were adjusted to price levels at the end of 1980 to be consistent with the project cost esti-

mates. Shadow prices are listed only if they differ significantly from anticipated market prices. A description of how these shadow prices were derived is presented in the following paragraphs.

The production of wheat or other cereals in the service area would reduce the amount of imports for these crops. Therefore, the shadow price for wheat reflects the value to the Egyptian economy of producing this crop in the project area rather than importing it from the United States. The computations of shadow prices at the farmgate are presented below:

Price FOB U.S. Gulf Coast per tonne of wheat	$236		
Freight and insurance	25		
CIF Alexandria	$261	=	LE 182.70
Losses (6%)			LE 10.96
Handling (4.5%)			LE 8.22
			LE 201.88
Transportation from project area to Alexandria			- LE 2.75
Farmgate shadow price			LE 199.13
Rounded			LE 200.00

The 1990 world market price for wheat, expressed in end-of-1980 constant U.S. dollars, is projected by the World Bank to be $236 per tonne FOB U.S. Gulf Coast. The freight and insurance cost from the U.S. Gulf Coast to Alexandria is estimated at $25 per tonne, yielding a price CIF Alexandria of $261 per tonne. At a foreign exchange rate of LE .70 = $1.00 US, this amounts to LE 182.70 per tonne. Losses and handling costs add another LE 10.96 and LE 8.22 per tonne, respectively, to the price of delivering wheat to the consumers, who are assumed to be located in Alexandria. This results in a delivered cost to the consumer of LE 201.88 per tonne. The delivery of wheat produced in the project area to consumers would cost LE 2.75 per tonne. Therefore, the shadow price at the farmgate would be LE 199.13 per tonne, which was rounded to LE 200 per tonne.

The shadow price for potatoes, the representative vegetable crop, was computed on the basis of the assumption that 25 percent of the production from the project will be exported and 75 percent consumed domesti-

cally. The shadow price for the export portion was estimated as follows:

Year	Export Value (10^3 LE)	Quantity Exported (tonne)	Export Price (LE/tonne)	Exch. Rate Adj. Factor	Exch. Rate Adj. Amount (LE/tonne)	Inflation Adj. Factor	Inflation Adj. Amount (LE/tonne)
1977	16	166,000	96.39	$\frac{2.56}{1.43}$	172.56	1.45	250.21
1978	6	98,000	61.22	$\frac{2.56}{1.43}$	109.60	1.25	137.00
1979	19	112,000	169.64	$\frac{1.43}{1.43}$	169.64	1.10	186.60
Average Price (weighted by quantity)							201.74
Delivery from Project Area							39.74
Farmgate Shadow Price (export portion)							162.00

The farmgate price for potatoes consumed domestically was calculated on the basis of a weighted 5-year average of wholesale Egyptian prices at LE 105 per tonne, adjusted for inflation to price levels at the end of 1980. A market commission of 4 percent of wholesale price, trader margin of 5 percent of net wholesale, and transportation charge of LE 2.75 per tonne were deducted from the wholesale price to obtain the farmgate price of LE 93 per tonne.

The average potato price for use in the project analysis was computed as a weighted average of the export-derived farmgate price (LE 162/tonne) and the domestic derived farmgate price (LE 93/tonne). This amounts to LE 110 per tonne.

Computing the shadow price of potatoes as an average of the export and domestic farmgate prices is a conservative approach. An argument could be made for applying the export derived farmgate price of LE 162 per metric ton to the entire production. Potatoes are an export product in Egypt and a substantial portion of any increased production is likely to contribute either directly or indirectly to the level of exports. In addition, since Egypt is becoming increasingly dependent on food imports to meet domestic needs, the local consumption of potatoes is likely to increase, and this could indirectly reduce the level of other food crop imports. Further consideration of these crop substitution relationships and their influence on shadow prices should be made in any subsequent feasibility studies.

The fertilizer requirements for most crops would be met by nutrients in the reclaimed water and the application of sludge. However, it is anticipated that some supplemental potash will be needed to meet the fertilizer requirements for potatoes and alfalfa. Therefore, a shadow price was calculated for potash (60 percent potassium).

Price FOB Vancouver per Tonne of Potash	$109	
Freight and Insurance	30	
CIF Alexandria	$139	LE 97.30
Port Handling		LE 8.20
Transportation to Project Area		LE 2.75
Farmgate Shadow Price		LE 108.25
Rounded		LE 108/tonne

The world market price for potash, expressed in end-of-1980 constant U.S. dollars, was adjusted for freight and insurance differentials to the CIF Alexandria price. Port handling and transportation costs to the project area were added to derive the farmgate shadow price of LE 108 per tonne.

The shadow price for fuel is based on the world market price for crude oil of $30 per barrel at end-of-1980 price levels. On this basis, the farmgate price for diesel oil was computed as LE .161 per litre.

The shadow price for wheat seed of LE 233 per tonne was computed on the basis of the shadow price for wheat (LE 200 per tonne) plus the cost of cleaning and treating (LE 33 per tonne).

The shadow price for electricity was computed as 56 Mills/kWh on the basis of the marginal costs for developing new sources of supply. This appears to be a conservative (relatively high) estimate in view of the marginal costs for electricity from various new sources summarized below:

- Barber and Carr (1980)
  New thermal plants
  42 Mills/kWh (capital recovery 8 Mills, O&M 2 Mills, fuel 32 Mills)

- Weitzman, Kirtley (September 1979)
  Mazout burning thermal plant (1980 prices)
  220/132 kV - 34 Mills/kWh
  66/33 kV - 37 Mills/kWh
  11 kV - 45 Mills/kWh
  Low voltage - 55 Mills/kWh

- Pacific Consultants
  Mid-1979 price estimated as 50 mills/kWh, apparently on the basis of the 1978 Sanderson and Porter Power Sector Survey and Hunting Technical Services, Report No. 4, "Alternative Strategies and Site Selection", July 1979.

Consideration was given to adopting a shadow price of zero for unskilled labor. There is a substantial amount of unemployment and under-employment among unskilled workers in Egypt. Therefore, it could be argued that there is no opportunity cost to the economy of employing these workers in the construction, operation, and maintenance of the project and as farm laborers under the land application alternatives. Recently, there have been some reports of difficulties in attracting sufficient numbers of laborers for state and private farms, so labor was valued in the analysis at end-of-1980 market wage rates. However, it should be recognized that this is a conservative approach and may tend to overstate the true economic costs of the project.

Farm Production Costs. Cost of farm production information was available from numerous sources. Farm cost and returns studies have recently been conducted for several reclamation projects such as "Economic Evaluation of Land Reclamation" (Ref. 4) prepared by the Ministry of Irrigation, and "New Lands Productivity in Egypt" (Ref. 5) prepared by a USAID contractor. Another important source of information is from studies and cost of production records maintained by the Ministry of Agriculture (Ref. 6). Information from these various sources was used in preparing an independent estimate of costs for producing representative crops under Alternative IIIA and IIIB.

To avoid double counting, farm budget costs do not include items accounted for in other project costs such as irrigation facilities and electric energy.

Farm labor costs are based on total work requirements. Therefore, they reflect a return to all farm labor even if it is performed by the farm family.

Nearly all of the fertilizer requirements are met by nutrients in the wastewater. Potatoes and alfalfa are the only crops requiring any supplemental commercial fertilizer.

Miscellaneous costs too numerous to itemize are estimated as 10 percent of the other crop production costs.

The return on investment for machinery is included with the production cost for machinery. All other investment costs are included in project costs.

The allowance for return to management is based on 1 percent of the gross income.

Farm Budgets. Individual crop cost and return studies (farm budgets) were used to estimate net farm income and returns to water. These budgets served as a basis for measuring the direct irrigation benefits.

Budgets were prepared for representative crops that would be grown under each land application alternative. The crops selected as representative of the cropping pattern include berseem, wheat, and potatoes for Alternative IIIA, and alfalfa and grass hay or pasture for Alternative IIIB. As discussed in the "Cropping Pattern" section, other crops would be grown under each alternative. However, these crops are reasonably representative of the costs and returns that can be expected on typical farming operations in the irrigation service area. Summaries of the economic budgets for each crop are presented in Tables N-4 through N-8.

Direct Benefits. Direct irrigation benefits amount to LE 42.0 million per year for Alternative IIIA and LE 5.8 million per year for Alternative IIIB at full project development. These benefits are calculated on the basis of the return to water for the representative crops and the anticipated cropping pattern. They are based on price levels at the end of 1980. The computations are shown in Tables N-9 and N-10.

A period of about 5 years from initial irrigation will be required to achieve full crop production. Crop production--and hence direct irrigation benefits--will be low during the first few years of irrigation. During this period, salts will be leached from the root zone and organic matter will be incorporated into the soil. This 5-year development period is reflected in scheduling the irrigation benefits from each land application alternative. The schedule of benefits also reflects the projected increase in wastewater flows up to the year 2000 design capacity.

Other Benefits. The primary benefits from the land application alternatives, in addition to the irrigation benefits, would be in protecting the public health and general environment in the Alexandria region. These are described in Appendix K and are at least equal and probably superior to the similar benefits provided by the sea disposal alternative. Benefits from increased tourism and improved labor productivity

because of better health would be similar to those from the sea disposal alternative. Local employment from construction, operation, and maintenance of the land application alternatives would be greater because of the larger project costs. Offsetting these environmental benefits somewhat would be some unavoidable changes in the ground-water conditions as a result of irrigation.

In addition to the direct irrigation benefits, Land Application Alternative IIIA provides important benefits in achieving the national goal of food security. The annual value expressed in 1980 shadow prices of food crops produced with the year 2000 wastewater flow is LE 71.2 million. This would have a beneficial influence on the balance of payments by reducing food imports and perhaps increasing exports of some crops.

Permanent employment opportunities would be provided on irrigated farms. The number of jobs will depend upon whether labor intensive or mechanized farming methods are adopted. A rough estimate indicates about 1100 farm jobs would be created by Alternative IIIA. Only about 350 farm jobs would be created by Alternative IIIB because of the different cropping pattern and more mechanized farming practices necessary to minimize health risks.

Tables K-28, K-29 and K-30 in Appendix K, Land Application of Wastewater, provide a summary of the social impacts, public health impacts, and environmental impacts of Alternatives IIIA and IIIB.

Project Costs

A schedule of construction, operation and maintenance, and replacement costs along with monetary benefits and net benefits for Alternatives IIIA and IIIB are presented in Tables N-11 and N-12, respectively. The costs are on an economic basis, excluding taxes and duties and utilizing shadow prices where appropriate. They are based on price levels at the end of 1980.

Energy costs are an important consideration in evaluating the land application alternatives. The energy requirements for both land application alternatives are substantially greater than for the sea disposal alternative. Energy costs in the economic analysis have been based on marginal rates, which reflect the full cost of developing new sources of energy supply. Therefore, the operation and maintenance costs in Tables N-11 and N-12 differ from those reported in other appendices and sections of the report.

COMPARISON OF ALTERNATIVES

The comparison of alternatives was made on the basis of an incremental analysis. Both of the land application alternatives are more costly than the sea disposal alternative. All three alternatives achieve the basic purpose of maintaining public health and environmental standards. Therefore, the focus of the analysis was on whether the additional costs (i.e., those costs in excess of the costs of implementating Alternative IB) for land application of the wastewater can be justified on the basis of the additional economic benefits.

Based on the projected economic costs and benefits presented in Tables N-1, N-11, and N-12, the IRR for incremental costs and benefits for Alternative IIIA is 7 percent over a 50-year period of analysis. For Alternative IIIB, incremental project costs would exceed incremental benefits even at an interest rate of zero.

SENSITIVITY ANALYSIS

A sensitivity analysis was conducted to determine whether the results would be significantly changed if different assumptions were used concerning the costs and benefits of the alternatives. The results of the sensitivity analysis for Alternative IIIA are presented in Table N-13.

A delay from 5 to 10 years in the time required to attain full crop yields would reduce the IRR from 7.0 to 5.5 percent. If construction costs increased 10 percent above the estimates, the IRR would be 6.3 percent.

Substantial amounts of electricity are used for pumping in Alternative IIIA. A marginal rate of LE .056 per kWh was used to compute energy costs in this economic analysis. If this rate is increased by 10 percent, the IRR would be 6.6 percent. Note that marginal rates are substantially higher than current market rates for electricity.

If the shadow prices for crops produced on irrigated lands are increased by 10 percent, the IRR would increase to 8.9 percent. This indicates the study results are significantly influenced by crop prices.

The IRR decreases to 6.8 percent if the salvage values at the end of the 50-year period of analysis are excluded. Although the salvage values are substantial, they are heavily discounted in the analysis on a present worth basis.

The IRR increases from 7.0 percent to 7.7 percent when the foreign exchange rate is changed from LE 0.70 = $1.00 US to LE 0.85 = $1.00 US. This indicates that the use of an artificially low exchange

rate tends to understate the benefits in relation to the costs for Alternative IIIA. If the trend toward devaluation of the Egyptian currency continues, land application of the wastewater under Alternative IIIA will become increasingly desirable.

In conclusion, it appears that some delays in reaching full project development or minor variations in construction costs, electricity costs, or crop prices could occur without significantly influencing the economic feasibility of Alternative IIIA. Of those variables tested, changes in crop prices and the foreign exchange rate appear to have the greatest potential influence on study results.

FINANCIAL CONSIDERATIONS

Arrangements for financing and recovery of costs for the project have not yet been determined. The financial feasibility of the project cannot be evaluated until sources and terms of financing are decided and policies are established for repayment. The estimates of capital costs, presented in Tables 4-1, 5-1, and 5-2 of the report, provide a basis for determining financing requirements. A breakdown between local and foreign exchange costs is provided in those tables.

A discussion is presented below of the potential role of water pricing and its effect on the project. Consideration also is given in this analysis to the viability of the project from the perspective of the farm operator.

Purpose of Water Pricing

Water pricing is a complex issue involving legal, social, economic, and financial considerations. The interest and participation by potential farm operators will be determined to a large extent by the cost of water. It is important to understand the role of prices in achieving project objectives and determining project feasibility.

Water prices serve two basic purpose. First, they are a source of revenue for financing the project. Second, if based on marginal cost principles, they serve to encourage the efficient use of resources. However, water prices also can be established to accomplish other goals, such as the redistribution of income from one group to another or the economic development of a particular region or sector of the economy.

Subsidies serve to mask the true cost of water. Water policies in the United States, for example, have sought to keep the prices of agricultural water low to achieve social objectives. If the true cost of the water were paid, farmers would probably grow less water-intensive

crops or improve irrigation efficiencies and farming methods. Waste should not be encouraged by providing water at prices substantially below its true cost.

The financial purpose of water pricing is to provide revenue for constructing, operating, and maintaining the project. However, it is often difficult to satisfy both the economic and financial requirements of a water project. Benefits are frequently widespread through the economy, making it difficult or impractical to identify and collect revenue from all the beneficiaries. When charges to beneficiaries do not produce sufficient revenues, other approaches must be used. Government financing, such as grant programs, is the most common method of overcoming this problem.

Financial Return to Water

The farm budgets prepared for the economic analysis were modified to include market prices for farm crops and production inputs rather than economic (shadow) prices. This provided a basis for determining whether farm operators will receive adequate returns to meet their financial obligations in view of the market prices that will prevail under government price controls, subsidies, and other such influences.

Summaries of the financial budgets for respresentative crops that would be grown under Land Application Alternative IIIA are presented in Tables N-14 through N-16. The crops representative of the cropping pattern include berseem, wheat, and potatoes.

The budgets indicate the income from each crop at full development is adequate to pay production costs, provide a return to management, and yield a residual return to water. The residual return to water amounts to LE 109.3 per feddan for berseem, LE 104.3 per feddan for wheat, and LE 255.1 per feddan for potatoes. Two of these crops can be produced per year under the cropping pattern anticipated for the service area.

The financial returns to water and related services provided by the project under Alternative IIIA are summarized in Table N-17. The weighted average annual financial return per feddan are presented for both the relatively low initial level of crop production and at the full production levels attained in the fifth year of irrigation. The average annual financial return is expected to increase from LE 115.8 per feddan in the first year to LE 434.8 per feddan in the fifth year. A portion of these returns could be used to help pay for project construction, operation, and maintenance costs.

In conclusion, the project would be financially feasible from the viewpoint of the farm operators, provided the water charges do not

exceed the ability to pay as indicated by the financial return to water for each crop. The policy formulated for assessing water charges should provide for reduced payments during the initial 5-year development period before full crop production is attained. It also should reflect the expansion in area served with reclaimed water as the supply of wastewater increases.

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Table N-1

ECONOMIC COSTS FOR ALTERNATIVE 1B, SEA DISPOSAL

(LE 0.70 = $1.00 US)

	Construction Costs				Replacement Costs		
Year	Local Cost (10^6 LE)	Foreign Exchange (10^6 $)	Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Local Cost (10^6 LE)	Foreign Exchange (10^6 $)	Total Cost (10^6 LE)
1	4.7	4.2	7.7	0	0	0	0
2	11.4	12.4	20.2	0	0	0	0
3	45.5	50.9	81.2	0	0	0	0
4	62.1	57.0	102.0	2.22	0	0	0
5	66.6	64.8	111.9	2.22	0	0	0
6	37.4	34.5	61.6	5.33	0	0	0
7	30.6	26.4	49.1	6.89	0	0	0
8	15.7	13.8	25.4	10.00	0	0	0
9	1.4	2.9	3.4	12.81	0	0	0
10	2.8	6.0	7.0	13.23	0	0	0
11	0	0	0	13.50	0	0	0
12	0	0	0	13.74	0	0	0
13	0	0	0	13.99	0	0	0
14	0	0	0	14.24	0	0	0
15	0	0	0	14.49	0	0	0
16	0	0	0	14.75	0	0	0
17	0	0	0	14.99	0	0	0
18	0	0	0	15.25	1.1	4.8	4.5
19	0	0	0	15.50	0	0	0
20	0	0	0	15.75	0	0	0
21	0	0	0	15.75	8.8	37.4	35.0
22	0	0	0	15.75	0	0	0
23	0	0	0	15.75	10.7	46.1	42.9
24	0	0	0	15.75	0	0	0
25	0	0	0	15.75	1.0	4.0	3.8
26	0	0	0	15.75	0	0	0
27	0	0	0	15.75	0	0	0
28	0	0	0	15.75	0	0	0
29	0	0	0	15.75	0	0	0
30	0	0	0	15.75	0	0	0

Table N-1 (continued)
ECONOMIC COSTS FOR ALTERNATIVE IB, SEA DISPOSAL
(LE 0.70 = $1.00 US)

Year	Construction Costs: Local Cost (10^6 LE)	Construction Costs: Foreign Exchange (10^6 $)	Construction Costs: Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Replacement Costs: Local Cost (10^6 LE)	Replacement Costs: Foreign Exchange (10^6 $)	Replacement Costs: Total Cost (10^6 LE)
31	0	0	0	15.75	0	0	0
32	0	0	0	15.75	0	0	0
33	0	0	0	15.75	1.1	4.8	4.5
34	0	0	0	15.75	0	0	0
35	0	0	0	15.75	0	0	0
36	0	0	0	15.75	8.8	37.4	35.0
37	0	0	0	15.75	0	0	0
38	0	0	0	15.75	10.7	46.1	42.9
39	0	0	0	15.75	0	0	0
40	0	0	0	15.75	1.0	4.1	3.9
41	0	0	0	15.75	0	0	0
42	0	0	0	15.75	4.1	2.0	5.5
43	0	0	0	15.75	22.0	10.4	29.3
44	0	0	0	15.75	16.2	7.7	21.6
45	0	0	0	15.75	22.9	11.0	30.6
46	0	0	0	15.75	15.5	7.4	20.7
47	0	0	0	15.75	19.2	9.2	25.6
48	0	0	0	15.75	9.9	8.9	16.2
49	0	0	0	15.75	1.6	0.9	2.2
50	0	0	0	15.75	3.4	1.6	4.5

Salvage value at end of 50-year period is LE 176.5 million.

Table N-2
CROP YIELDS AT FULL PRODUCTION

Crop	Crop Yield (tonnes/fd) Alternative IIIA	Alternative IIIB
Winter		
Berseem	12.0	--
Wheat	1.3	--
Barley	1.3	--
Cabbage	9.0	--
Chickpeas	2.0	--
Potatoes	7.0	--
Wheat byproduct	2.0	--
Summer		
Sugar beets	14.0	--
Tomatoes	7.0	--
Potatoes	7.0	--
Squash	7.0	--
Corn	1.6	--
Both Seasons		
Alfalfa	--	23
Grass hay/pasture	--	10

Table N-3
COMPARISON OF MARKET AND SHADOW PRICES

Item	Unit	Average Farm Gate Price (LE) Market	Shadow
Farm Products			
Berseem	Tonne	15	15
Wheat	Tonne	7	200
Potatoes	Tonne	80	110
Alfalfa	Tonne	16	16
Grass hay/pasture	Tonne	14	14
Wheat byproduct	Tonne	30	30
Production Inputs			
Potash Fertilizer (60% K)	Tonne	35	108
Fuel (diesel)	Litre	0.03	0.161
Wheat seed	Tonne	115	233
Project Inputs			
Electricity	kWh	0.008	0.056

Table N-4

ECONOMIC FARM BUDGET FOR BERSEEM

Item	LE/Feddan
Income (12 tonne/fedden @ LE 15/tonne)	180.0
Crop Production Costs	
Farm Labor	2.5
Machinery	46.6
Seeds	9.6
Fuel	20.7
Miscellaneous	7.9
Total	87.3
Net Income (Gross Income less production costs)	92.7
Less Return to Management (1% of gross income)	1.8
Residual Return to Water (LE)	90.9

Notes:

1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Shadow prices are used to reflect the true value to the national economy of farm products.

Table N-5
ECONOMIC FARM BUDGET FOR WHEAT

Item	LE/Feddan
Income	
Crop Income (1.3 tonne/feddan @ LE 200/tonne)	260.0
Byproduct Income (2.0 tonne/feddan @ LE 30/tonne)	60.0
Gross Income	320.0
Crop Production Costs	
Farm Labor	1.6
Machinery	40.6
Seeds	12.8
Fuel	8.6
Miscellaneous	6.4
Total	70.0
Net Income (gross income less production costs)	250.0
Less Return to Management (1% of gross income)	3.2
Residual Return to Water (LE)	246.8

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Shadow prices are used to reflect the true value to the national economy of farm products and inputs.

Table N-6
ECONOMIC FARM BUDGET FOR POTATOES

Item	LE/Feddan
Income (7.0 tonne/feddan @ LE 110/tonne)	770.0
Crop Production Costs	
Farm Labor	3.8
Machinery	74.3
Seeds	160.0
Pesticides and Chemicals	25.5
Fertilizer (150 kg potash-60% K @ LE 108/kg)	16.2
Fuel	17.7
Miscellaneous	29.8
Total	327.3
Net Income (gross income less production costs)	442.7
Less Return to Management (1% of gross income)	7.7
Residual Return to Water (LE)	435.0

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Shadow prices are used to reflect the true value to the national economy of farm products and inputs.

Table N-7
ECONOMIC FARM BUDGET FOR ALFALFA

Item	LE/Feddan
Income (23 tonne/feddan @ LE 16/tonne)	368.0
Crop Production Costs	
Crop Establishment Cost	14.3
Farm Labor	4.7
Machinery	77.5
Pesticides and Chemicals	3.8
Fertilizer (34 kg potash-60% K @ LE 108/kg)	3.7
Fuel	24.0
Miscellaneous	12.8
Total	140.8
Net Income (gross income less production costs)	227.2
Less Return to Management (1% of gross income)	3.7
Residual Return to Water (LE)	223.5

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Shadow prices are used to reflect the true value to the national economy of farm products and inputs.

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Table N-8

ECONOMIC FARM BUDGET FOR GRASS HAY/PASTURE

Item	LE/Feddan
Income (10 tonne/feddan @ LE 14/tonne)	140.0
Crop Production Costs	
Crop Establishment Cost	8.2
Farm Labor	1.4
Machinery	2.7
Pesticides and Chemicals	3.8
Fuel	4.0
Miscellaneous	2.0
Total	22.1
Net Income (gross income less production costs)	117.9
Less Return to Management (1% of gross income)	1.4
Residual Return to Water (LE)	116.5

Notes:

1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Shadow prices are used to reflect the true value to the national economy of farm products and inputs.

Table N-9
DIRECT IRRIGATION BENEFITS AT FULL DEVELOPMENT
ALTERNATIVE IIIA, LAND APPLICATION

Crop	Representative Budget	Harvested Area[a] (Feddan)	Return to Water Per Feddan (LE)	Total Return to Water (LE)
Grain	Wheat	27,080	246.8	6,683,000
Vegetables	Potatoes	81,240	435.0	35,339,000
Direct Irrigation Benefit				42,022,000

[a]Harvested area exceeds irrigation service area because more than one crop can be grown per year (crop intensity 2.0).

Table N-10
DIRECT IRRIGATION BENEFITS AT FULL DEVELOPMENT
ALTERNATIVE IIIB, LAND APPLICATION

Crop	Representative Budget	Area (Feddan)	Return to Water Per Feddan (LE)	Total Return to Water (LE)
Alfalfa	Alfalfa	17,120	223.5	3,826,000
Grass Hay	Grass Hay	17,120	116.5	1,994,000
Direct Irrigation Benefit				5,820,000

Table N-11
ECONOMIC COSTS AND BENEFITS
ALTERNATIVE IIIA, LAND APPLICATION
(LE 0.70 = $1.00 US)[a]

Year	Construction Costs: Local Cost (10^6 LE)	Construction Costs: Foreign Exchange (10^6 $)	Construction Costs: Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Replacement Costs: Local Cost (10^6 LE)	Replacement Costs: Foreign Exchange (10^6 $)	Replacement Costs: Total Cost (10^6 LE)	Monetary Benefits (10^6 LE)	Net Benefits (10^6 LE)
1	1.4	2.0	2.8	0	0	0	0	0	-2.80
2	3.6	6.7	8.3	0	0	0	0	0	-8.30
3	6.2	10.4	13.5	0	0	0	0	0	-13.50
4	17.7	7.3	22.8	2.22	0	0	0	0	-25.02
5	35.3	12.4	43.9	2.79	0	0	0	0	-46.69
6	237.5	77.6	291.8	2.79	0	0	0	0	-294.59
7	237.5	77.6	291.8	2.79	0	0	0	0	-294.59
8	100.8[b]	38.4	127.8[b]	12.35	0	0	0	6.62	-133.53
9	0	0	0	22.63	0	0	0	9.74	-12.89
10	10.5	5.9	14.6	23.35	0	0	0	20.85	-17.10
11	0	0	0	23.86	0	0	0	21.65	-2.21
12	0	0	0	24.39	0	0	0	32.78	8.39
13	0	0	0	24.91	0	0	0	33.89	8.98
14	0	0	0	25.41	0	0	0	34.93	9.52
15	12.0	5.6	15.9	25.92	0	0	0	35.93	-5.89
16	0	0	0	26.43	0	0	0	36.89	10.46
17	0	0	0	26.93	0	0	0	37.88	10.95
18	0	0	0	27.44	1.13	4.82	4.50	38.85	6.91
19	0	0	0	27.95	0	0	0	39.82	11.87
20	0	0	0	28.45	0	0	0	40.63	12.18
21	0	0	0	28.45	0	0	0	41.28	12.83
22	0	0	0	28.45	9.89	42.39	39.56	41.67	-26.34
23	0	0	0	28.45	11.38	48.77	45.52	41.99	-32.98
24	0	0	0	28.45	0	0	0	42.02	13.57
25	0	0	0	28.45	1.04	4.47	4.17	42.02	9.40

[a]Shadow prices rather than existing market prices are used to reflect the true opportunity costs of resources.

[b]Represents LE 144,500,000 construction cost less LE 16,700,000 salvage value for West plant.

Table N-11 (continued)
ECONOMIC COSTS AND BENEFITS,
ALTERNATIVE IIIA, LAND APPLICATION
(LE 0.70 = $1.00 US)[a]

Year	Construction Costs: Local Cost (10^6 LE)	Construction Costs: Foreign Exchange (10^6 $)	Construction Costs: Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Replacement Costs: Local Cost (10^6 LE)	Replacement Costs: Foreign Exchange (10^6 $)	Replacement Costs: Total Cost (10^6 LE)	Monetary Benefits (10^6 LE)	Net Benefits (10^6 LE)
26	0	0	0	28.45	0	0	0	42.02	13.57
27	0	0	0	28.45	0	0	0	42.02	13.57
28	0	0	0	28.45	0	0	0	42.02	13.57
29	0	0	0	28.45	0	0	0	42.02	13.57
30	0	0	0	28.45	1.16	4.96	4.63	42.02	8.94
31	0	0	0	28.45	0	0	0	42.02	13.57
32	0	0	0	28.45	0	0	0	42.02	13.57
33	0	0	0	28.45	1.13	4.82	4.50	42.02	9.07
34	0	0	0	28.45	0	0	0	42.02	13.57
35	0	0	0	28.45	0	0	0	42.02	13.57
36	0	0	0	28.45	0	0	0	42.02	13.57
37	0	0	0	28.45	26.70	50.39	61.97	42.02	-48.40
38	0	0	0	28.45	11.38	48.77	45.52	42.02	-31.95
39	0	0	0	28.45	0	0	0	42.02	13.57
40	0	0	0	28.45	1.05	4.49	4.19	42.02	9.38
41	0	0	0	28.45	0	0	0	42.02	13.57
42	0	0	0	28.45	4.13	1.96	5.50	42.02	8.07
43	0	0	0	28.45	8.40	4.00	11.20	42.02	2.37
44	0	0	0	28.45	0	0	0	42.02	13.57
45	0	0	0	28.45	2.19	5.45	6.01	42.02	7.56
46	0	0	0	28.45	0	0	0	42.02	13.57
47	0	0	0	28.45	10.88	5.18	14.51	42.02	-0.94
48	0	0	0	28.45	20.58	47.48	53.81	42.02	-40.42
49	0	0	0	28.45	0	0	0	42.02	13.57
50	0	0	0	28.45	0	0	0	42.02	13.57

Salvage value at end of 50-year period is LE 356.1 million.

[a]Shadow prices rather than existing market prices are used to reflect the true opportunity costs of resources.

Table N-12
ECONOMIC COSTS AND BENEFITS
ALTERNATIVE IIIB, LAND APPLICATION
(LE 0.70 = $1.00 US)[a]

Year	Construction Costs: Local Cost (10^6 LE)	Construction Costs: Foreign Exchange (10^6 $)	Construction Costs: Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Replacement Costs: Local Cost (10^6 LE)	Replacement Costs: Foreign Exchange (10^6 $)	Replacement Costs: Total Cost (10^6 LE)	Monetary Benefits (10^6 LE)	Net Benefits (10^6 LE)
1	1.0	1.5	2.0	0	0	0	0	0	-2.00
2	3.2	6.0	7.4	0	0	0	0	0	-7.40
3	6.2	10.4	13.5	0	0	0	0	0	-13.50
4	17.7	7.3	22.8	2.22	0	0	0	0	-25.02
5	32.0	8.8	38.1	2.79	0	0	0	0	-40.89
6	228.9	63.0	273.0	2.79	0	0	0	0	-275.79
7	228.9	63.0	273.0	2.79	0	0	0	0	-275.79
8	97.0[b]	31.2	118.9[b]	11.00	0	0	0	2.20	-127.70
9	0	0	0	19.83	0	0	0	3.20	-16.60
10	8.2	2.4	9.9	20.46	0	0	0	3.35	-27.01
11	0	0	0	21.01	0	0	0	3.46	-17.55
12	0	0	0	21.36	0	0	0	4.63	-16.73
13	0	0	0	21.81	0	0	0	4.77	-17.04
14	0	0	0	22.23	0	0	0	4.75	-17.48
15	8.7	3.6	11.2	22.68	0	0	0	5.05	-28.83
16	0	0	0	23.10	0	0	0	5.19	-17.91
17	0	0	0	23.54	0	0	0	5.32	-18.22
18	0	0	0	23.96	1.13	4.82	4.50	5.45	-23.01
19	0	0	0	24.41	0	0	0	5.58	-18.83
20	0	0	0	24.83	0	0	0	5.69	-19.14
21	0	0	0	24.83	0	0	0	5.73	-19.10
22	0	0	0	24.83	5.11	21.91	20.45	5.76	-39.52
23	0	0	0	24.83	11.38	48.77	45.52	5.80	-64.55
24	0	0	0	24.83	0	0	0	5.82	-19.01
25	0	0	0	24.83	0.83	3.56	3.32	5.82	-22.33

[a]Shadow prices rather than existing market prices are used to reflect the true opportunity costs of resources.
[b]Represents construction cost less LE 16,700,000 salvage value for West plant.

Table N-12 (continued)
ECONOMIC COSTS AND BENEFITS
ALTERNATIVE IIIB, LAND APPLICATION
(LE 0.70 = $1.00 US)[a]

	Construction Costs				Replacement Costs				
Year	Local Cost (10^6 LE)	Foreign Exchange (10^6 $)	Total Cost (10^6 LE)	O&M Costs (10^6 LE)	Local Cost (10^6 LE)	Foreign Exchange (10^6 $)	Total Cost (10^6 LE)	Monetary Benefits (10^6 LE)	Net Benefits (10^6 LE)
26	0	0	0	24.83	0	0	0	5.82	-19.01
27	0	0	0	24.83	0	0	0	5.82	-19.01
28	0	0	0	24.83	0	0	0	5.82	-19.01
29	0	0	0	24.83	0	0	0	5.82	-19.01
30	0	0	0	24.83	0.83	3.56	3.32	5.82	-22.33
31	0	0	0	24.83	0	0	0	5.82	-19.01
32	0	0	0	24.83	0	0	0	5.82	-19.01
33	0	0	0	24.83	1.13	4.82	4.50	5.82	-23.51
34	0	0	0	24.83	0	0	0	5.82	-19.01
35	0	0	0	24.83	0	0	0	5.82	-19.01
36	0	0	0	24.83	0	0	0	5.82	-19.01
37	0	0	0	24.83	19.03	28.54	39.00	5.82	-58.01
38	0	0	0	24.83	11.38	48.77	45.52	5.82	-64.53
39	0	0	0	24.83	0	0	0	5.82	-19.01
40	0	0	0	24.83	0.83	3.56	3.32	5.82	-22.33
41	0	0	0	24.83	0	0	0	5.82	-19.01
42	0	0	0	24.83	4.13	1.96	5.50	5.82	-24.51
43	0	0	0	24.83	8.40	4.00	11.20	5.82	-30.21
44	0	0	0	24.83	0	0	0	5.82	-19.01
45	0	0	0	24.83	1.78	4.01	4.58	5.82	-23.59
46	0	0	0	24.83	0	0	0	5.82	-19.01
47	0	0	0	24.83	5.00	2.38	6.67	5.82	-25.68
48	0	0	0	24.83	38.11	22.43	53.81	5.82	-72.82
49	0	0	0	24.83	0	0	0	5.82	-19.01
50	0	0	0	24.83	0	0	0	5.82	-19.01

Salvage value at end of 50-year period is LE 283.4 million.

[a]Shadow prices rather than existing market prices are used to reflect the true opportunity costs of resources.

Table N-13
SENSITIVITY ANALYSIS

Item	Internal Rate of Return for Alternative IIIA (Percent)
Base Case	7.0
Attainment of Full Crop Yields Delayed to 10 Years	5.5
Construction Cost Increased 10%	6.3
Electricity Costs Increased 10%	6.6
Prices Received Increased 10%	8.9
Excluding Salvage Value	6.8
Change in Foreign Exchange Rate from LE 0.70 = $1.00 to LE 0.85 = $1.00	7.7

Table N-14
FINANCIAL FARM BUDGET FOR BERSEEM

Item	LE/Feddan
Income (12 tonne/feddan @ LE 15/tonne)	180.00
Crop Production Costs	
Farm Labor	2.5
Machinery	46.6
Seeds	9.6
Fuel	3.9
Miscellaneous	6.3
Total	68.9
Net Income (gross income less production costs)	111.1
Less Return to Management (1% of gross income)	1.8
Residual Return to Water (LE)	109.3

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Existing market prices are used for all items in the farm budget to assess the ability of farm operators to meet their financial obligations.

Table N-15
FINANCIAL CROP BUDGET FOR WHEAT

Item	LE/Feddan
Income	
Crop Income (1.3 tonne/feddan @ LE 77/tonne)	100.0
Byproduct Income (2.0 tonne/feddan @ LE 30/tonne)	60.0
Gross Income	160.0
Crop Production Costs	
Farm Labor	1.6
Machinery	40.6
Seeds	6.3
Fuel	1.6
Miscellaneous	4.9
Total	55.0
Net Income (gross income less production costs)	105.0
Less Return to Management (1% of gross income)	1.6
Residual Return to Water (LE)	103.4

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Existing market prices are used for all items in the farm budget to assess the ability of farm operators to meet their financial obligations.

Table N-16
FINANCIAL CROP BUDGET - POTATOES

Item	LE/Feddan
Income (7.0 tonne/feddan @ LE 80/tonne)	560.0
Crop Production Costs	
Farm Labor	3.8
Machinery	74.3
Seeds	160.0
Pesticides	25.5
Fertilizer (150 kg potash-60% K @ .035/kg)	5.2
Fuel	3.3
Miscellaneous	27.2
Total	299.3
Net Income (gross income less production costs)	260.7
Less Return to Management (1% of gross income)	5.6
Residual Return to Water (LE)	255.1

Notes:
1. To avoid double counting, farm budget costs do not include items accounted for in project costs such as irrigation facilities and electric energy.
2. Existing market prices are used for all items in the farm budget to assess the ability of farm operators to meet their financial obligations.

Table N-17

ALTERNATIVE IIIA, FINANCIAL RETURNS TO [illegible]

Crop Production Level	Representative Budget	Harvested Area[a] (Feddan)	Financial Return to Water per Feddan (LE)	Financial Return to Water (LE)	Weighted Average Return Financial Return Per Feddan
Initial Production[b]	Berseem	54,160	57.3	3,103,000	
	Wheat	54,160	58.5	3,168,000	
				6,271,000	115.8
Full Production[c]	Wheat	27,080	103.4	2,800,000	
	Potatoes	81,240	255.1	20,724,000	
				23,524,000	434.3

[a]Harvested area exceeds irrigation service area because more than one crop can be grown per year (crop intensity 2.0).

[b]Crop yields in the first year of irrigation will amount to only 60 percent of those attained at full production.

[c]Full production will be attained in the fifth year of irrigation.