3.6.2 Reuse of Treatment Plant Effluent

(a) General

In view of the save of water resources, reuse of treatment plant effluent is one of the most important strategies in the El-Arish area. Of various reuse schemes, top priority is laid on the agricultural development.

It is well known that agriculture in the El-Arish area is the most important economic sector, and in recent years, accounts for almost 30 per cent of the Gross Domestic Product. And 40 per cent of the labour forces in the El-Arish area is engaged in or dependent on farming and related activities.

In the existing farm land of the El-Arish area, various crops such as olive, dates, watermelon, cucumber, corn, tomatoes and others are harvested annually. Nevertheless, the total output lags behind the rate of population growth.

In order to increase the output, the Governorate of North Sinai is newly planning to promote the agricultural development utilizing treatment plant effluent, besides the development of the existing agriculture. In accordance with the Governorate's policy, an agricultural development has been planned in the land of about 600 feddans extending within the Jarada area.

(b) Proposed Cropping Pattern, Projected Yield and Prospective Production

In the selection of appropriate crops for the farm land of the proposed Jarada area, first of all, attention must be paid to sanitary problems as well as water requirements rather than agronomic and economic problems from the viewpoint of utilizing treatment plant effluent, because the availability of water and sanitary problems are the most important limiting factors in the proposed Jarada area.

Other important and interrelated factors are marketability, or possibility of demand increases in the near future, and profitability. Presently, the El-Arish area has shortage of supply in most of major upland crops as well as food crops. Rice and sugarcane are undesirable in the farm land of the Jarada area despite of the higher profitability and large marketability mainly due to the high water consumption.

The Governorate of North Sinai suggests that most of upland crops currently consumed in the El-Arish area have good market prospects. Especially, production of upland crops such as tomatoes, watermelon, cucumber, potatoes, and fruits like citrus and olive are likely to increase in parallel with rises in per-capita income derivable from the regional development.

Taking into consideration the present status and the above-said conditions of the farm land in the Jarada area, appropriate crops for agricultural production has been selected as given below.

Winter crops : berseem, barley, potatoes and broad beans
Summer crops : okra, watermelon and green pepper
Perennial crops : citrus, olive and alfalfa

Figure 3.6.1 illustrates a representative cropping pattern to be applied for the farm land of the Jarada area. As seen in the cropping pattern, the winter crops are berseem, barley, potatoes and broad beans, of which berseem is consumed on the farm by settlers' livestock. Almost all of crop products are consumed for local use. The summer crops are okra, watermelon and green pepper of which almost all of crops are consumed for local use. The perennial crops are citrus, olive and alfalfa, of which part of olive can be sent Port Said for export, and alfalfa is consumed for feed.

Production at the full development stage in the farm land is shown in Table 3.6.1 The contribution which the farm land of the Jarada area can make to meet the incremental demand of the El-Arish area for agricultural products in the near future is relatively modest if a suitable regional plan is implemented.

Table 3.6.1 Prospective Production at Full Development

(ton/year)

Crop	Production	Crop	Production
Winter		Summer	
Barley	86.4	Okra	146.9
Potatoes	230.4	Watermelon	1,751.0
Broad beans	86.4	Green pepper	404.5
Perennial			
Citrus	201.6		

(c) Irrigation Requirements

Irrigation efficiency (Ei) is a product of conveyance efficiency (Ec) and field application efficiency (Ea). Conveyance efficiency (Ec) in the farm land of the Jarada area is expected to be above 95 per cent in view of the soil conditions and related factors. Generally, field application efficiency (Ea) of the drip system is designed at a level of 95 to 100 per cent in case of fields and green houses. In the farm land of the Jarada area, it is assumed at 95 per cent in due consideration of the field cultivation. The irrigation efficiency (Ei) is determined as below.

$$Ei = Ec \times Ea = 0.95 \times 0.95 = 0.903 = 0.90$$

Irrigation requirements are obtained dividing crop water requirements by irrigation efficiency (Ei), and the calculation formula is as follows.

$$q = ETc \times 4.2 \times 1/Ei \times 1/86{,}400 \times 24/Ti$$

Here,

q	=	irrigation requirements (l/sec/fd)
ETc	=	crop water requirements (mm/day) 1/
Ei	=	irrigation efficiency == 0.90 in case of drip system

4.2 = ratio of conversion in feddan

Ti = irrigation hours == 24 hours in case of drip system

Therefore,

$$q = 7.0 \times 4.2 \times 1/86,400 \times 1/0.90 \times 24/24 = 0.3781 \text{ l/sec/fd}$$

According to the proposed treatment plan, an amount of effluent derived from the treatment plant is estimated at 20,000 m^3 per day, nearly equal to 0.231 m^3/sec. As the irrigation requirements are 0.3781 l/sec/fd as mentioned above, irrigation for farm lands of 611 feddans will become possible as below.

As described later, an experimental farm of 12 feddans in scale is planned to establish in the Jarada area. Of 12 feddans, eight feddans will be utilized as farm lands. Irrigation water necessary for the operation and management of the farm land is estimated at about 300 m^3 per day as seen in the following formulae.

$$0.3781 \text{ l/sec/fd} \times 86,400 = 32.7 \text{ m}^3\text{/day/fd}$$

$$32.7 \text{ m}^3\text{/day/fd} \times 8 \text{ fd} = 261.6 \text{ m}^3\text{/day/8 fd}$$

(d) Proposed Irrigation Method

In the El-Arish area, drip irrigation is one of the most popular and recommendable methods which is most applicable for the soils of the area and supplies the quantity of necessary water on a daily basis in general. Water is applied from each of many small emitters at a low rate. The timing and

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1/ In this study, crop water requirements (ETc) were calculated utilizing Blaney - Criddle Method on the basis of the report prepared by KUP - Engineer Consult, Federal Republic of Germany, in 1983 in due consideration of 'Pan Evaporation Method' authorized by Food and Agriculture Organization of the United Nations.

Climatic data such as evaporation, humidity, wind velocity and others were obtained from the Meteorological Station at the El-Arish city. Details of crop water requirements are shown in APPENDIX SIX.

According to our calculation, the peak water requirements (weighted mean in June) reach 7.00 mm/day or 29.4 m^3/day/fd and the annual total is 6.103 m^3/year/fd. These amounts mean net irrigation requirements.

Each crop water requirement varies 2.24 mm/day of potatoes in winter to 8.73 mm/day of green pepper in summer and 9.45 mm/day of perennial alfalfa.

duration of each irrigation can often be regulated by hand valves for adjusting water being applied through the duration of irrigation, and/or by changing the number of emitters.

It is noticeable that even with poor quality of water as seen in case of treatment plant effluent, drip irrigation is expected to bring better yields thanks to the continuous high moisture content of soils and daily replenishment of water lost by evapotranspiration. Even though a great benefit is not expected as high as those found utilizing good quality of water, it is considered that other benefits such as possible savings in water, fertilizer or labour forces will be greater. It will be also useful for the justification of the added investment costs of the drip system.

As known already, irrigation method is largely divided into three, namely 1) Surface irrigation; 2) Drip irrigation; and 3) Spray irrigation. Surface irrigation and spray irrigation are further divided into four and two respectively. These are shown as below.

1)	Surface irrigation	Furrow method
		Contour ditch method
		Boarder method
		Basin method
2)	Drip irrigation	
3)	Spray irrigation	Sprinkler method
		Perforated pipe method

Surface irrigation is required for a great quantity of irrigation water. In areas where permeability rate is higher as seen in the farm land of the Jarada area, surface irrigation is not adequate. It should be also considered that use of a great quantity of water causes salt accumulation. Also spray irrigation is required for a relatively great quantity of water. As most of the Jarada area is occupied with sand and sand dunes, spray irrigation is not desirable.

(e) Proposed Settlement Pattern

In our plan, the proposed land of the Jarada area is allocated to settlers 12-feddan unit each. Leaving aside 12 feddans for the proposed experimental farm, the total area available for distribution amounts to about 600 feddans. This means that about 50 families will be selected involving a total farm population of about 300 (assuming an average of six persons per family).

In view of the size of population involved, the elongated East-West extension of the proposed Jarada area and for irrigation management, settlers will be settled in one village unit. The exact location of the unit and the allocation of land to them can only be determined after the economic and social survey of the El-Arish area and detailed design have been completed and the characteristic of settled farmers are known.

The proposed village unit should be located fairly close to the El-Arish - Rafah road. In the unit, a social and service center for the entire Jarada area should be established in such a way that all settlers will secure a certain amount of vegetables obtainable from gardens adjacent to their house plots for self-consumption in a raw state. Further, a certain amount of fresh water necessary for such a vegetable cultivation, livestock rearing as well as their daily living should be provided.

(f) Prospective Farm Management and Organization

The agricultural development of the proposed Jarada area involves three water management blocks and one experimental farm equipped with irrigation facilities. Location of three water management blocks, one experimental farm, proposed housing site and others is illustrated in Figure 3.6.2. Further, basic layout of water management block and experimental farm is illustrated in Figure 3.6.3 and 3.6.4 respectively.

As mentioned already, it is planned to settle one farm household in each 12-feddan unit farm. The irrigation farming will be mostly practised by

Family labour. Effective use of irrigation water, operation of cooperative marketing system for the agricultural products and bulk procurement of the agro-chemicals and fertilizers will be successfully performed with the farmers' cooperative which will be organized by the small-scale farmers.

Agricultural research as well as experimental practices will be carried out at the proposed experimental farm which should be put under the control of the Governorate of North Sinai. It is needless to say that the small-scale farmers will need the guidance by experts for applying the new farming practices.

(g) Livestock Rearing

The position of livetock on the development plan under study is of considerable importance. In the early stages of settlement, forage crops are of principal necessity. The cultivation of leguminous forage crops can assist reclamation by improving soil structure and fertility. Livestock rearing not only produces valuable manures. It also provides the main opportunity for a cash return in the initial years. In the plan, in and around the Jarada area, as a chain of tree planting campaign, forage crops will be cultivated for livestock rearing utilizing surplus of the effluent, especially during the winter season.

(h) Experimental Farm

The proposed settlers have little experience on the production technique for year-round irrigated cultivation utilizing treatment plant effluent. In order to accumulate the experience and increase the knowledge, actual training in an experimental farm is especially required. From the viewpoint of the above, an experimental farm of 12 feddans in scale, together with necessary facilities such as office, laboratory, staff quarters, garage, storage, dormitory and others will be constructed. Principally, the experimental farm will test cultivation methods of various crops in view of utilizing treatment plant effluent. And also it will demonstrate to the farmers the following.

(1) Construction of upland irrigation facilities;

(2) Irrigation techniques;

(3) Crop rotation; and

(4) Modern farming techniques.

The experimental farm will be guided by extension workers through a governorate level extension officer supported by at least three specialists in field crops, water management and agro-economy, respectively. These specialists will be assisted by consultants. In order to improve cultivation techniques for future development, some farm machinery such as 35HP tractor, potato planter, potato harvester, motor-sprayer will be provided on an experimental basis in addition to an audio-visual aid set and some farm tools under the projet.

It must be noted that in view of the objectives of the experimental farm, agriculture in the farm will not always pursue the maximum farm benefit, but it will rather include trials and errors. The operation should not be bound in a single framework.

(i) Water Quality Standards

To ensure the safety of water supplies, first of all, standards have to be prepared and then applied. Standards governing the quality of water in rivers and lakes are becoming common. In some countries such standards have been already provided and in other countries those are being formulated.

However, standards applicable directly to treatment plant effluent have been not yet available except for a few examples in some countries such as U.S.A., Federal Republic of Germany, Israel and South Africa. Some representative standards for the reuse of treatment plant effluent are as shown in Table 3.6.2.

As seen in the table, direct reuse of treatment plant effluent towards crops for human consumption in a raw state is strictly prohibited in almost all of the countries even though the effluent is treated tertiarily.

In Egypt, the use of treatment plant effluent for irrigation is regulated by Law No.93 of 1962 and its executive regulations issued by Decree No.649 of 1962 (Ministry of Housing).

According to Article 14 of the Law No.93, the surface discharge of liquid waste without authorization from the authority in charge of sewage is prohibited, and it is also described that the above matter must be in accordance with the condition, specifications and norms established by the Ministry of Health and decreed by the Ministry of Housing.

In Article 15 of the Law it is stated that the Ministry of Housing shall, after approval by the Ministry of Health, issue a decree indicating the standard specifications for the methods of obtaining and analysis of samples, and the specifications and conditions required in liquid waste used for irrigation or other purposes.

In Chapter Six of the Law's executive regulations by Decree No.649 of 1962, the norms and specifications required in liquid waste that is intended for surface irrigation or for irrigation of agricultural lands are dealt.

In the meantime, according to the law and regulations concerned, liquid waste is divided into three categories. The first is waste from public sewers that are under the direct control of central or local governmental authorities or public authorities. The second is industrial waste and the third is waste from private sewers. For the first two, surface discharge is allowed, provided that certain specifications are observed.

According to the above-said decree, it is prohibited to cultivate vegetables and fruits which are consumed in a raw state under irrigation utilizing treatment plant effluent.

(j) Effective Utilization of Treatment Plant Effluent for Irrigation

Treatment plant effluent has been utilized for crop irrigation for many years though its utilization is not always worldwide but only in water short areas. The effluent could be a usable water resource if suitable precautions are taken. If properly managed, the nutrients and trace element in the effluent can be asset to agriculture. Decision on utilizing treatment plant effluent must be made based on water, soil and environmental considerations. Judged from the present knowledge and experience, treatment plant effluent can be sufficiently utilized for crop irrigation in areas where water quality fits with the Guidelines as shown in Table 3.6.3.

Utilization of treatment plant effluent usually assumes at least primary treatment consisting on separation, aeration and digestion, and discharge of clear stable liquid. Aside from the above-said problems shown in Table 3.6.2, therefore, it should be considered that problems of diseases, contamination, odours, trace element toxities, along with aesthetic factors are completely avoided for crop irrigation.

Treatment plant effluent is surely a usable supplemental water supply. A well managed irrigation system utilizing treatment plant effluent can reduce the pollution potential as compared to disposal in rivers and other water bodies. As mentioned repeatedly, such a utilization requires a higher level of management. In due consideration of the above-said matter, the agricultural development in the Jarada area has been planned.

In the meantime, a change of quality in the effluent is very important when concentration of the plant nutrients may become higher compared to those necessary for plant growth. A change of irrigation water quality is a simple but drastic solution from the viewpoint of improving water quality. Where different sources of superior water are available a blend can help reduce the possible hazard. Any change in quality due to blending must be evaluated by use of the Guidelines of Table 3.6.3.

According to the study on quality of the effluent derivable from the 'Oxidation Ditch Method' to be actually applied for the project, the amount of total nitrogen is too much for the plant growth, and worked out at 24 mg/l which is classified to 'Degree of Increasing Problem' in the Guidelines mentioned above.

The water usable for belending the treated sewage may be available in and around the Jarada area, and the success of blending may be fully expected. It should be noted, however, that regardless of whether or not the result is acceptable, it will depend to a great extent on the specific situations as to availability of good quality water, overall area water management, long range water quality management, and many other factors. Some of the details for effective use of effluent for irrigation are described in Appendix-Six, Volume Three.

(k) Nitrogen Control

As discussed in the previous sections, lower nitrogen concentration in the effluent may be needed during certain time in the year, particularly during the fruition of certain kinds of crops. To reduce the nitrogen concentration, the blending of the effluent by the groundwater will be the most economical and reliable means, however, if the sufficient belnding water is not available a further advanced sewage treatment process may be considered.

Ammonia nitrogen can be reduced further in concentration or removed from the sewage by several processes. These processes can be classified into two broad categories; biological methods and physical-chemical methods. Most of these processes will hardly be economically justifiable to apply for irrigation purpose, because these will require significant amount of capital investment and operation and maintenance costs togehter with careful management of the system.

The proposed oxidation ditch process has a superior characters in the effective nitrogen control by applying the biological nitrification-denitrification process. Experience gained from the sewage treatment plants under the similar conditions as El-Arish indicates that if the operation of the process is properly made the nitrogen removal rate of as high as 90 per cent or the nitrogen concentration of lower than 10 mg/l can be achieved.

The process is the biological conversion of nitrogenous matter into nitrates (nitrification), followed by the anaerobic biological conversion of the nitrates to nitrogen gas (denitrification). The process is based on the principle that the nitrogen compounds found in the raw sewage may be converted to the nitrate form in a properly operated oxidation ditches. These nitrates may then be removed by further treatment in the absence of oxygen. Underthese anaerobic conditions, the nitrogen is released as nitrogen gas. Because nearly 80 per cent of the atomosphere consists of nitrogen, there will be no air pollution associated with the release of nitrogen from the oxidation ditches to the atomosphere.

The actual operation of the oxidation ditches as the nitrification-denitrification process is that the oxidation ditch may be divided into two distinctive portions, aerobic and anaerobic portions. In order to create such condition, the oxygen supply should be controlled and adjusted to maintain the DO level in the anaerobic portion under anaerobic condition. Effluent from the nitrifying portion is exceptionally free from BODc and for this reason, denitrification may be very slow unless a readily oxidizable source of carbonaceaous matter is added. Methyl alcohol (methanol) may be the cheapest commercial source of carbonaceaous matter at this time. Methanol is preferable because it is more completely oxidized than glucose or other materials and, consequently less sludge for disposal.

The amount of methanol fed must be very closely controlled by a system which is provided with appropriate equipment such as pH controller, methanol dosing equipment, to insure that enough is fed to reduce the nitrates and to avoid an excess. The process will generate no significant added sludge for disposal, nor does it have any objectionable side effects on air or water quality. Moreover, the oxidation ditch process can easily be converted to the nitrification-denitrification method by adding methanol dosing and pH control equipment with a little modification of the original facilities with a small amount of investment.

Although careful experiment will be needed before the design basis is determined for the modification, it may be reasonable assumed that the methanol requirement for a typical domestic wastewater is in the range of 30 to 60 mg/l or 3 to 4 kg of methanol per kg of nitrate nitrogen which is required to consume DO and leave sufficient to reduce the nitrate to nitrogen gas.

Assuming that about a half of the year nitrogen is controlled with a methanol dosing rate of 40 mg/l, the annual operation and maintenance costs for controlling a total of 10,000 m^3/day sewage will be approximately 3 million, excluding that of civil works. The additional equipment for the nitrogen control, consisting of pH control and nitrogen dosing equipment, is roughly estimated to be L.E.100,000.

It should be noted, however, that these costs are estimated based on the above mentioned assumptions and as such the figures are to be considered only as a reference and need to be reviewed at the time when more detailed and reliable information are made available based on the investigations conducted at the Jarada experimental farm lands.

(l) Other Possible Sewage Reuses

The possible treated sewage reuse other than irrigation purpose will be industrial and municipal uses. The following material in this item (l) are to describe briefly some of the possible reuses of the treated sewage, therefore, the purpose is not to give a detailed description of all the possibilities for reuse but to show the major areas of water reuse.

Presently only small scale industries exist in El-Arish area, comprising auto-repair shops, headware stores, concrete products, textile, wood and furniture industries, etc., most of which are non-water consuming industries. Although no precise information are available at present to predict the future development of industries in the area, it may be assumed that the industries in the future consist mainly of food processing and textile industries. The assumed water quality requirements for such industries by the type of use will be as follows:

(in mg/l)

Type of Use	Turbidity	pH	Alkalinity	Hardness	SS	Cl-	Fe	Mn
Food industries								
Cooling	10	7	35	50	75	30	0.1	0.1
Washing	5	7	35	50	80	20	0.1	0.1
Processing	1	7	60	60	80	20	0.1	0.1

(to be continued)

(Continued)

Type of Use	Turbidity	pH	Alkalinity	Hardness	SS	Cl-	Fe	Mn
Food								
Air con.	10	7	50	50	80	30	0.1	0.1
Textile industries								
Cooling	20	7	60	50	200	30	0.1	0.1
Washing	20	7	50	50	200	20	0.1	0.1
Cloth manufacturers								
Air con	20	7	60	60	150	20	0.1	0.1
Processing	20	7	50	50	150	15	0.1	0.1

To meet the above-mentioned requirements, an advanced treatment will be needed beyond the secondary treatment level. Several alternative advanced treatment processes are available including biochemical, physical-chemical, or the combination of both, depending upon the purpose of use and extent of purification. In view of the possible industrial reuse in the area, the physical-chemical treatment processes seem to be desirable. Since most of the comes from domestic wastes and only small amount from commercial activities, heavy metals will not be problematic. The major items to be considered in treating the sewage will be of hardness and chloride ion. These dissolved materials may not be removed by the secondary treatment methods and require an advanced method.

The most efficient means to remove these materials are ion echange, reverse osmosis, and electrodialysis. Of these alternative methods, the reverse somosis method can be most effectively applied to the water with ion echange concentration of 1,000 to 10,000 mg/l, therefore, this process is considered to be superior to other methods on account of the dissolved material concentration of the groundwater. The system may comprise chemical precipitation, sand filter, activated carbon adsorption, and reverse osmosis, as illustrated in the following:

Secondary process effluent → Chemical precipitation → Sand filter → Activated carbon adsorption → Reverse osmosis → Reuse

For reference anticipated performance of unit processes are summarized below:

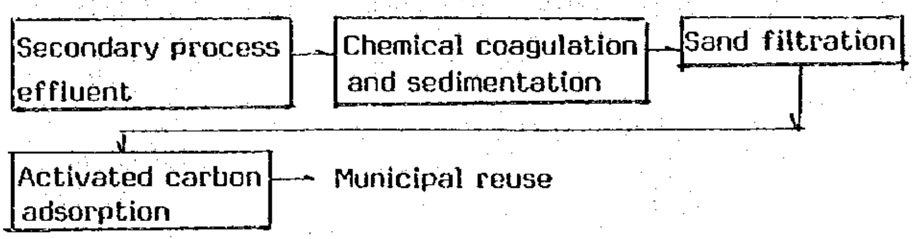
(in %)

Unit process	SS	BOD	NH_3	NO_3	COD	Org.C	Dissolved solids
Rapid sand filtration	70 - 80	50 - 70	-	-	-	-	-
Ultra filtration	100	30 - 40	-	-	30 - 35	25 - 30	-
Granular activated carbon	90	80	-	45	80	80	-
Chemical coag. micro strainer	90 - 95	100	-	-	48	40	-
Ozon, sand bed, activated carbon	90 - 95	50	-	-	47	45	-
Ion exchange activ. carbon	-	-	82	88	-	-	89
Electrodialysis	-	-	43	50	15	-	34
Reverse osmosis	100	98	90	50	98	95	90

Source: R.W.Bayley and A. Waggott, Water Pollution Control 45(1972)

Direct reuse of the treated sewage as drinking water, after dilution in natural waters to the maximum possible extent and after coagulation, filtration, and heavy chlorination for disinfection, may be practicable on an emergency basis. Advanced methods of sewage, such as demineralization and desalination, are capable of almost complete removal of impurities, and water treated by such methods, after chlorination, may be safe to drink. These methods are very expensive and, where they are found to be necessary due to inadequate water supplies, may be economically feasible only if a dual supply system is adopted. In such cases, adequately treated and disinfected sewage effluent could be reused for flushing toilets, yard watering, and other direct applications.

For the advanced treatment of the oxidation ditch process effluent, the following process may be applied to produce the water acceptable for municipal reuse:



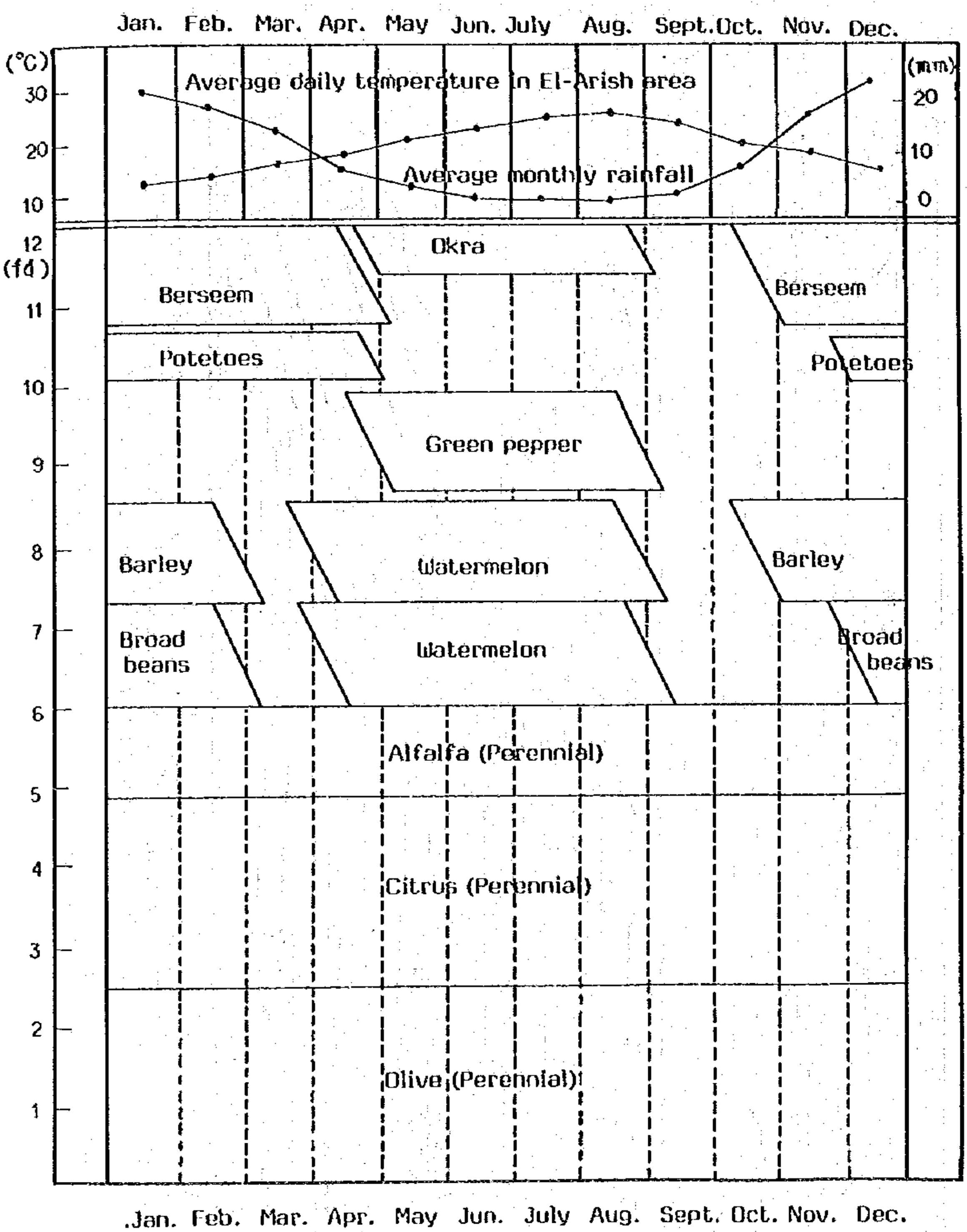
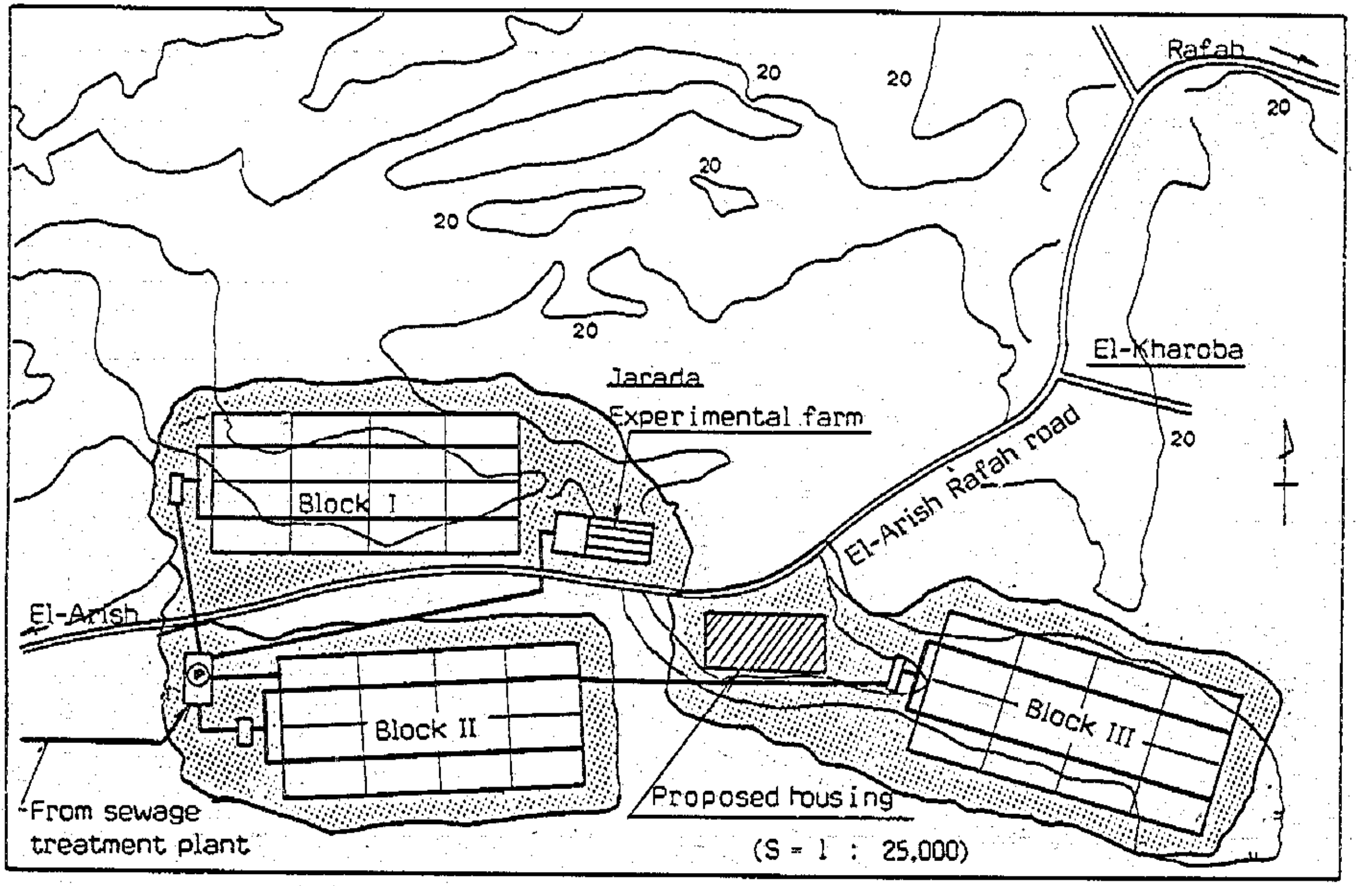


Figure 3.6.1. Proposed Cropping Pattern (12-Fedan-Farm Unit)



Remarks: Proposed tree-planting area. In some parts of the area forage crops are cultivated for livestock rearing utilizing surplus of the treatment plant effluent, especially during the winter season

Figure 3.6.2. Location of Water Management Blocks, Experimental Farm, Housing Site and Others in the Proposed Jarada Area

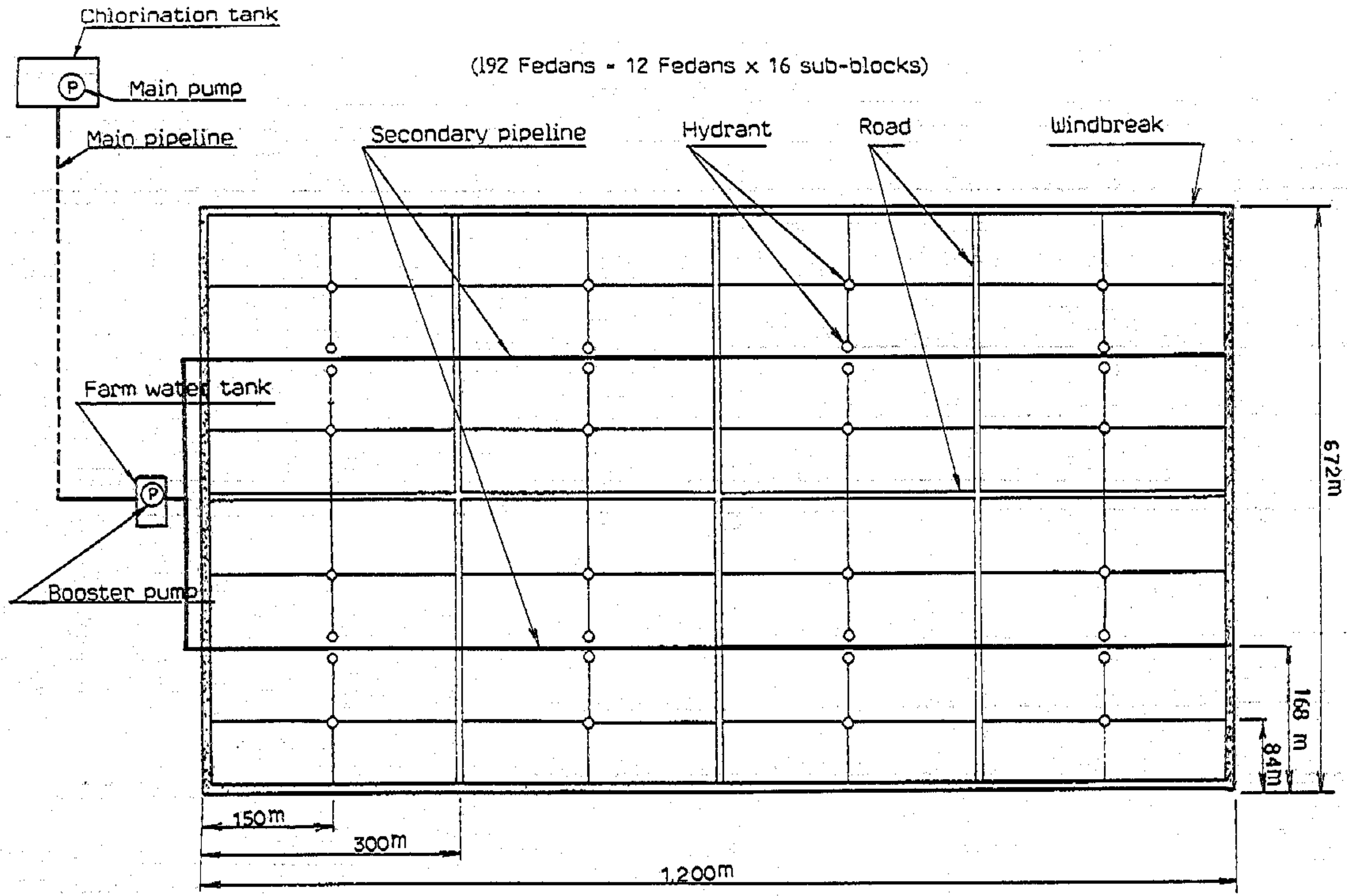


Figure 3.6.3. Layout Plan of Water Management Block

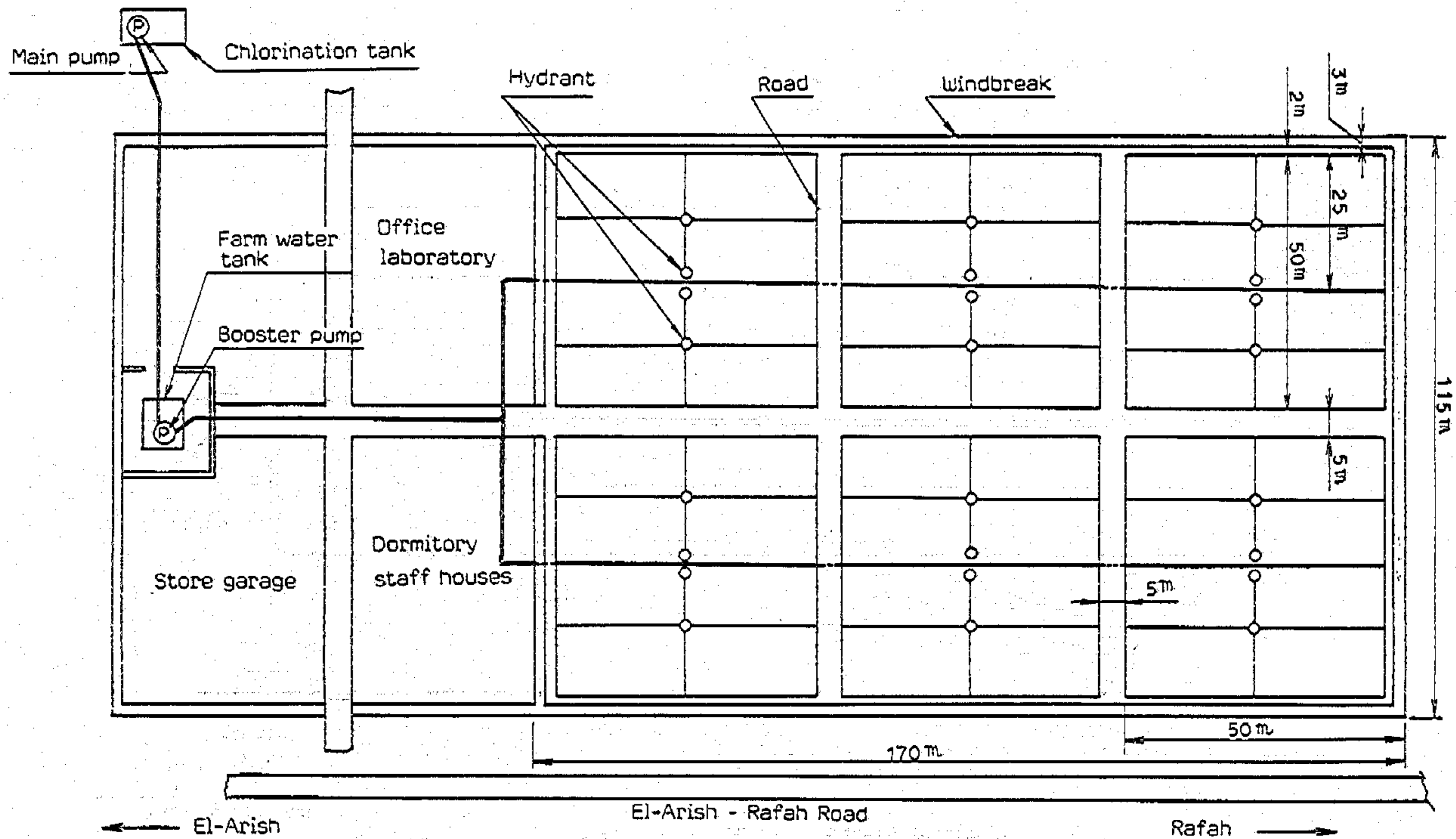


Figure 3.6.4. Layout Plan of Experimental Farm

Table 3.6.2 Existing Standards Governing the Use of Renovated Water in Agriculture

Item	California, U.S.A.	Israel	South Africa	Federal Republic of Germany
Orchards and vineyards	Primary effluent; no spray irrigation; no use of dropped fruit.	Secondary effluent.	Tertiary effluent, heavily chlorinated where possible. No spray irrigation.	No spray irrigation in the vicinity.
Fodder, fibre crops, and seed crops.	Primary effluent; surface or spray irrigation.	Secondary effluent, but irrigation of seed crops for producing edible vegetables not permitted.	Tertiary effluent.	Pretreatment with screening and setting tanks. For spray irrigation, biological treatment and chlorination.
Crops for human consumption that will be processed to kill pathogens.	For surface irrigation, primary effluent. For spray irrigation, disinfected secondary effluent (no more than 23 coliform organisms per 100 ml).	Vegetables for human consumption not to be irrigated with renovated wastewater unless it has been properly disinfected (1000 coliform organisms per 100 ml in 80% of samples).	Tertiary effluent.	Irrigation up to 4 weeks before harvesting only.
Crops for human consumption in a raw state.	For surface irrigation, no more than 2.2 coliform organisms per 100 ml. For spray irrigation, disinfected, filtered wastewater with turbidity of 10 units permitted, providing it has been treated by coagulation.	Not to be irrigated with renovated wastewater unless they consist of fruits that are peeled before eating.		Potatoes and cereals - irrigation through flowring stage only.

Source: World Health Organization (1973) Reuse of Effluent: Methods of Wastewater Treatment and Health Safeguards

Table 3.6.3 Guidelines for Interpretation of Water Quality for Irrigation

Irrigation Problem	Degree of Problem: No Problem	Increasing Problem	Severe Problem
Salinity (affects crop water availability) ECw(mmhos/cm)	< 0.75	0.75-3.0	> 3.0
Permeability (affects infiltration rate into soil) ECw(mmhos/cm)	> 0.5	0.5 -0.2	< 0.2
adj. SAR 1/ 2/			
Montmorillonite (2:1 crystal lattice)	< 6	6- 9 3/	> 9
Illite-Vermiculite (2:1 crystal lattice)	< 8	8-16 3/	> 16
Kaolinite-sesquioxides (1:1 crystal lattice)	< 16	16-24 3/	> 24
Specific Ion Toxicity (affects sensitive crops)			
Sodium 4/ 5/ (adj. SAR)	< 3	3-9	> 9
Chloride 4/ 5/ (meq/l)	< 4	4-10	> 10
Boron (mg/l)	< 0.75	0.75-2.0	> 2.0
Miscellaneous Effects (affects susceptible crops)			
NO_3-N (or) NH_4-N (mg/l)	< 5	5-30	> 30
HCO_3 (meq/l) (overhead sprinkling	< 1.5	1.5-8.5	> 8.5
	(Normal range : 6.5 - 8.4)		

1/ adj. SAR means adjusted Sodium Adsorption Ratio.

2/ Values presented are for the dominant type of clay mineral in the soil since structural stability varies between the various clay types. Problems are less likely to develop if water salinity is high; more likely to develop if water salinity is low.

3/ Use the lower range if ECw=0.4 mmhos/cm;
Use the intermediate range if ECw=0.4-1.6 mmhos/cm;
Use upper limit if ECw=1.6 mmhos/cm.

4/ Most tree crops and woody ornamentals are sensitive to sodium and chloride. Most annual crops are not sensitive.

5/ With sprinkler irrigation on sensitive crops, sodium or chloride in excess of 3 meq/l under certain conditions has resulted in excessive leaf absorption and crop damage.

Source: World Health Organization (1973) Reuse of Effluent: Methods of Waste-water Treatment and Health Safeguards

3.6.3 Potential Farm Lands for Plant Effluent Reuse

As a chain of the sewerage system project for the El-Arish city, an identification investigation on selecting potential farm lands for utilizing treatment plant effluent as irrigation water has been carried out in four areas located in and around the El-Arish city called 'Lower Wadi El-Arish', 'Area adjoining Lower Wadi El-Arish on the west', 'Middle Wadi El-Arish' and 'El-Arish - Rafah strip area', respectively. Locations for four alternative areas are illustrated in Figure 3.6.5.

These areas have already been identified as candidate farm lands by the the Governorate of North Sinai as well as the Ministry of Development, and the respective estimated net acreage that can be reclaimed are 7,000 feddans, 10,000 feddans, 25,000 feddans and 25,000 feddans.

According to our investigation, in the former three areas excluding 'El-Arish - Rafah strip area', much rainfall runoff for irrigation is expected during the winter season, and also groundwater is available though the salinity is relatively high and the quantity is limited.

On the other hand, in 'El-Arish - Rafah strip area', low salinity groundwater is obtainable though limited to specified sub-areas. The soil conditions are relatively superior to those of the former three areas judged from the soil survey results, and fortunately much annual rainfall is expected compared to those of the former three areas.

Jarada area is located about 10 km eastwards of the El-Arish city, close to the El-Arish - Rafah road. The area occupies one of the most important parts of the 'El-Arish - Rafah strip area', and is the most desirable candidate farm lands specified by the Governorate of North Sinai. In the area, until a few years ago, irrigation farming had been conducted by Israeli people. However, after the withdrawal of Israeli army, the farming was abondoned due mainly to shortage of irrigation water. Presently, rain-fed farming is partly found only during the winter season.

As mentioned already, the former three areas are candidate farm land identified by the Governorate of North Sinai. However, most of areas are undeveloped desert lands, and it is considered that for the new reclamations, a great amount of investment is required. Compared to this, in the 'El-Arish - Rafah strip area', especially in most of the Jarada area, such a great amount of investment might be not required because of already cultivated farm lands.

In due consideration of the socio-economic conditions as well as the above features, the Jarada area has been selected as the farm land for the effluent reuse.

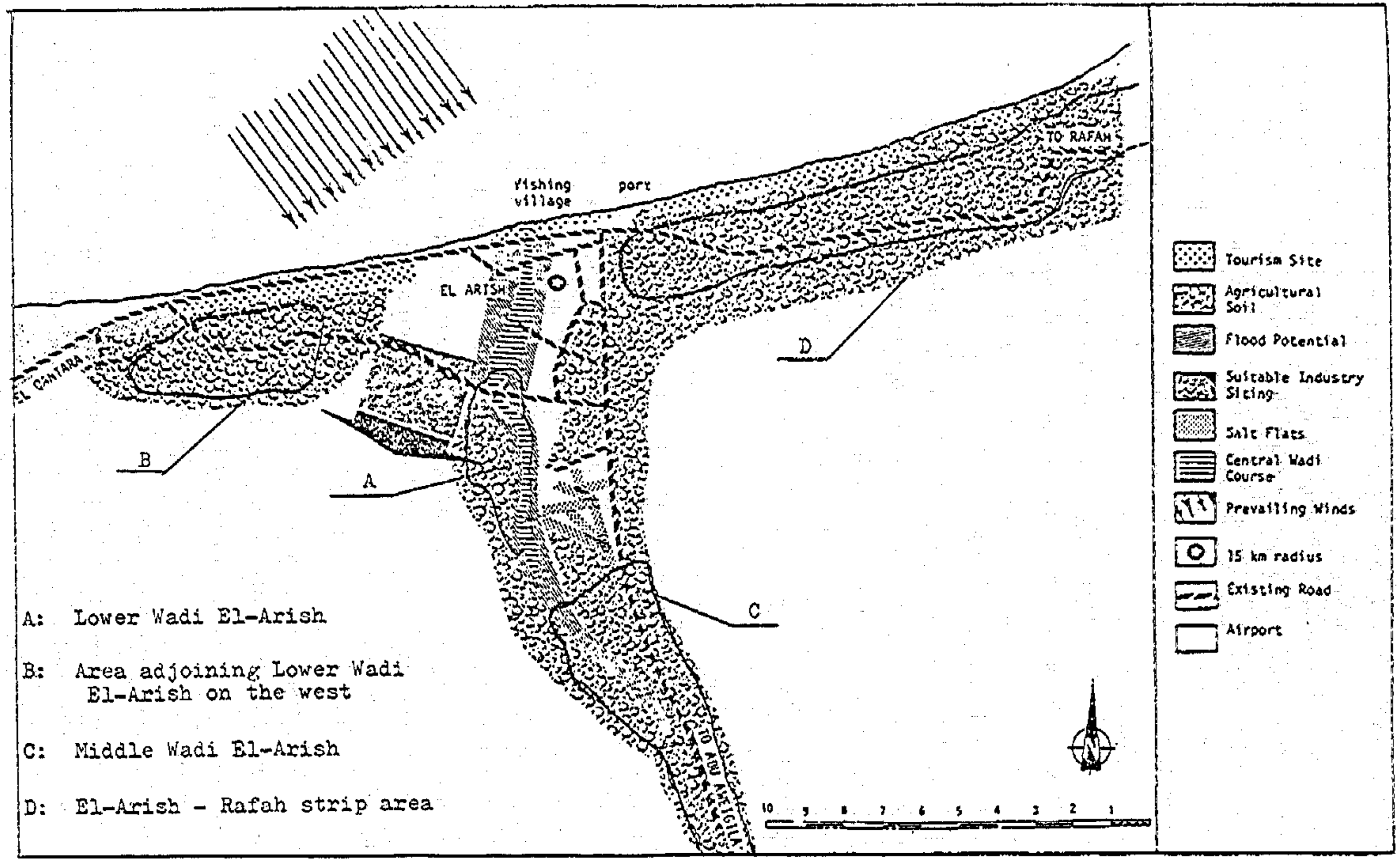


Figure 3.6.5. Potential Farm Lands Identified by the Government of Development

3.6.4. Sewer Routes and Locations of Pumping Station

As described in Section 2.5., El-Arish City is situated on the sand dune of the Wadi El-Arish river-mouth. The area slowly declines towards the Wadi El-Arish with noticeable undulations. Due to these topographic and geological features prevailing in the area, pumping facilities are unavoidable to lift the wastewater in the sewerage system.

Preliminary layout of the sewerage system has been determined on the basis of the actual topographic surveys and intensive field investigations under the study, so that the sewers will in general slope in the same direction as the street or ground surface. District boundaries conform to watershed or drainage basin areas, but at the locations where development is expected in the immediate future and the topographic nature necessitate to discharge the wastewater into the sewerage district, the wastewater quantities were considered in sewer capacities downstream. A general map indicating the locations of main sewers and pumping stations are shown in Figure 3.6.6.

(a) Main Sewer Routes

The main sewer routes have been determined taking into account the economy of the overall sewerage system, so as to avoide excessively deep excavations and structures for the submain sewers entering the main sewers, which, in turn, would affect the necessary branch and lateral sewers. The recommended system is based on construction of open-cut sewers covering the entire sewerage planning area. The main sewers thus determined comprise five main sewer routes. For the purpose of identification, each main sewer has been named as indicated below with a brief description of the particular features.

- Beach line; The main sewer line receives the wastewater from the urban city area of El-Arish, area along the beach, Abu Saghal, and Salem areas, leading the wastewater to the sewage treatment plant at Jarada area. This is the largest main sewer finally intercepting all the wastewater from the sewerage districts.
- Wadi line; This sewer runs from the southern part of El-Arish City towards the north through the left bank of the Wadi El-Arish.

- Central line; This sewer intercepts the wastewater from the western and central portion of El-Arish City and flows towards the Wadi El-Arish.

- Salem line; This sewer receives the wastewater mainly from the Salem housing complex and their environs, running on the road west of the Salem area.

- Masaid line; This sewer receives the wastewater from Masaid housing complex and lead it to the existing sewage treatment plant located at southwest of the Masaid area.

(b) Wastewater Pumping Stations

Locations of wastewater pumping stations have been determined based on the studies on the balance of the costs between the deep sewers without pump provision or shallower sewers with pump station. Besides, such factors as availability of land and topographic features have been taken into account for selecting the pumping station locations. The wastewater pumping stations finally selected are as follows:

- Main pumping station; East of the Wadi El-Arish river mouth.

- El-Risa pumping station; In between the Main pumping station and the Jarada sewage treatment plant.

For low-lying areas scattered in the sewerage planning area, small pumping facilities are to be installed to lift the wastewater, instead of providing deeper main sewers. Locations of these pumping stations are indicated in Volume Three 'Drawings.'

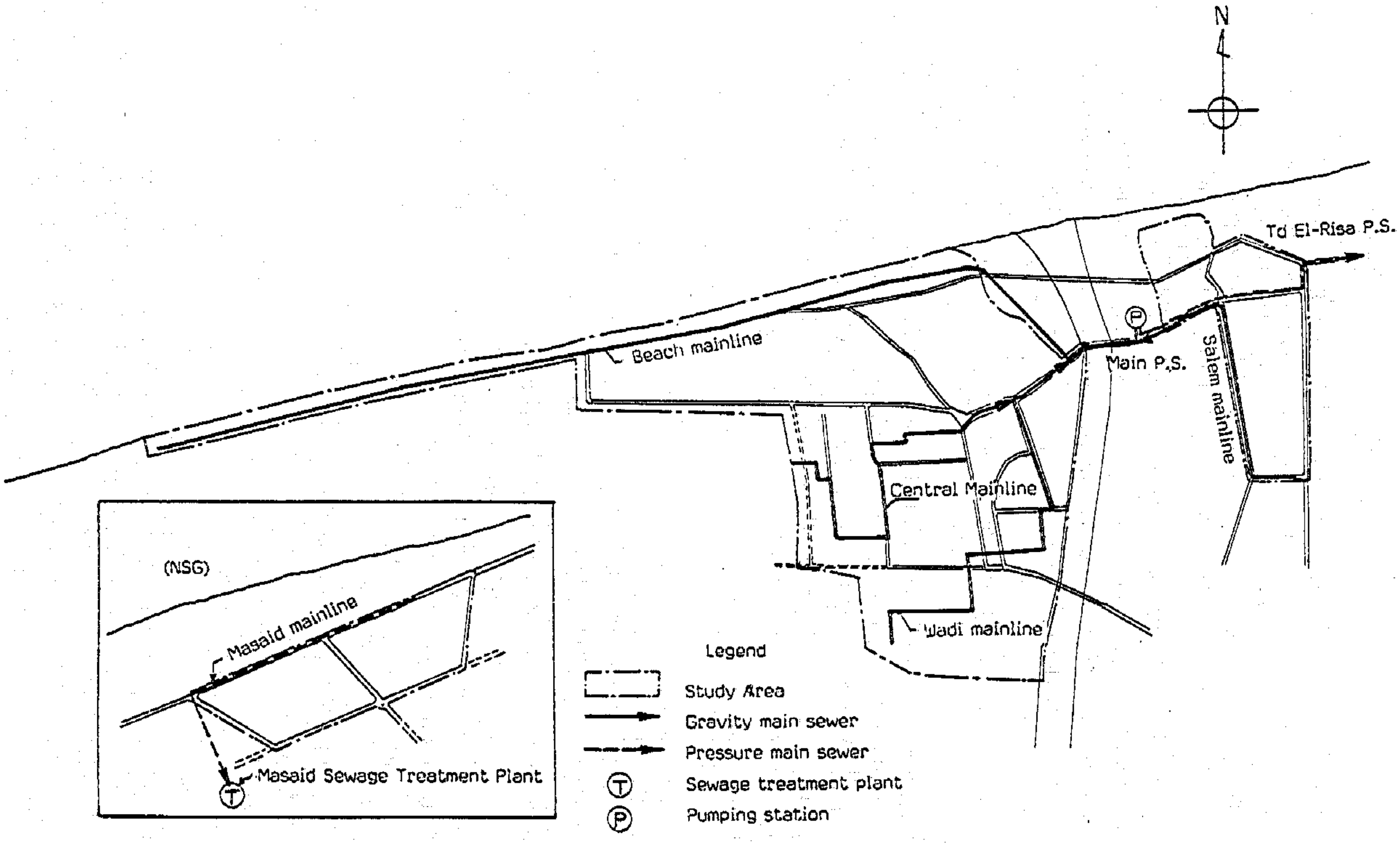


Figure 3.6.6. Sewerage System Components

3.6.5. Sewage Treatment Process Selection

(a) Alternative Processes Considered

The following six possible alternative biological treatment methods have been studied to select the most desirable treatment process:

- Conventional activated sludge
- Extended aeration
- Oxidation ditch
- Modified aeration
- Aerated lagoon
- Oxidation pond

The conventional activated sludge process comprises grit chambers, primary sedimentation tanks, aeration tanks, final sedimentation tanks, chlorine contact tanks, sludge thickeners, unheated sludge digesters, and sludge drying beds. Because of the possible intrusion of sand into the plant equipment, surface aerators are applied to the aeration tanks. Digested sludge in the digesters is dried on the sludge drying sand beds. The expected BOD removal efficiency is 90 or higher per cent when the system is properly operated. The flowsheet and layout plan of the conventional activated sludge process are shown in Figures 3.6.7. and 3.6.8. respectively.

The extended aeration process operates in the endogenous respiration phase of the bacterial growth cycle, which occurs when the BOD loading is so low that organisms are starved and undergo partial auto-oxidation. Because of the oxidation of more volatile solids during the long sludge retention time, the waste sludge production is relatively low. The hydraulic retention time in the aeration tank is about 24 hours. As shown in Figures 3.6.9 and 10, this process consists of grit chamber, aeration tank, final sedimentation tank. chlorine contact tank, sludge storage tank, and sludge drying sand bed, but primary sedimentation tank is omitted. BOD removal efficiency is expected to be 90 per cent or higher if the system is properly operated.

The oxidation ditch process is an extended aeration consisting of a ring-shaped channel about 2.5 m deep and other facilities same as those for the extended aeration process. An aerator is placed across the ditch to provide aeration and circulation of the sewage. BOD removal efficiency is almost same as that of the extended aeration process. The flowsheet and layout plan of the process are illustrated in Figures 3.6.11. and 3.6.12. respectively.

The flowsheet of the modified aeration process is identical with that of the conventional process. The difference in the system is that the modified aeration uses shorter aeration times, usually 1.5 to 3 hours. Because of the low MLSS concentration and low requirement of air, the energy cost of the process is generally low, but the resultants BOD removal is in the range of 60 to 75 per cent. The flowsheet of the process and the layout plan of the treatment plant facilities are shown in Figures 3.6.13. and 3.6.14., respectively

An aerated lagoon is a basin in which wastewater is treated on a flow-through basis. Oxygen is supplied by means of surface aerators. The action of the aeration and that of the rising air bubles from the diffuser is used to keep the contents of the basin in suspension. As illustrated in Figures 3.6.15. and 3.6.16., the process consists of aerated lagoon and maturation ponds, but the aeration is carried out in the aerated lagoons. Bacterial degradation of contaminants and removal of additional BOD will undergo in the maturation ponds. When designed and operated properly, this process will achieve a high performance and the better effluent than other processes can be expected in terms of bacteria removal.

Odidation ponds are large, shallow ponds in which organic wastes are decomposed by micro-organisms in a combination of natural processes involving both bacteria and algae. Oxidation ponds are one of the most economical method of sewage treatment wherever land is available at relatively low cost. Their principal advantages are that they remove excreted pathogens at a much lower cost than any other form of treatment and that they have minimum operating and maintenance requirements. As shown in Figures 3.6.17. and 3.6.18., this process consists of facultative ponds and maturation ponds with necessary auxiliary facilities to remove the BOD to the level of 90 per cent or higher.

For evaluation of the six alternative processes, a detailed comparative study has been made on each of the alternative plans with regard to :

- Land requirements
- Capital costs

- Operation and maintenance costs
- Characteristics of operation and others

For the comparison purpose, a layout plan for each of the treatment processes with a treatment capacity of 20,000 m3/day has been prepared and the required land area and costs estimated as discussed in the succeeding paragraphs.

(b) Land Requirements

On the basis of the layout plans for the alternative treatment processes, the required land area and costs for acquisition for each of the processes have been estimated as summarized in Table 3.6.4. For the estimation of the required land area, a buffer zone of 10 or more metres is provided for aesthetic purpose. The zone will be planted with shrubs and trees to landscape the area and to reduce the odour and noise expected from the plant facilities. For land cost estimation, an average unit land cost of L.E. 10 is used.

Table 3.6.4. Land Requirements of Treatment Processes

Type of Process	Required Land Area (m2)	Cost (L.E. 1,000)
Conventional activated sludge	31,600	31.6
Extended aeration	39,600	39.6
Oxidation ditch	46,400	46.4
Modified aeration	30,600	30.6
Aerated lagoon	100,600	100.6
Oxidation pond	557,800	557.8

Land requirements for the different treatment processes are illustrated in Figure 3.6.19.

(c) Capital Costs

Capital costs of civil works, mechanical equipment and electrical equipment for each of the alternative processes have been estimated as summarized in Table 3.6.5. The costs are direct expenditures required for construction of facilities, covering civil works, buildings, mechanical and electrical equipment, pipings, plant facilities, site roads, parkings, sotres, etc. Capital costs for the different treatment processes are shown in functions in Figure 3.6.20.

Table 3.6.5. Initial Costs of Alternative Processes

(unit in L.E. 1,000)

Type of Process	Capital Costs			
	Civil works	Mechanical equipment	Electrical equipment	Total
Conventional activated sludge	3,679	3,200	920	7,799
Extended aeration	4,435	2,571	907	7,913
Oxidation ditch	4,557	2,185	760	7,502
Modified aeration	3,508	2,911	878	7,297
Aerated lagoon	4,415	2,938	801	8,154
Oxidation pond	11,121	238	70	11,429

(d) Operation and Maintenance Costs

Operation and maintenance costs of sewage treatment works comprise those for electricity and other energy, operation and maintenance labour, supplies and maintenance materials, raw materials, chemicals, administration and staff, etc. For each process these costs have been estimated as summarized in Table 3.6.6.

Table 3.6.6. Annual Average Operation and Maintenance Costs of Alternative Processes

(unit in 1,000 L.E./yr)

Type of Process	O & M Costs			
	Labour	Energy	Others	Total
Conventional activated sludge	15.4	60.8	3.0	79.2
Extended aeration	13.1	75.3	3.5	91.9

(to be continued)

(continued)

Type of Process	O & M Costs			
	Labour	Energy	Others	Total
Oxidation ditch	13.1	111.1	5.0	129.2
Modified aeration	15.4	42.7	2.3	60.4
Aerated lagoon	12.4	35.3	1.9	49.6
Oxidation pond	11.3	0.5	0.5	12.3

Operation and maintenance cost for each of the treatment processes is shown in curves and functions in Figure 3.6.21.

(e) Cost Effectiveness

On the basis of the foregoing estimates for each of the alternative plans, a cost effectiveness analysis has been made on each of the alternative plans. For the purpose of comparison, constant annual costs have been estimated considering the following conditions:

- Service life; civil works and buildings 50 years
  mechanical and electrical equipment 15 years
- Salvage value; civil and building works 0
  mechanical and electrical equipment 10 %
- Discount rates: 3.5, 8.0 and 15.0 per cent per year

The results of the estimates are summarized in the following table.

Table 3.6.7. Annual Costs of Alternative Processes

(1,000 L.E./yr)

	C.A.S.	ET.A.	OX.D.	MD.A.	A.L.	O.P.
Interest at 3.5%	274.1	278.3	264.2	256.5	288.9	419.5
Depreciation	320.8	297.4	267.8	297.5	327.8	392.1
O & M	79.2	91.9	129.2	60.4	49.6	12.3
Total annual cost	674.1	667.6	661.2	614.4	666.3	823.9

(to be continued)

(continued)

	C.A.S.	E.T.A.	O.X.D.	M.D.A.	A.L.	O.P.
Interest at 8%	626.4	636.2	603.9	586.2	660.4	958.9
Depreciation	320.8	297.4	267.8	297.5	327.8	392.1
O & M	79.2	91.9	129.2	60.4	49.6	12.3
Total annual cost	1,026.4	1,025.5	1,000.9	944.1	1,037.6	1,363.3
Interest at 15%	1,174.6	1,192.9	1,132.3	1,099.1	1,238.2	1,798.0
Depreciation	320.8	297.4	267.8	297.5	327.8	392.1
O & M	79.2	91.9	129.2	60.4	49.6	12.3
Total annual cost	1,574.6	1,582.2	1,529.3	1,457.0	1,615.6	2,202.4

Note: C.A.S. - Conventional activated sludge. E.T.A. - Extended aeration. O.X.D. - Oxidation ditch. M.D.A. - Modified aeration. A.L. - Aerated lagoon. O.P. - Oxidation pond.

Based on the above estimated costs for the alternative plans, the unit sewage treatment costs by each of the alternatives have been calculated as shown in the following table.

Table 3.6.8. Per Unit Volume Treatment Costs by Treatment Type

(P.T./m3)

Type of Treatment	Discount Rates (%/yr)		
	3.5	8.0	15.0
Conventional activated sludge	9.2	14.1	21.6
Extended aeration	9.1	14.0	21.7
Oxidation ditch	9.1	13.7	20.9
Modified aeration	8.4	12.9	20.0
Aerated lagoon	9.1	14.2	22.1
Oxidation pond	11.3	18.7	30.2

As may be seen from the above table, the per unit sewage treatment costs by the oxidation pond are highest among the alternative plans, due mainly to it high construction. Among the alternatives the costs by the modified aeration system are the lowest at three different discount rates followed by the oxidation ditch process.

(f) Evaporation of Sewage

One of the factors to be considered in selecting a treatment process in an arid or semi-arid region like El-Arish is the effect of evaporation of the sewage from plant facilities. The daily average sewage evaporation from each process has been estimated using an average evaporation rate of 4.6 mm (Ref. No.7) as shown in the following table.

Table 3.6.9. Sewage Evaporations from Treatment Facilities

Type of Treatment Process	Amount of Evaporation (m3/day)	Percentage of Loss (%)
Activated sludge	12	0.06
Extended aeration	25	0.13
Oxidation ditch	43	0.22
Modified aeration	9	0.05
Aerated lagoon	308	1.50
Oxidation pond	2,252	11.30

Because the amount of sewage evaporation increases in proportion to the surface area of tanks or other facilities exposed to the air, the evaporation from the oxidation pond system is the highest among the alternatives, losing 11.3 per cent of the total sewage inflow. While the sewage loss by the modified aeration process is the lowest, losses from other processes, except from the oxidation pond, are not so high. This makes the oxidation pond inferior to other processes.

(g) Process Evaluation

As discussed hereinabove, characteristics of each alternative process may be summarized as follows:

- As may be seen from Table 3.6.7. the annual expenditures of the modified aeration process at three different interest rates are the least costly among the six alternatives; however, this process is not appropriate to apply to the sewerage system because this method is in general suitable for treating the sewage in low concentration. Besides, the BOD removal efficiency ranges at around 60 per cent which is considerably lower than other processes. This low removal efficiency may produce the effluent not suitable for the crop irrigation in view of public health problems and may also cause clogging in drip irrigation nozzles and pielines. The operation and maintenance of this process is not necessarily simple in spite of its apparent simplicity and low organic loading removal efficiency. The low over-all efficiency of the process is subject to produce the effluent with a high BOD concentration, which may not meet the require-ments as set under the Water Pollution Control Standards requiring all the industrial wastewater effluents be 60 or lower mg/l in terms of BOD. For these reasons, the modified aeration method is eliminated for further scrunity.

- The land requirements for treatment facilities are widely at variance with the type of treatment process. As shown in Table 3.6. , the oxidation pond requires the widest land area followed by the aerated lagoon. The land requirements also vary due to the difference in wastewater detention time within the plant facilities. The extended aeration and oxidation ditch methods have almost the same capacity of facilities, but the land requirement for the extended aeration system is approximately 87 per cent of the oxidation ditch because of the shallower depth in the oxidation ditch aeration basins.

- Mechanical equipment costs varies by the type of treatment process. The conventional activated sludge process having more sophisticated unit processes is the most costly among other alternatives. The costs for electrical equipment, however, appear to differ not so significantly among the other alternative plans except those for the oxidation pond because these costs are governed in general by the power loads of the plant rather than treatment process per se.

- Among the alternative processes, the oxidation pond method has the least complexity in operation and maintenance of the system followed by the aerated lagoon, whereas the operation of the conventional activated sludge system is rather complex. Both oxidation ditch and extended aeration systems require less complex operation and maintenance procedures than in the activated sludge, while the oxidation pond system requires the least care in operation. As shown in Table 3.6.6., the operation and maintenance cost for the extended aeration system is the highest, whereas other systems are more or less the same except that for the oxidation pond process. This may be explained that the power requirements in the aeration tanks are high in the extended aeration system. The relatively high power require ments in the aerated lagoon is due mainly to the higher power requirements of surface aerators.

- One of the advantages accruing to the activated sludge system is the possible utilization of sludge gas for power generation at the treatment plant. The gas to be produced in the activated sludge process of this scale may not be viable, particularly during the stages when the sewage inflow is expected to be low. Sludge digestion is in general not needed in oxidation ditch and extended aeration, but when it becomes really necessary to digest the excess sludge for the reason of disposal or another, sludge treatment process can be easily added to the plant. For small treatment plants, most of types of sludge processing are too complicated and require a high level of experience than is usually available, and for these reasons, sludge treatment facilities are not planned for the alternative plans except for the activated sludge process.

- The conventional activated sludge and modified aeration processes are not resistant to shock organic or toxic loadings, while other methods are in general resistant to such loadings.

- The treatment cost per unit volume of sewage by the oxidation ditch system is the lowest except that of the modified aeration process. This means the oxidation ditch process is the most cost effective among the alternative plans eligible for adoption to El-Arish sewerage system.

- As mentioned in item (f), the amount of evaporation from the aerated lagoon and oxidation pond are high compared with other alternative plans. Thereby, these two processes are inferior to other systems from the viewpoints of effective use of water sources.

For the convenience of the evaluation of the alternative processes, the discussions described hereinabove are summarized by item of evaluation, including both tangible and intangible factors, as shown in Table 3.6.10.

(g) Conclusions

In selecting a treatment process each alternative has been scruitinized in the light of the following four major factors:

i) Cost effectiveness
ii) Simplicity of process
iii) Ease of process operation and maintenance
iv) Treatment efficiency and reliability of operation and maintenance of process

With reference to annual costs of the processes, the modified aeration is the least expensive among the alternatives, however, as already described, BOD and other organics removal efficiencies of this process are lower than other methods, thus this process does not meet the requirements under the above items ii), iii) and iv). These requirements are particularly important for operation crews to properly operate and maintain the new treatment plant system. Besides, for the purpose of sewage effluent reuse, the product of this process will not be appropriate because of its low quality. Also, this process involves rather sophisticated process operation and requires careful operation and maintenance procedures in spite of its low efficiency. For these reasons, the modified aeration process has been screened out for further evaluation.

Among the remaining five alternatives, the oxidation pond will be the only alternative which meets the requirements of the above items ii), iii) and iv). This process is the most costly because of the high costs for prevention ofsewage exfiltration from ponds. Under the climatic conditions of the region, evaporation of the sewage from the pond surface will be significant thus making this method less advantageous than other methods. A total water loss of more than 11 per cent is too high in consideration of the efficient use of valuable water resources. For the same reasons above, both oxidation pond and aerated lagoon processes have been excluded from further consideration.

From the foregoing discussions, the best alternative is to be selected among the activated sludge, oxidation ditch and extended aeration. Of the three alternatives, the oxidation ditch process has many advantages both tangible and intangible, and considered superior to other two alternatives for the reasons mentioned below:

- The annual costs of the oxidation ditch process is the lowest.
- Process of the oxidation ditch system is simple and operation and maintenance of the facilities are easier than other methods.
- Sludge production is small and the activated sludge is easily dried on sand beds.
- Relatively high organics removal efficiencies expected.

In a sewage treatment system in desert region, sand accumulation in the plant facilities and water loss due to the evaporation are important factors to be considered. The evaporation loss of sewage from the oxidation ditch is estimated to be only 0.2 per cent which is negrigible level. In preventing the possible sand intrusion to the facilities, structures will be provided sufficiently higher than ground elevation, and sufficiently wide buffer zones with grasses and trees around facilities and boundary of the plant.

In view of the above discussions and the results of alternative study, it is concluded that the sewage treatment plant by the oxidation ditch process be adopted for El-Arish sewerage system.

Table 3.6.10. Comparison of Alternative Processes.

(Unit 10^3 L.E)

Item / Treatment Process	Characteristics	Operation & Maintenance (O/M): Easiness	Operation & Maintenance (O/M): Number of Checking Points	Operation & Maintenance (O/M): High Technology Requirement	Rate of BOD Removal	Evapo-ration rate to inflow	Land area (m^2)	Land cost	Constr-uction Cost	O/M cost (year)	Capital cost (year)	Treatm-ent cost (year)	Remark
Conventional Activated Sludge Process	1) High treatment efficiency, 2) widely used process 3) Con-struction & O/M costs are comparably economical, 4) Plant occupied area is rather small 5) Not capable for variation of inflow sewage quality 6) Higher production of sludge 7) Complicated plant operation 8) Sludge driving bed system is not suitable (sludge digestion tank required) 9) Difficulty in aeration due to sand accumulation	Comparably Easy	Many	Required	> 90%	0.06%	31,600	31.6	7,799	79.2	594.9	674.1	Most popular treat-ment process, especially in large plants
Extended Aeration Process	1) Primary sedimentation is not required 2) Capable for variation of inflow for various ubfkiw sewage quality 3) Sludge generation is low 4) Advanced in N removal 5) Low treatment efficiency in SS 6) Odor production is high 7) Large plant area is required 8) Carry over of sludge is high 9) Difficulty in aeration due to sand accumulation	Easy	Less	Required	> 90%	0.13%	39,600	39.6	7,913	91.9	575.7	667.6	Process developed for less sludge generation, suitable to small plants
Oxidation Ditch Process	1) Primary sedimentation is not required 2) Capable for various inflow sewage quality 3) Sludge generation is low 4) O/M is rather simple 6) Comparably large plant area is required 7) Odor production is high 8) Carry over of sludge is high	Easy	Less	Less Required	> 90%	0.22%	46,400	46.4	7,502	129.2	532.0	661.2	Process developed for high treatment efficiency with low O/M costs and less mechanical facility, suitable for small plants
Modified Aeration Process	1) Suitable for low BOD load inflow sewage treatment 2) Most economical in construction & O/M costs 3) Changeable to conventional activated sludge process 4) Not capable for variation of inflow sewage quality 5) Low treatment efficiency 6) Sludge drying bed system is not suitable (Sludge digestion tank required	Comparably Easy	Many	Required	± 60%	0.05%	30,600	30.6	7,297	60.4	554.0	614.4	Process is not for high level treatment, different from others
Aerated Lagoon Process	1) Most simple in O/M (Mechanical portion is small) 2) No return sludge required 3) Effective in bacteria treatment 4) Large plant area is required (Next to oxidation pond process) 5) Algal growth problem in reuse 6) Problems in evapolation and sand accumulation 7) Large amount of safty fences required	Very Easy	Less	Not Required	> 90%	1.5%	86,800	86.8	8,154	49.6	616.7	666.3	Process is developed with oxidation pond with mechanical aeration facility
Oxidation Pond Process	1) Most simple in O/M (Mechanical portion is smallest) 2) Effective in bacteria treatment 3) Largest plant area is required 4) Algal growth problem in reuse 5) Problems in evapolation and sand accumulation 6) Large amount of safty fences required	Very Easy	None	Not Required	> 90%	11.3%	557,800	557.8	11,429	12.3	811.6	832.9	Most simple old process but required large land area, suitable for hot climate regeon plants

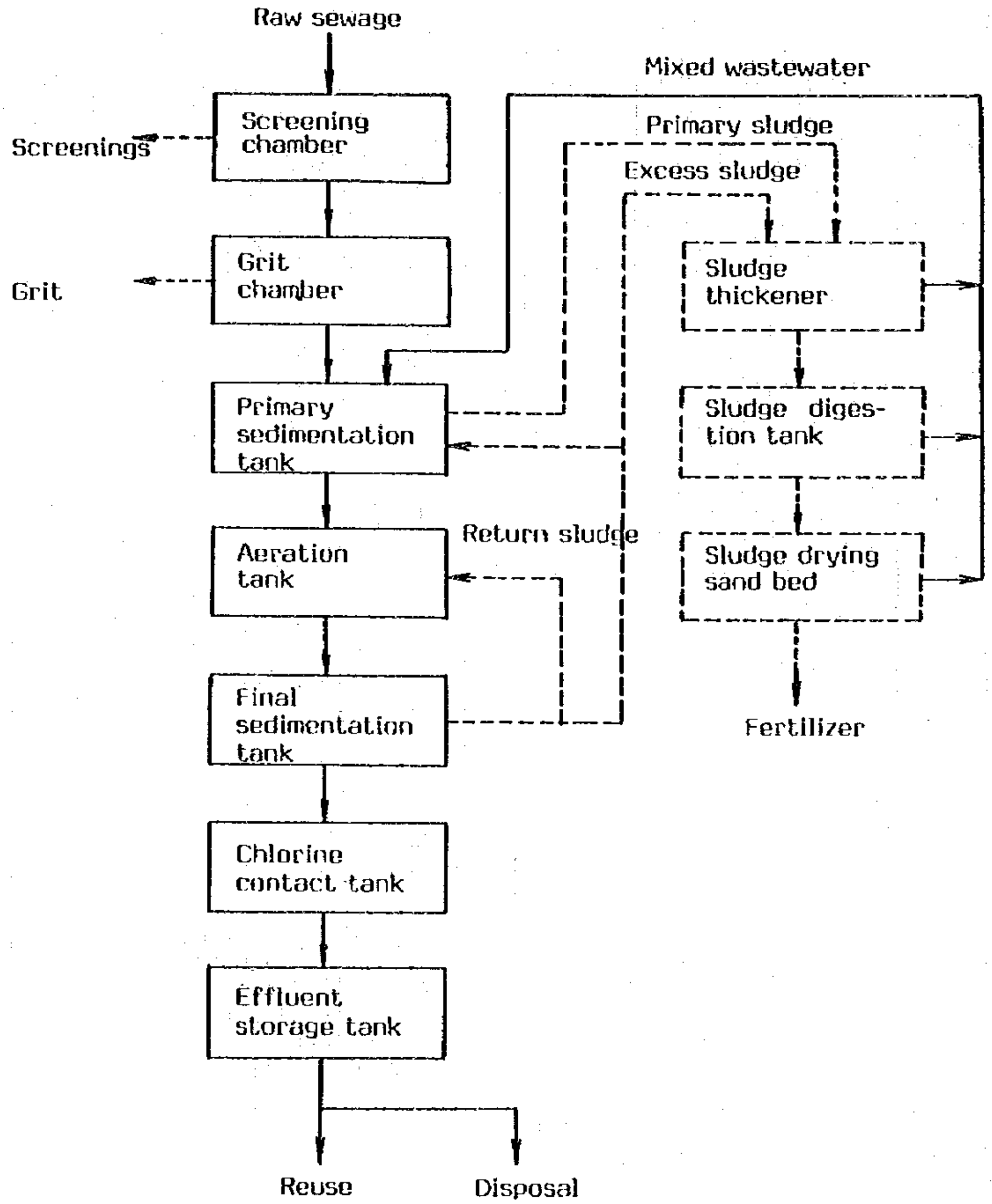


Figure 3.6.7. Flowsheet of Conventional Activated Sludge Process

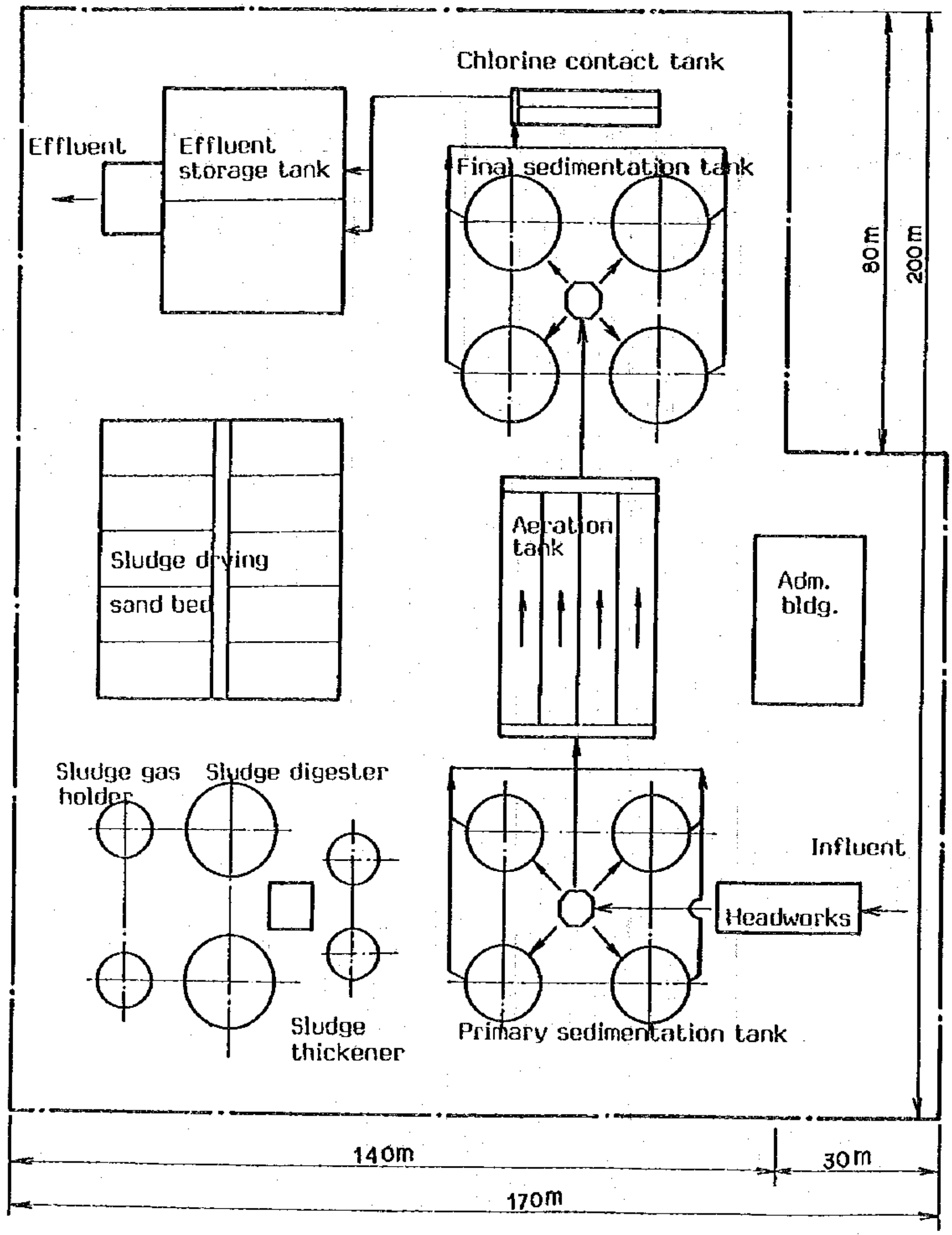


Figure 3.6.8. Layout Plan of Conventional Activated Sludge Process

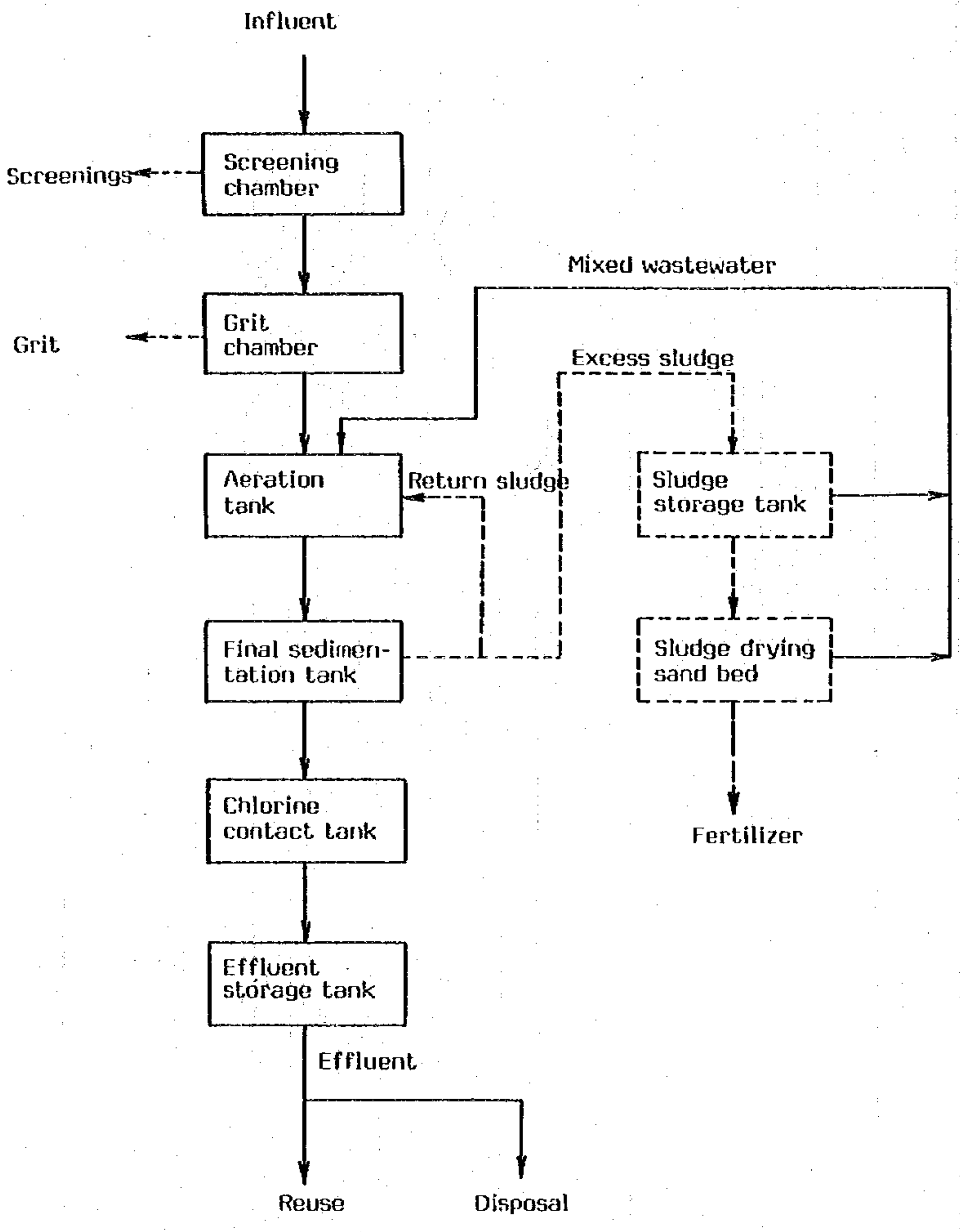


Figure 3.6.9. Flowsheet of Extended Aeration Process

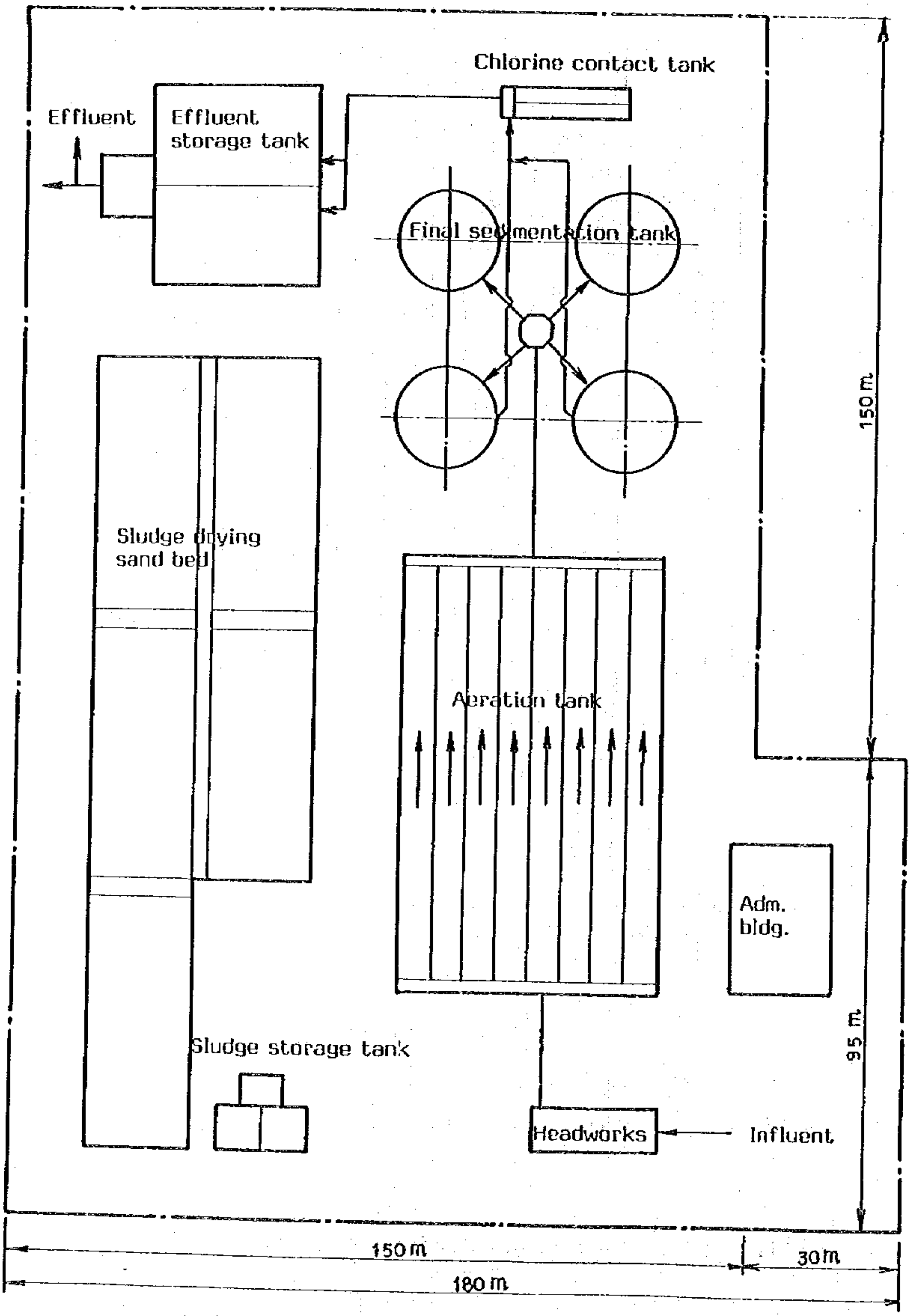


Figure 3.6.10. Layout Plan of Extended Aeration Plant

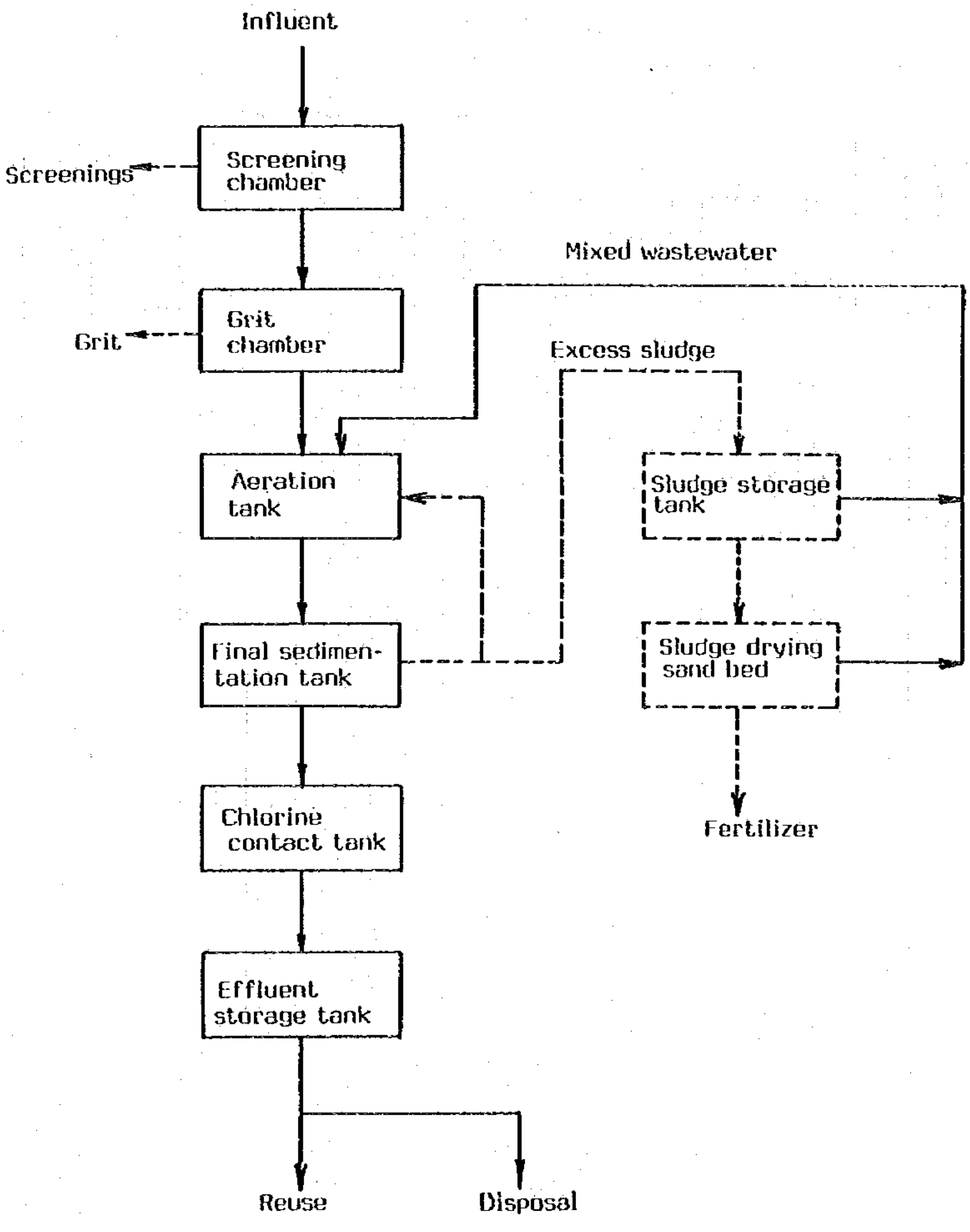


Figure 3.6.11. Flowsheet of Oxidation Ditch Process

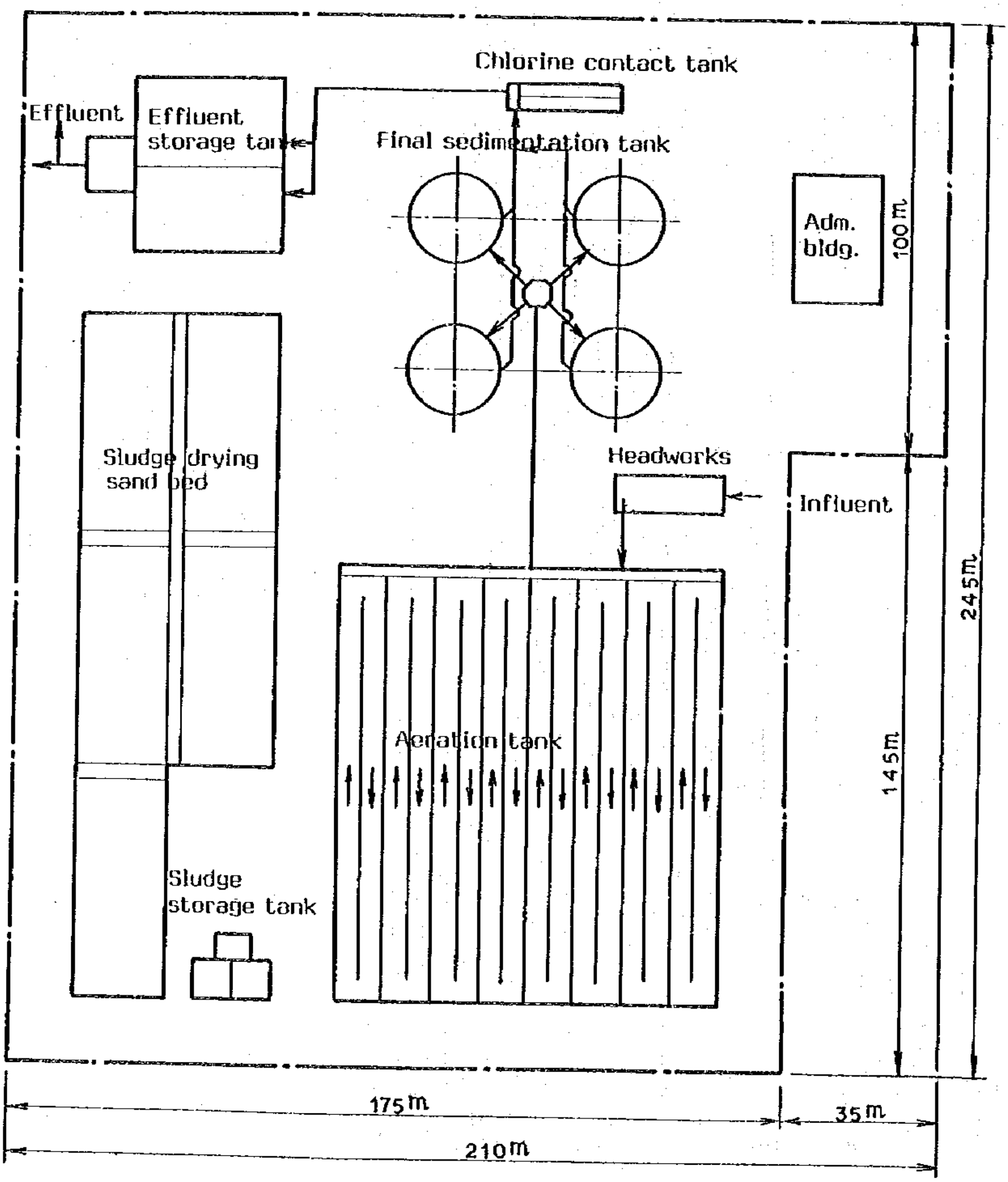


Figure 3.6.12. Layout Plan of Oxidation Ditch Plant

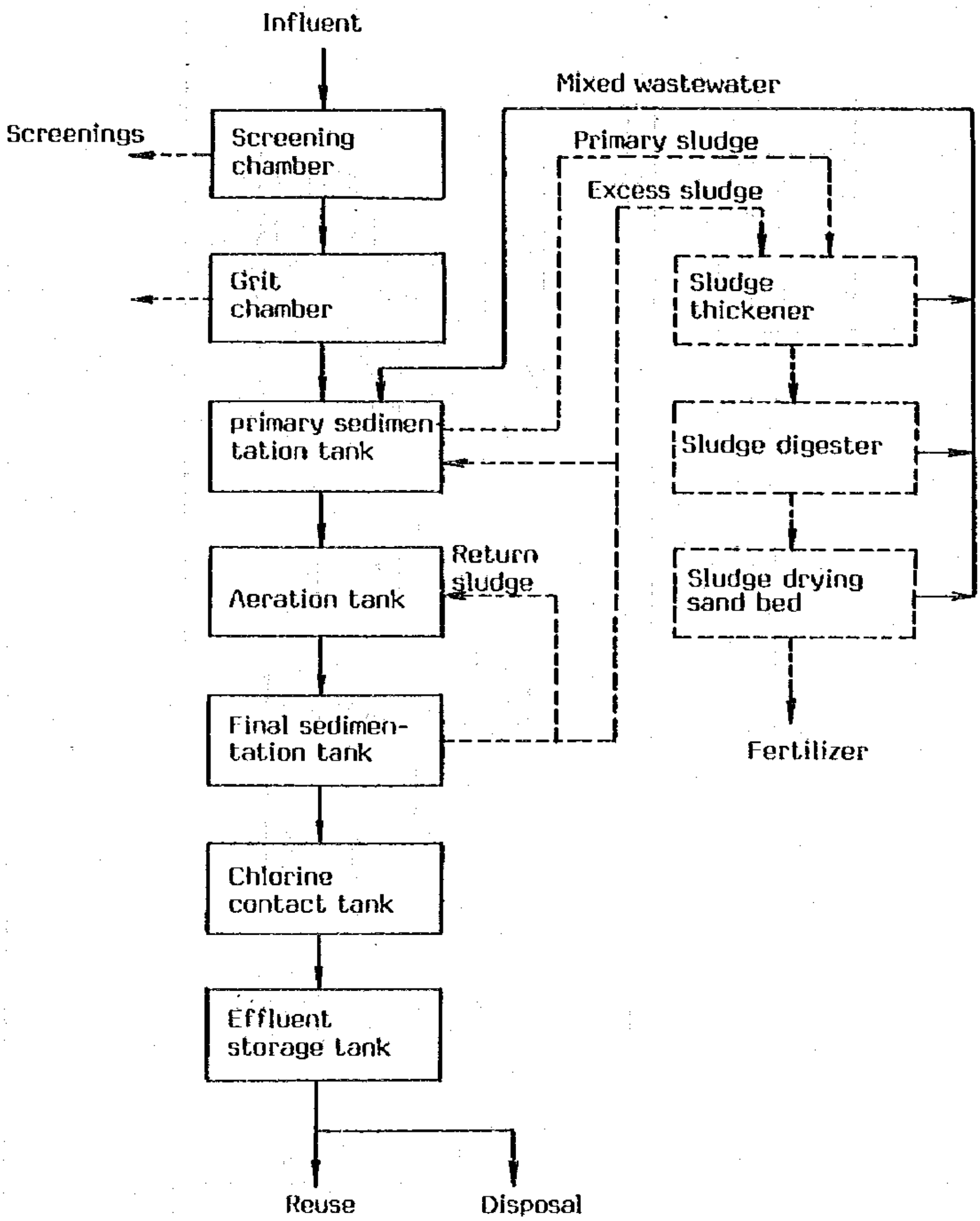


Figure 3.6.13. Flowsheet of Modified Aeration Process

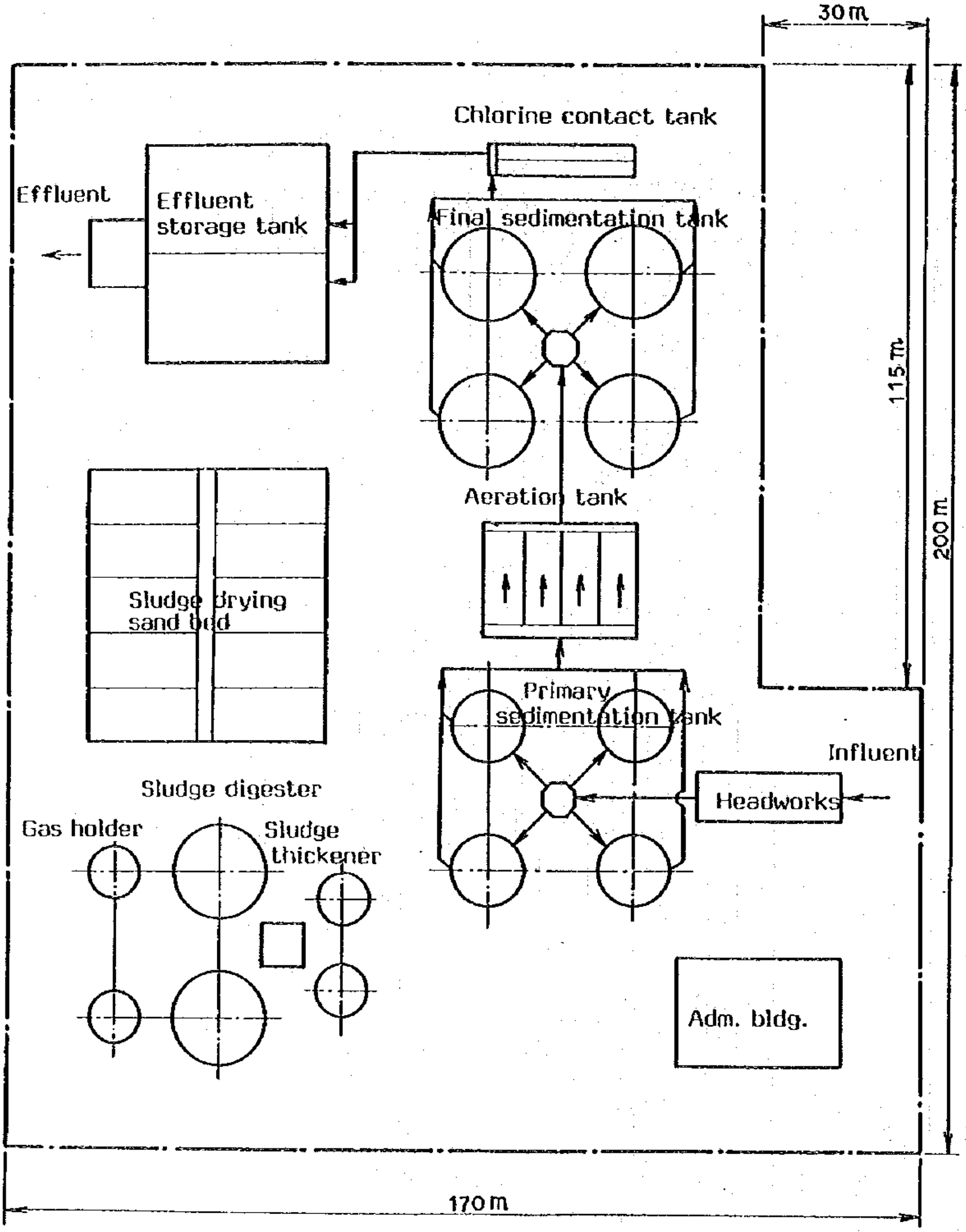


Figure 3.6.14. Layout Plan of Modified Aeration Plant

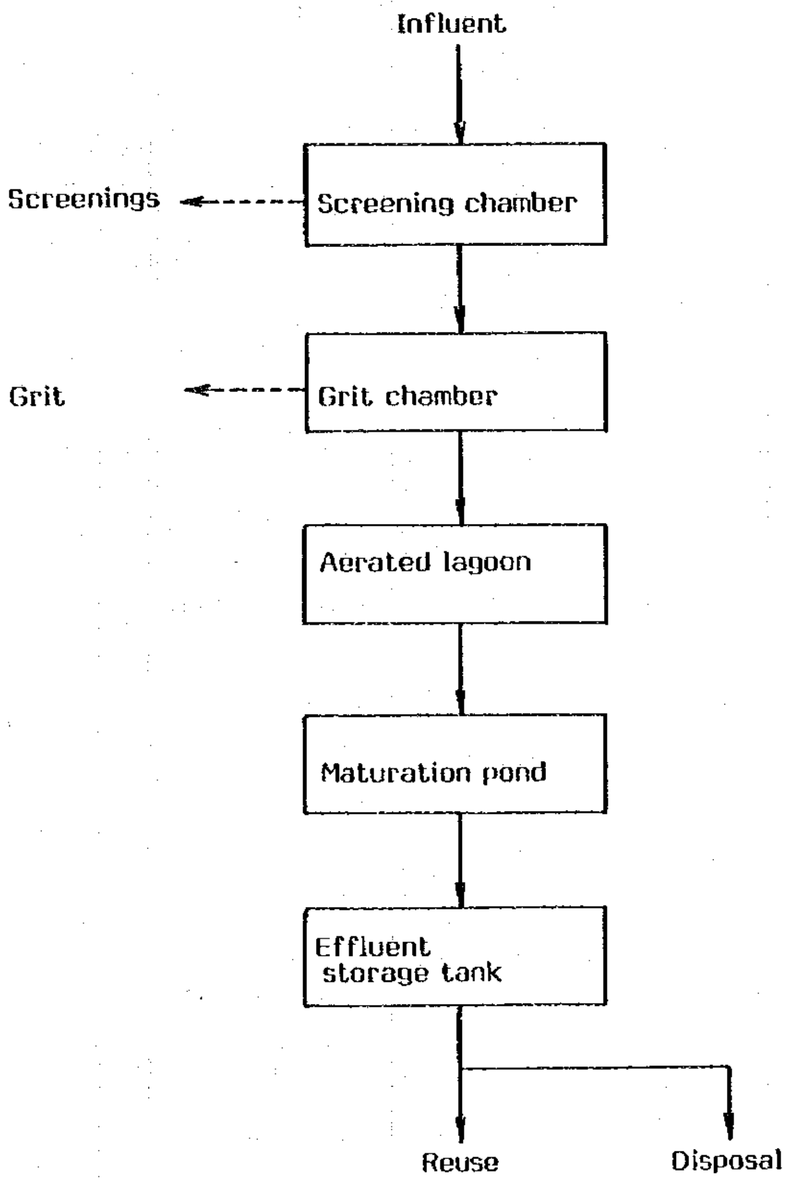


Figure 3.6.15. Flowsheet of Aerated Lagoon Process

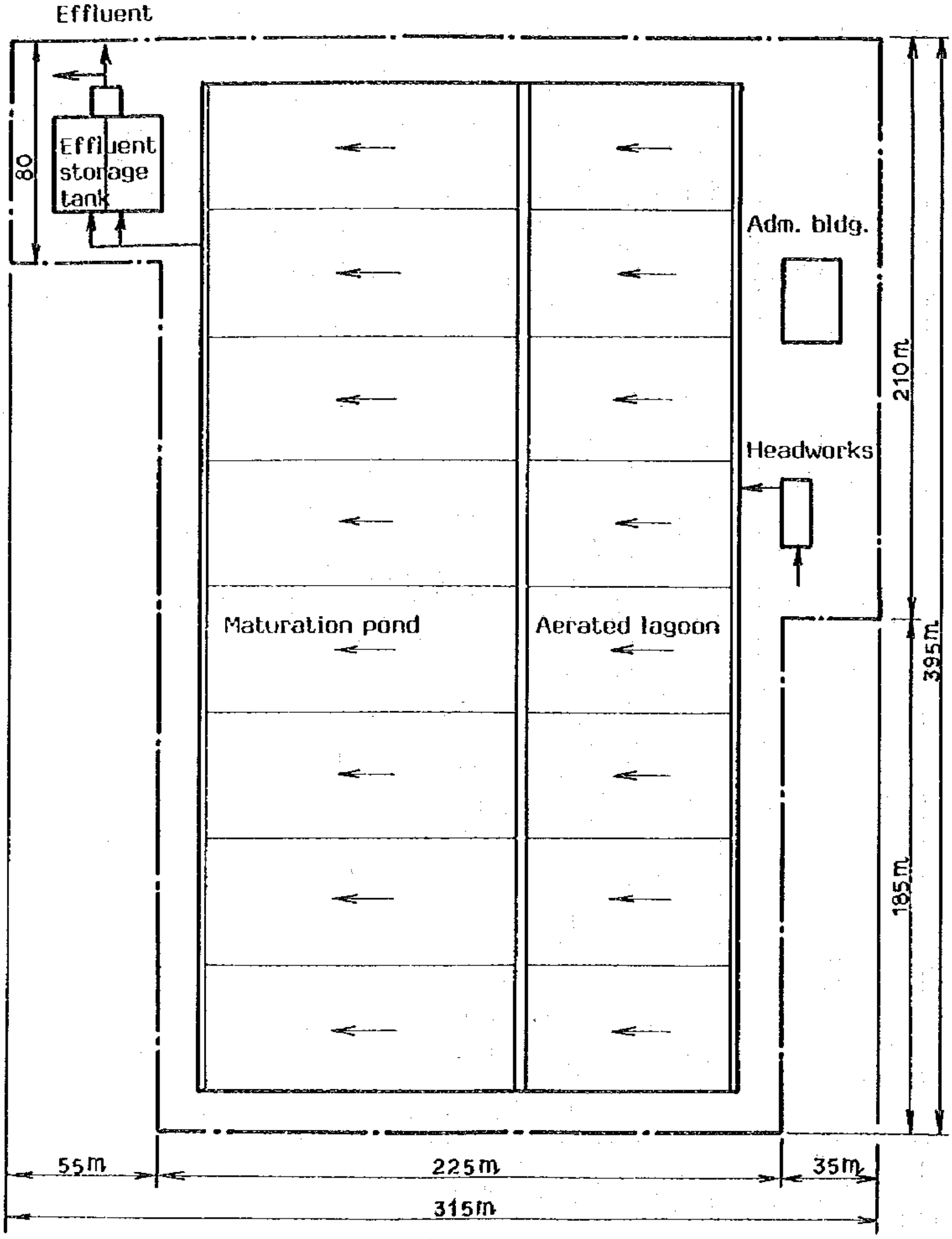


Figure 3.6.16. Layout Plan of Aerated Lagoon Plant

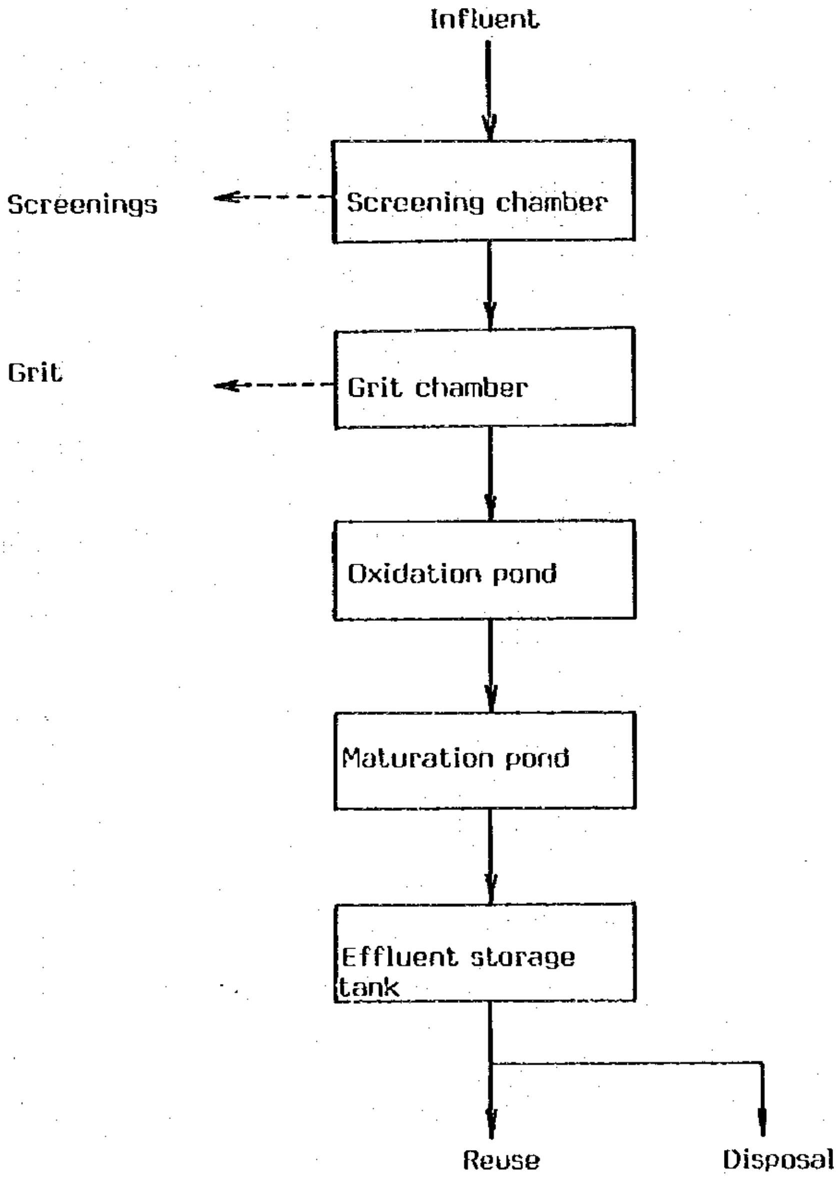


Figure 3.6.17. Flowsheet of Oxidation Pond Process

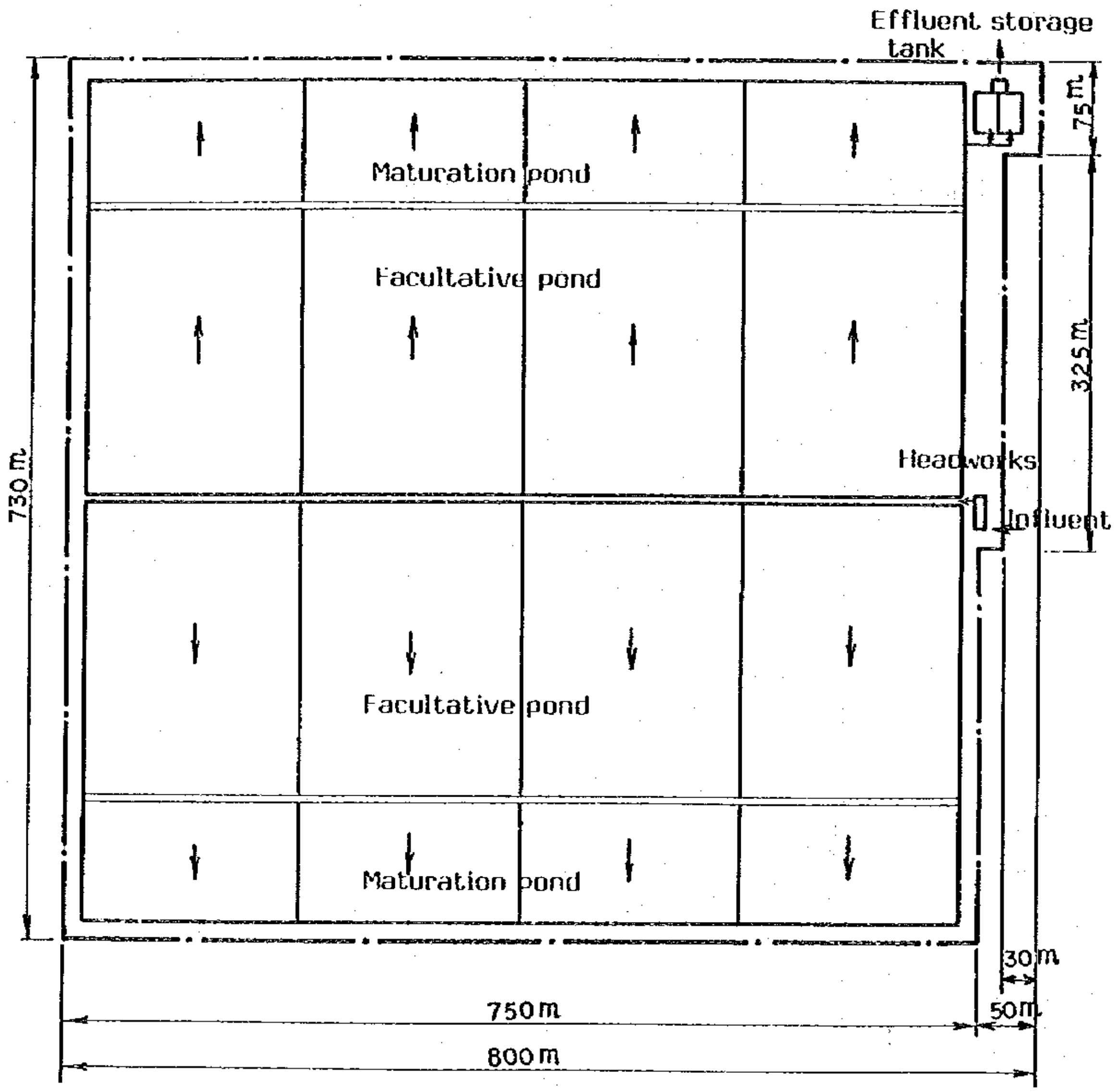


Figure 3.6.18. Layout Plan of Oxidation Pond Plant

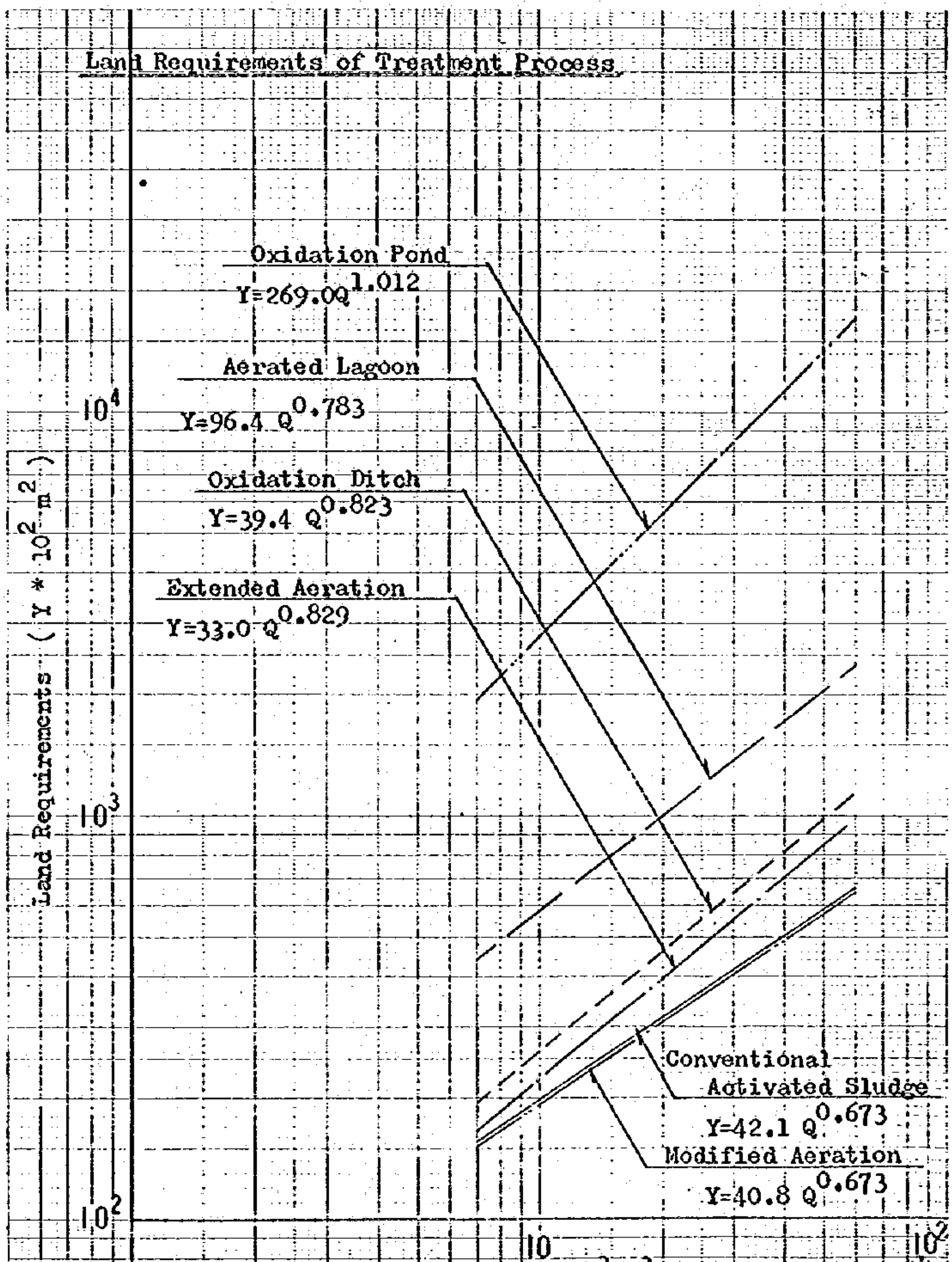


Figure 3.6.19. Land Requirements of Treatment Processes

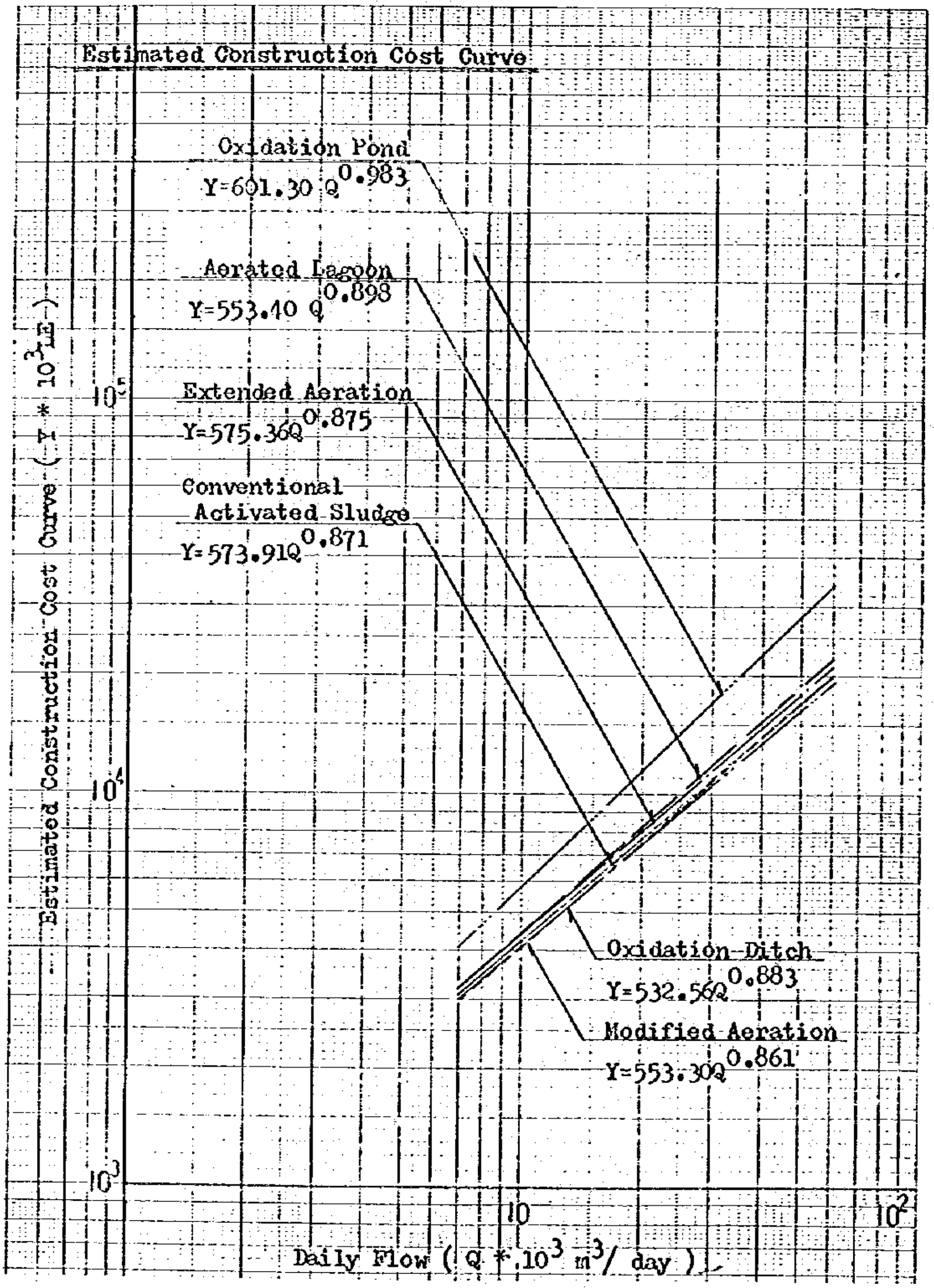


Figure 3.6.20. Capital Costs of Treatment Processes

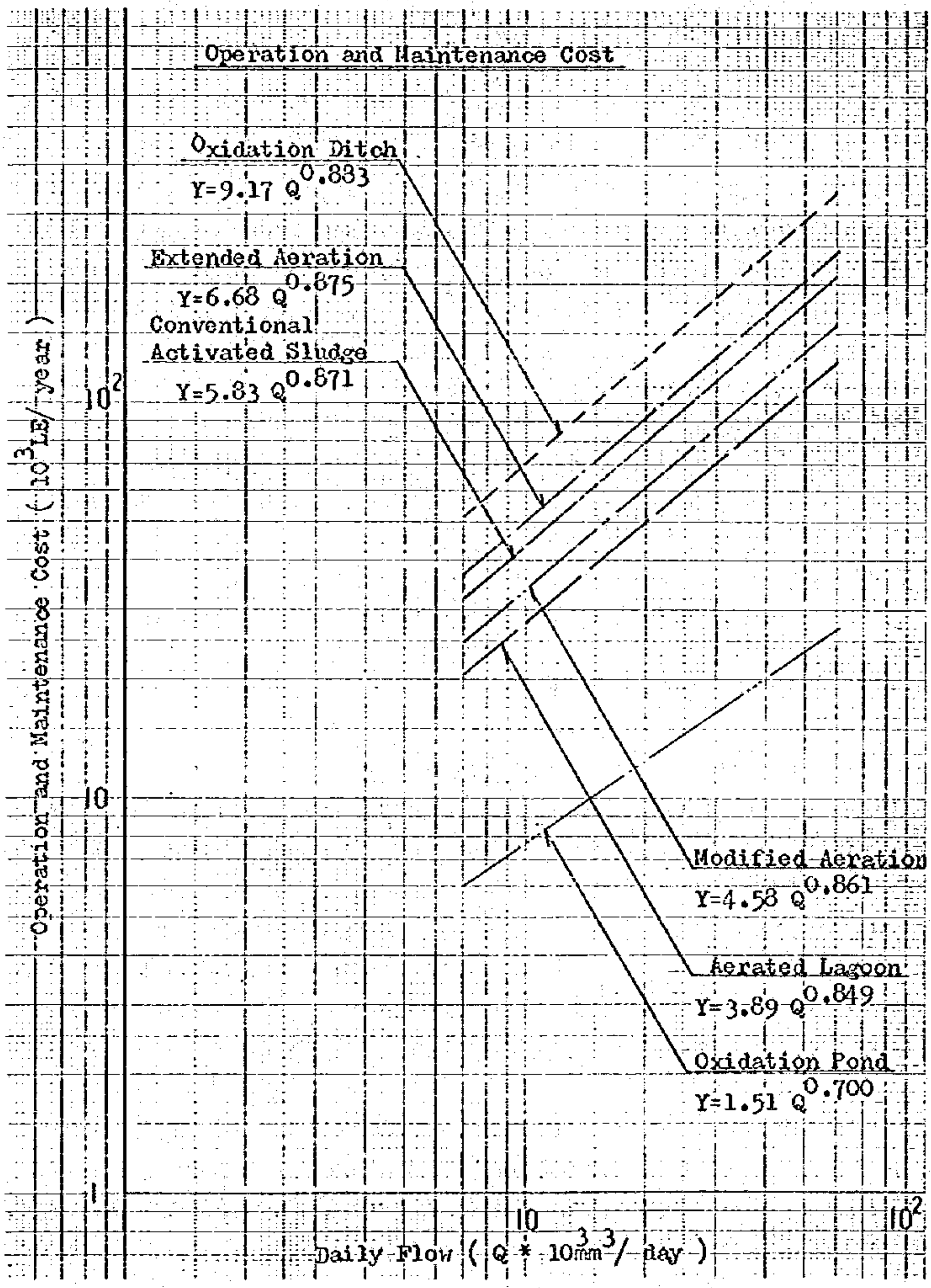


Figure 3.6.21. Operation and Maintenance Costs of Treatment Processes

3.6.6. Sewage Treatment Plant Site Selection

(a) Alternative Construction Sites Considered

In selecting the most viable site for the sewage treatment plant construction, several possible alternative sites have first been selected and studied as to their suitability for the site, evaluating such elements as future development schemes, agricultural development, environmental aspects, and economy of the system. After evaluating each of the alternative plans, less viable alternative sites have been screened out and the following three sites selected for detailed analysis (For locations of sites, see Figure 3.6.22.):

Alternative I - A site in the middle of the city and Jarada

Alternative II - A site close to Jarada

Alternative III - A site east of Salem housing scheme district now under construction

(b) Evaluation of Alternative Sites

Alternative I: This site is situated approximately 5 km east of the city along the highway connecting to Rafah. This site is surrounded by desert and some farm land, and there exists neither house nor public facility. In view of these there will be no significant impacts on the environment by the provision of the sewage treatment plant in this area. The effluent of the plant can be used for irrigation of farm land adjacent to the plant site. The ground-water in this area is expected to have high salinity level, as such it may need to provide tertiary treatment facilities for the process water.

Alternative II: The Government considers Jarada area as the top priority district for agricultural development by using the plant effluent as a new water resource. The soil of the area is arable and promising for crop irrigation by the effluent. This district used to be irrigated by the water transmitted from a source outside the area to produce many kinds of vegetables. In fact, even after the water transmission system was disrupted, certain kinds of rain-fed crops have been grown by Bedouins. On account of these, an efficient use of the plant effluent as well as dried sludge can be expected.

On the other hand, this alternative site may have certain disadvantages. The raw sewage pressure pipe line will be about 10 km long from the sewerage system in the City, thus requiring more energy costs for transporting the sewage than other alternative sites. Moreover, it may be difficult to use the effluent economically for other purpose such as industrial purpose, because an additional equipment will be required to further convey the effluent for such use.

Alternative III: This site is relatively flat land situated close to the urban area and surrounded by cultivable land. Therefore, the sewage effluent will be most effectively used for crop irrigation at the least cost among other alternative plans. However, this area is planned by the Government to develop as the urban area, particularly zones of 2 km wide along the east side of the road connecting to the airport. Also, part of the surrounding area of the site is reserved as the military zone, and it is unlikely that the land is alloted for the treatment plant site. For these reasons, this alternative site appears to be not practical and thereby this plan was screened out from further economic analysis.

(c) Wastewater Transportation

One of the governing factors in selecting the sewage treatment plant construction site is the economy of wastewater transportation. For an economic analysis on the wastewater transportation system alternatives, the costs for the treatment plants were not considered because both initial and operation and maintenance costs of Alternatives I and II are same. Therefore, the only difference between the two alternatives are those for wastewater transmission and intermediate pumping stations to be provided in between the sewer terminal at the point 'O' and the treatment plant site.

In either case, it needs to transport the wastewater from the sewer terminal to the plant site. As may be seen from the profile of the pipe line route in Figure 3.6.23. it is not practicable to send directly to the plant sites due to the possible water hammering actions expected and topographic features. Under such conditions, the possible plans available will be:

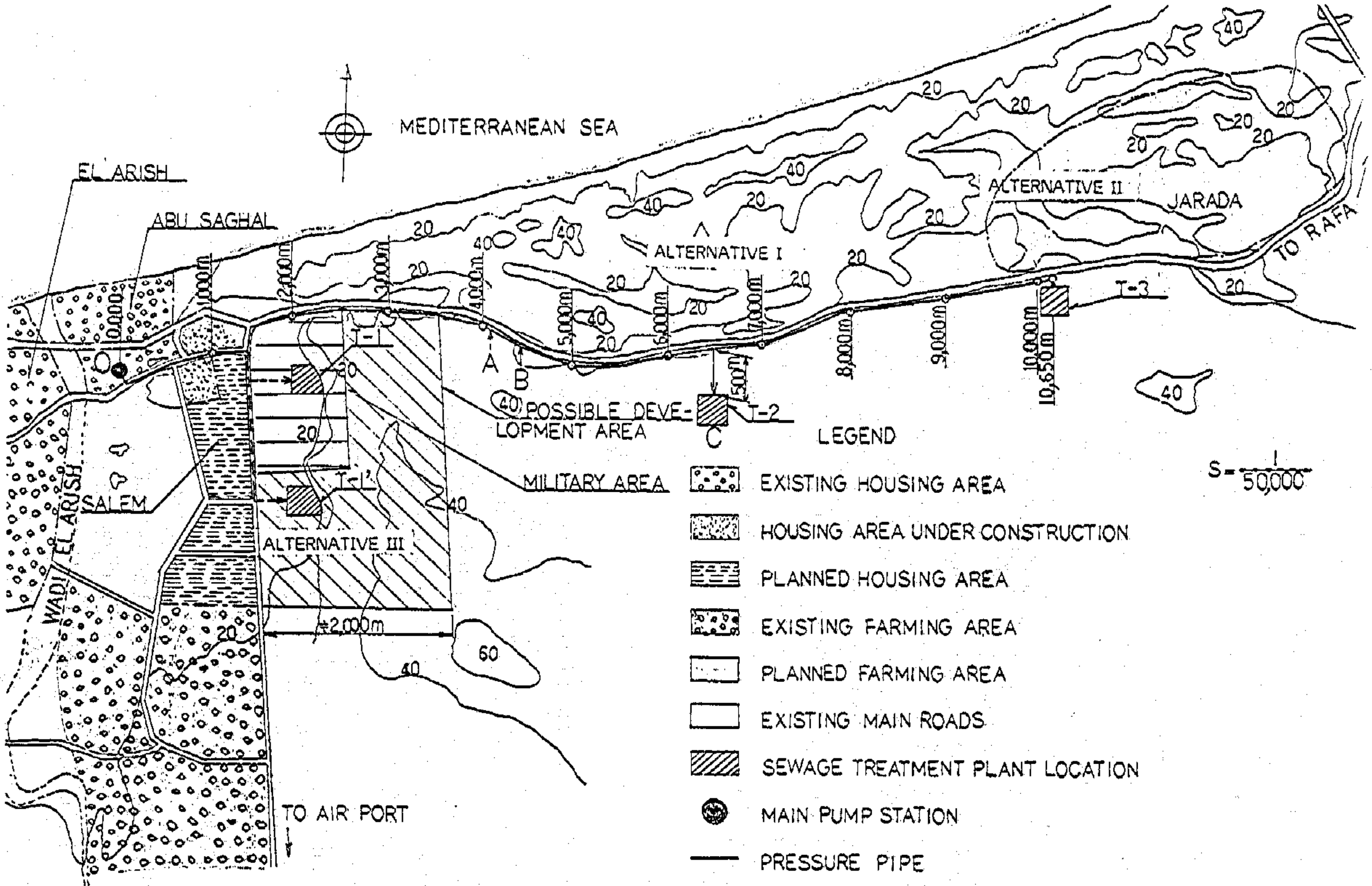


Figure 3.6.22. Locations of Alternative Sewage Treatment Plant Sites

Alternative I: As can be seen from Figure 3.6.23, the wastewater may be coveyed from point'O' to point'A' by pressure, and then by gravity to point'C.' The wastewater is treated in the plant and the effluent will be further transported to the storage tank of Jarada crop irrigation system at point'P.'

Alternative II: The procedure for sending the wastewater from point'O' to point'A' is same as that of Alternative II. From this point as far as point'B,' the wastewater will flow by gravity, but from'B' to'P' by pressure. In this case, the pressure pipeline have to be laid at an elevation lower than 22.0 m above mean sea level so as to avoid impacts of the possible water hammering actions. By doing so, the wastewater can be transmitted from point'B' to point'P' by pressure. The reason why a pumping station is provided at the point'B' is to avoid as much as possible the upland downs under the conditions of gravity flow in the pipeline. One inherent disadvantage for providing a pumping station at point'B' will be that the difference of heads between'A' and'B' is such a significance that the wastewater will possibly overflow if pumping station 'B' stops and the wastewater continues to flow from pumping station A, although this may be avoided by the provision of appropriate gates and valves at the pumping stations.

(d) Availability of Water Sources

Another important factor to be considered in selecting the plant site is the availability of water sources, particularly soft water for process and drinking purposes. There will be two alternative cases for water supply to the plants, one from El-Arish city water and the other from Sheikh Zuwayid. At the both alternative sites, it is expected that the groundwater contains as high as 4,000 mg/l salt and not potable.

In the case of transmitting water from El-Arish, Alternative II will be more costly, whereas sending water from Sheikh Zuwayid, this will be reversed, because of the difference of water pipeline length. At present, however, no comprehensive water resources development programme has been een established yet, and it is not possible to estimate exact costs required for the water supply system at this stage. For this reason, the cost for the water system was excluded from the costs for alternative study. Water for other purposes such as cooling and cleansing of plant equipment, may be produced by the tertiary treatment process that further polishes the secondary effluent.

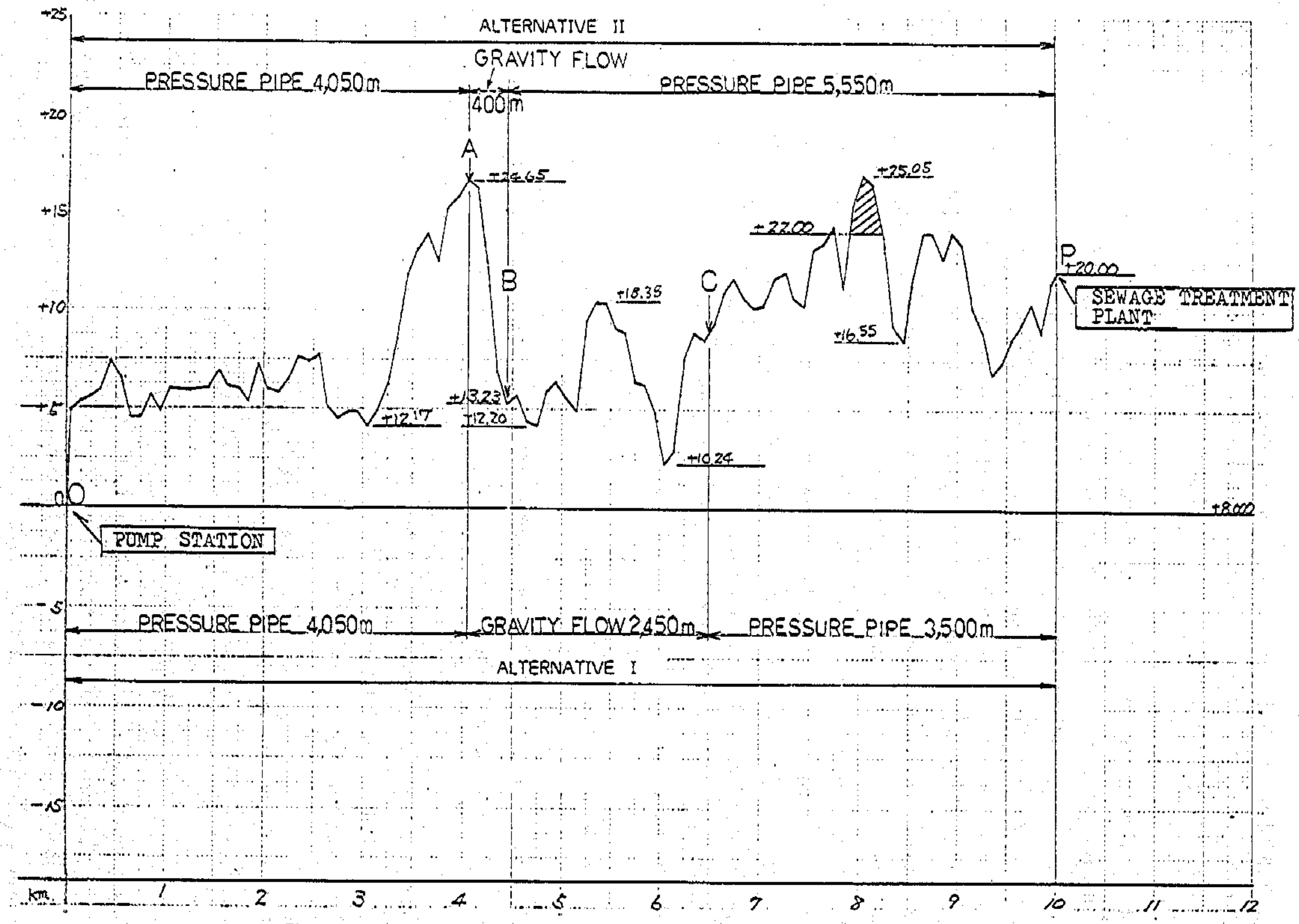


Figure 3.6.23. Longitudinal Profile of Pressure Sewer Route

(e) Cost Effectiveness Analysis of Alternatives

As discussed in the previous sections, an economic analysis has first been made on the case with three pumping stations at points 'O', 'B' and 'C'. The initial costs for Alternatives I and II are summarised as follows:

Item	Alt. I		Alt. II	
P.S. of 'O'	1,092.2	(1000 L.E.)	1,092.2	(1000 L.E.)
P.S. of 'B'	-		935.5	"
P.S. of 'C'	870.6	"	—	
Pressure pipe	2,479.4	"	3,064.6	"
Gravity pipe	908.6	"	123.2	"
Total	5,350.8	"	5,215.5	"

As seen from the above, the costs for pumping station 'O' is the same irrelevant to the location of the stations. In the pumping stations 'B' and 'C', the costs for civil works and buildings are almost same, but due to the higher costs in mechanical and electrical equipment in station 'B', the overall cost for the station 'B' is more costly. As to the pressure pipe, 500 more metres pipeline is required for the station 'C'. thus the costs for station 'C' is higher.

The operation and maintenance costs for the pumping stations are as follows:

Item	Alt. I	Alt. II
P.S. 'O'	109.6 (x 1,000 L.E./yr)	109.6 (x 1,000 L.E./yr)
P.S. 'B'	-	103.3
P.S. 'C'	71.6	-
Total	181.2 "	212.9 "

The above alternatives have been studied for cost-effectiveness analysis under the following conditions:

- Prices, at mid-1984 level.
- Discout rates, 8, 10 and 15 per cent per annum.
- Service life; land, permanent, structure 50 years, process equipment, 20 years.

The comparison of the annual costs of the two alternative plans at the three different discount rates become the following:

Item	Alternative I	Alternative II
Interest (3.5 %)	187.1 (1,000 L.E./yr)	182.4 (1,000 L.E./yr)
Depreciation	151.2	151.1
O & M	181.2	212.9
Total annual cost	519.5	546.4
Interest (8 %)	428.0	417.2
Depreciation	151.2	151.1
O & M	181.2	212.9
Total annual cost	760.4	781.2
Interest (15 %)	802.6	782.2
Depreciation	151.2	151.1
O & M	181.2	212.9
Total annual cost	1,135.0 (1,000 L.E./yr)	1,146.2 (1,000 L.E./yr)

Having analysed the above calculation results, it has been revealed that the provision of a pumping station at the point 'B' would add certain costs to Alternative II, thereby making the plan less economical. For this reason, it appeard to be more desirable to flow the wastewater by gravity from the point 'A' as far as the condition allows, say up to the point 'C'.

In view of the conditions as discussed above, another alternative study has been further conducted, i.e., to provide a pumping station at the point 'C' but omit all other pumping stations for Alternative II. In this case, the costs for both capital and operation and maintenance are almost same, the only difference being that required for pipe laying to and from the treatment plant at the point 'C' which is about 500 m south of the highway. The difference of the costs between the alternatives is approximately 308,000 L.E.

The economic analysis for the alternative plans carried out by the same procedures as above are summarized in the following:

Table 3.6.11. Capital and O & M Costs of Alternative Plans (in 1,000 L.E.)

Alternative Plan	Capital Cost	O & M Cost (per year)
I	5,350.8	181.2
II	5,215.5	212.9

Annual costs for alternatives at the three different discount rates are as summarized in the following table:

Table 3.6.12. Annual Costs for Alternative Plans (in 1,000 L.E./yr)

Item	Alternative I	Alternative II
Interest (3.5 %)	187.1	176.4
Depreciation	151.2	145.1
O & M	181.2	181.2
Total Annual Cost	519.5	502.7
Interest (8 %)	428.0	403.3
Depreciation	151.2	145.1
O & M	181.2	181.2
Total Annual Cost	760.4	729.6
Interest (15 %)	802.6	756.4
Depreciation	151.2	145.1
O & M	181.2	181.2
Total Annual Cost	1,135.0	1,082.7

As revealed in the foregoing discussions, it is economically and technically not feasible to transmit the wastewater directly to Jarada area, and intermediate pumping station(s) is indispensable for either treatment plant. The only difference is that Alternative I plant will send the effluent to Jarada, while that for Alternative II is the raw sewage.

Discussions as described hereinabove have led to the conclusions that the treatment plant at Alternative II location at Jarada area with an intermediate sewage pumping station at the point 'C' be provided.

3.6.7. Stormwater Drainage System

(a) Combined Versus Separate System

In general, combined system is less costly than separate system because the combined system needs only a single barrel to convey the mixture of rainstorm runoffs and wastewaters, whereas the separate system requires two barrels separately for sanitary sewage and rainstorm water. In arid or semi-arid region like El-Arish, where rainfalls are low and soil permeability is high, a large capacity storm sewers are not required and thus in many cases the separate system is more advantageous than the combined system. For the cost comparison between the two systems, a cost estimate has been made for each system under the conditions of the Study Area. The estimate indicates that the costs for the separate system with stormwater soakaway facilities and the combined system are approximately L.E. 19 million and L.E. 30 million, respectively, making the separate system more favourable under the conditions.

The stormwater transh or other soakaway system in the separate system can be easily added when required, thereby an excessive investment can be avoided. A possible merit by the combined system may be that the mixed water collected through the combined sewers can be used as a new water source for crop irrigation or other purposes. However, if the water is to be used efficiently for such purposes, a large capacity rainwater storage facilities will be inevitably required because of the low amount of rainfalls in the region. Besides, the mixture of rainwater and sanitary sewage will definitely require a treatment to produce the mixture to the extent acceptable for the safe use of the water if the water is to be utilized directly for crop irrigation. The treatment requires additional costs. For these reasons, the combined system is considered not appropriate to the drainage system in El-Arish.

(b) Planning Basis for Drainage System

Because the Study Area is predominantly sandy soil with high permeability having the low precipitation, the stormwater runoffs can be easily absorbed into ground. The conditions do not necessitate to collect and discharge the surface runoffs through storm sewers, making the stormwater transh or infiltration basin systems more economical. Furthermore, by these on-site system enables an effective recharge of the rainwater to the ground.

The infiltration through the natural soil surface requires no special facilities and the cost is low, although a wide space may be needed to store and infiltrate the water to the ground. The transh system on the other hand requires certain costs for the construction of soakaway facilities, and they will not function effectively if they are provided at locations where the groundwater elevation is high.

Since the groundwater elevation is in general high in Masaid district, the transh system will not be suitable under such conditions. Moreover, wide land space is still available in the district so that the infiltration basin system is the most feasible solution for the district. In the El-Arish district, however, sufficient land space is not available to provide the infiltration basins, and as such the transh system is considered most desirable for the district.

(c) El-Arish Drainage District

As described in the preceding paragraphs, the stormwater drain system for the district is proposed to be of the transh system, using the circular type transh for the following reasons:

- The system has been widely used for leaching the wastewater and residents have experience for installation and maintenance.
- Standards for design and construction have already been established.
- Construction costs for the transh is in general reasonable and affordable by the citizens.
- High permeability of soil is expected over long period.

In order to ascertain the practicability of the stormwater transh, a field survey was carried out at a selected existing stormwater transh near the NSG headquarter on 2nd October 1984. The survey revealed that a significant amount of infiltration could be expected though the transh was almost filled by sand.

Since the stormwater transh will be in general installed on the public roads or vacant lots, the size of the transh should be of such small as to avoid any possible interruption of traffic or pedestrians. In addition, the stormwater runoffs within private plots should be absorbed as much as possible within their boundaries to prevent the overflow of such surface water to public places. In planning the transh system, the following cares should be taken:

- Stormwater in private plots should be soaked to the ground within their boundaries to the maximum extent so as to prevent excessive overland flow to public land.

- In the case that no such vacant bare space is not available in private plots, a transh may be provided within the plots or abandoned wastewater transh may be converted to the stormwater transh.

Although the amount of stormwater overflows from private houses to the public transh system can be significantly reduced by the steps above, it is still unlikely that all the stormwater in private houses can be infiltrated within their plots. Therefore, the capacity of a public transh should be determined providing an allowance for the amount of water coming from roofs of the nearby houses.

In actual planning of the public transh system, locations of transh may change depending upon the conditions of roads or surrounding areas. In principle, the transh should be provided not to interrupt any other public facilities and underground structures such as water pipelines, telephone cables, etc. Although the exact locations of the transh should be determined in the detailed design stage, the transh should in principle be installed at every road crossing, or one transh at every 0.25 ha drainage area. Construction of the transh may be proceeded according to the priority of the requirements of such system.

As already mentioned, the existing drainage system involves some problems in its performance. In addition, costs for drainage pipes are generally higher than those of the transh system. In view of these, it is considered appropriate that no further extension of stormwater drain pipes be planned. When the existing drainage facilities become obsoleted, these should be gradually replaced by the new stormwater transh system.

(d) Masaid Drainage District

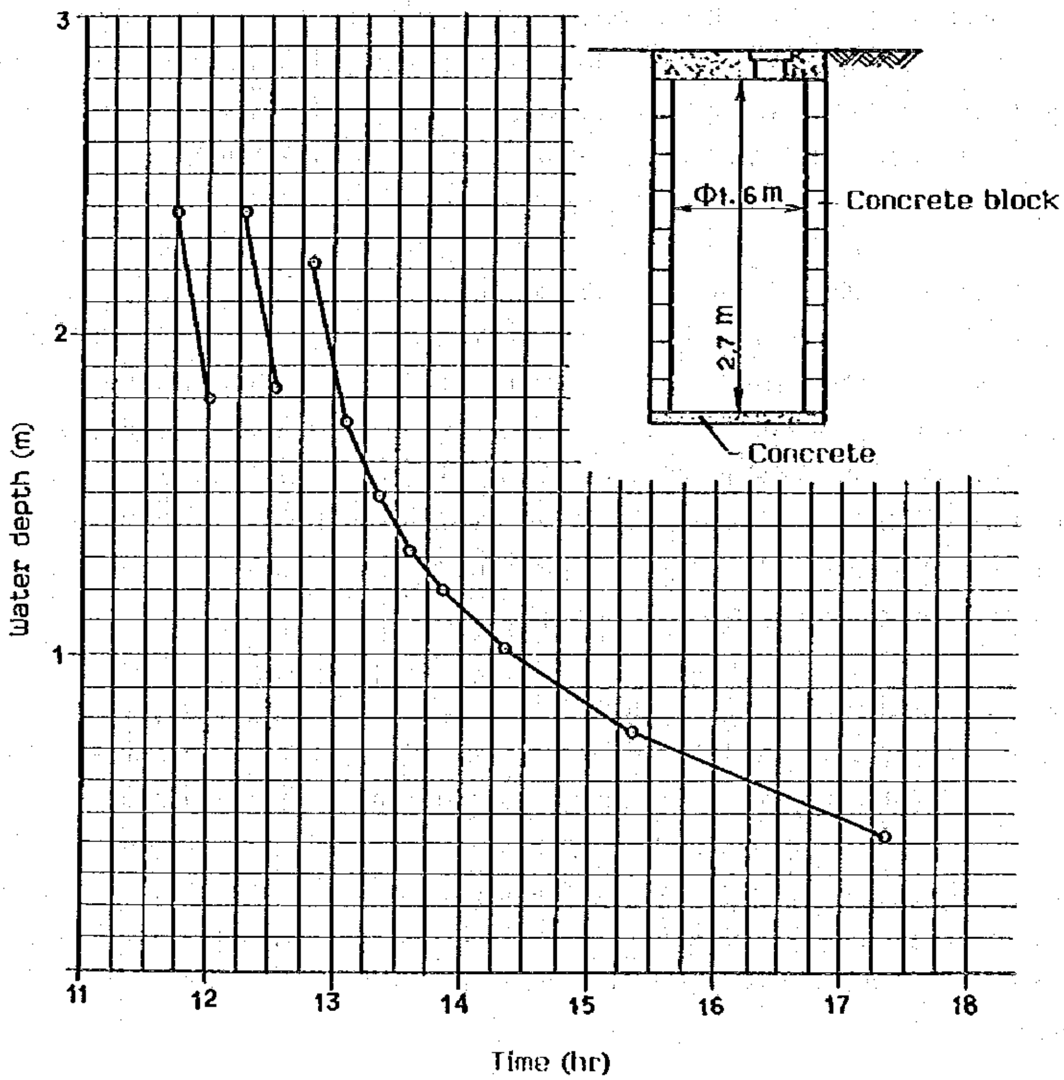
The drainage system in this district should be planned by the infiltration system, including two different types to soak the surface water, one is to absorb the water through porous paved road surface and the other by filtration basin to be provided in vacant public space. As the groundwater elevation is high in this part of the City, the infiltration basin system is more suitable than the paved road system.

The construction of the infiltration basin system is the ponds excavated about 30 cm deep to which the surface runoffs inflows from the nearby area and then infiltrates into the ground. The stormwater within private plots should be disposed of into soil within their house plots as much as practicable, so that the public infiltration system will receive stormwater only from public lands. The infiltration basins should be provided according to the priority of the drainage requirements to be determined based on the flooding conditions.

(e) Stormwater Transh System

To obtain basic data on the function of stromwater transh, a field survey was undertaken on 2nd October 1984 at a selected existing stormwater transh located near the headquarter of North Sinai Governorate. For the test of the transh infiltration capacity, well water was filled in the transh and then water elevation drops measured as shown in Figure 3.6.24.

The results of the survey indicate that the fall of water elevations in the transh of 1.6 metres diameter and 2 metres deep was about 0.4 metres in 15 minutes or 0.8 m3 per 15 minutes. This value is equivalent to 3.2 m3/hr and,



Water depth	2.38	1.80	2.38	1.83	2.12	1.72	1.49	1.32	1.20	1.01	0.75	0.42
Time	11:45	12:00	12:17	12:32	12:51	13:06	13:21	13:36	13:51	14:21	15:21	17:21

Figure 3.6.24. Infiltration Capacity Test of Transh

for the drainage area of 400 and 800 m2, the infiltration capacity can be expressed as 8 to 4 mm/hr.

The infiltration capacity of soil is expected to decrease to 1/3 to 1/5 after several years use of transh. For planning purpose, a decrease to 1/10 may be used in consideration of a safety factor, thus, the infiltration capacity of soil per unit area will be about 1 mm/hr, which is a negligible value. For this reason the capacity of a transh should be determined based on the amount of receiving rainwater runoff.

A time required for absorbing the water depends on soil characters, transh structure, etc. Since most of the Study Area is sandy soil,the soil characteristics are more or less same throughout the area,and since the tested transh is a standard type most widely used in the area, the results of the survey can be applied to the planning of transh in other part of the area. According to the survey results, the filled water was completely absorbed in 5 hours. Even though the decrease of infiltration capacity is expected after using for several years, the water may be absorbed within 2 days at the maximum.

(f) Structure of Transh

As described previously, the use of concrete blocks for transh has some merits, however, in determining the transh structure the following considerations should be given:

- The transh diameter may be 2 metres or smaller because of the available size of ready-made concrete blocks. Transh depth will be governed by the groundwater elevation.
- For the convenience of the maintenance of transh, particularly by vacuum trucks, the depth of transh shall be not more than 5 metres.
- At least 30 cm depth shall be provided for grit catch at the bottom of transh.

The recommended standard structure for stormwater transh is illustrated in Figure 3.6.25. As shown in the figure, the bottom of transh should be such

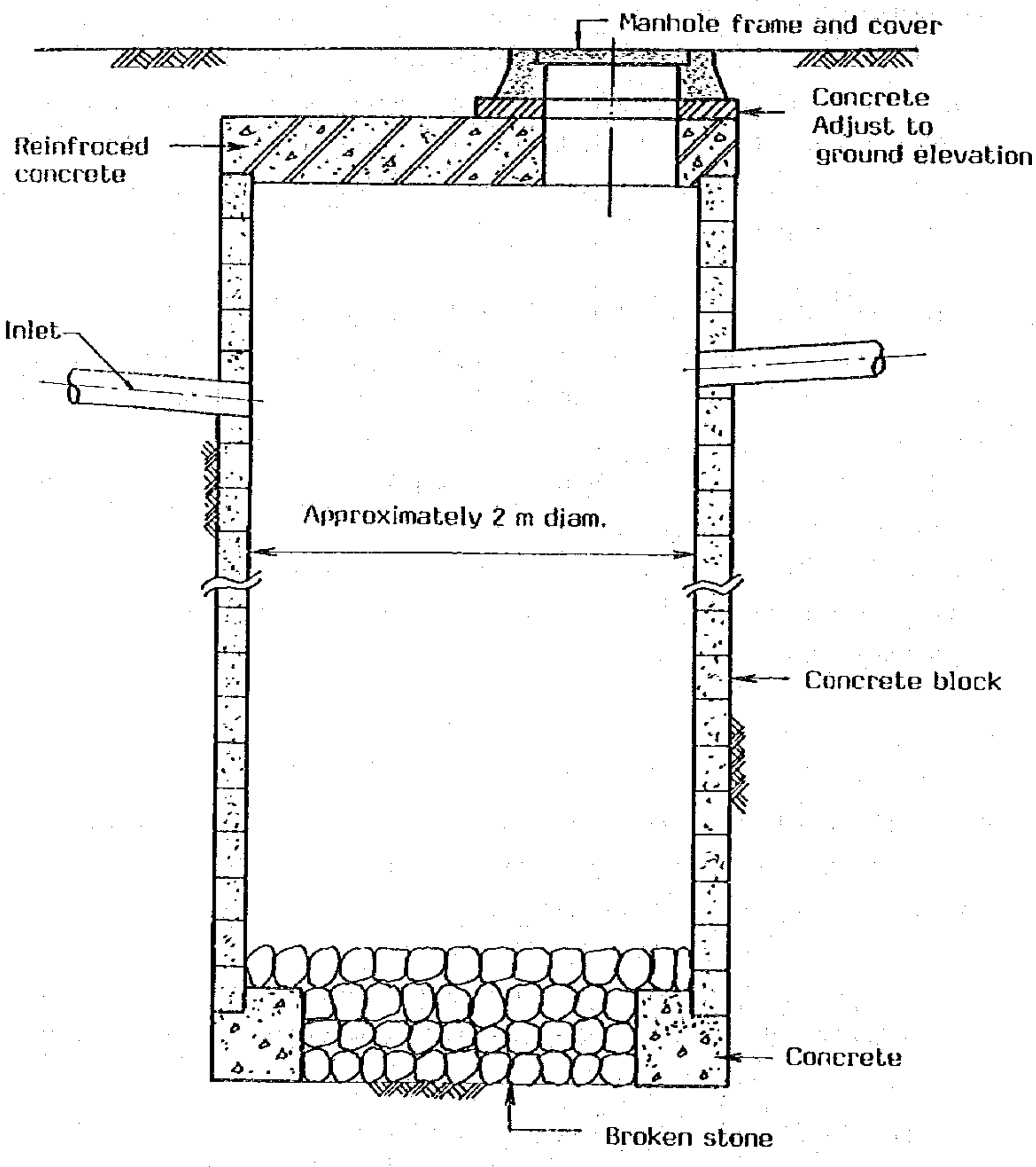


Figure 3.6.25. Typical Drainage Transh

that cleansings by vacuum truck can be easily made and stagnation of water can be prevented.

(g) Capacity of Stormwater Transh

The capacity of transh shall be determined on the basis of the 5-year frequency rainfall intensity. Assuming that the transh diameter of 2 metres, the required transh depth can be calculated using the 5-year rainfall intensity-duration equation.

A transh capacity is equivalent to the total amount of rainfall over the impermeable land surface area. The total amount of rainfall may increase at the longer duration, however, the difference in the amount between rainfalls will be smaller if the rainfall durations exceed a certain duration. For example, the total amount of precipitations in 8-hour and 16-hour durations over a drainage area of 400 ha are;

$$1060/8\times 60+22 \times 1/1000 \times 8 \times 400 = 6.8\ m3$$
$$1060/16\times 60+22 \times 1/1000 \times 16 \times 400 = 6.9\ m3$$

In the case of 2 metres transh diameter, the depth can be determined adding 30 cm allowance for grit catch as follows:

$$H = 0.0174\ A/(2^2/4 \times 3.142) + 0.3$$
$$= 0.0055\ A + 0.3$$

where

H = depth of transh (m)

A = impermeable surface area (m2)

0.0174 = 1060/1440+22 x 1/1000 x 24

(h) Stormwater Infiltration Basin

The advantages of the adoption of the stormwater infiltration basin to the Masaid drainage district may be summarized as follows:

- The infiltration basin will be dry throughout the year and clogging of soil by the growth of micro-organisms is unlikely to occur.

$$A = 0.0174\ a/0.4$$
$$= 0.0435\ a$$

where

A = required surface area of basin, m2
a = inpermeable surface area in the tributary, m2

The structure of the infiltration basin shall be designed taking the following into account:

- The depth of the basin shall be 40 cm or deeper. The depth should, be, however, as shallow as practicable because of the high groundwater elevation in the district.

- Appropriate marks should be provided to indicate the location of the basin, because of the possible soil erosion of the bank soil or moving sand intrusion.

Typical structure of an infiltration basin and layout of stormwater drainage and sanitary sewerage facilities are illustrated in Figures 3.6.26. and 3.6.27. respectively.

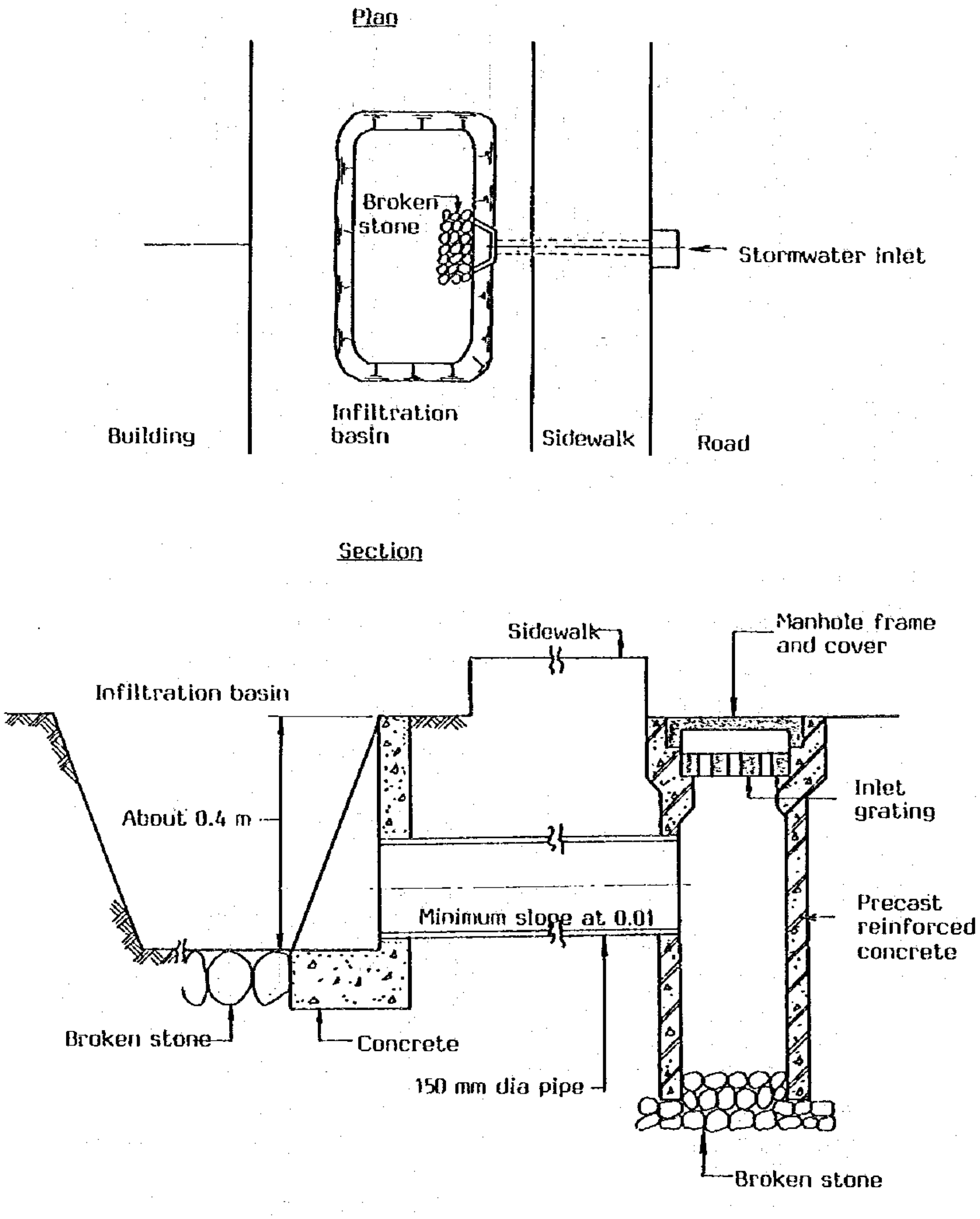


Figure 3.6.26. Typical Stormwater Infiltration Basin

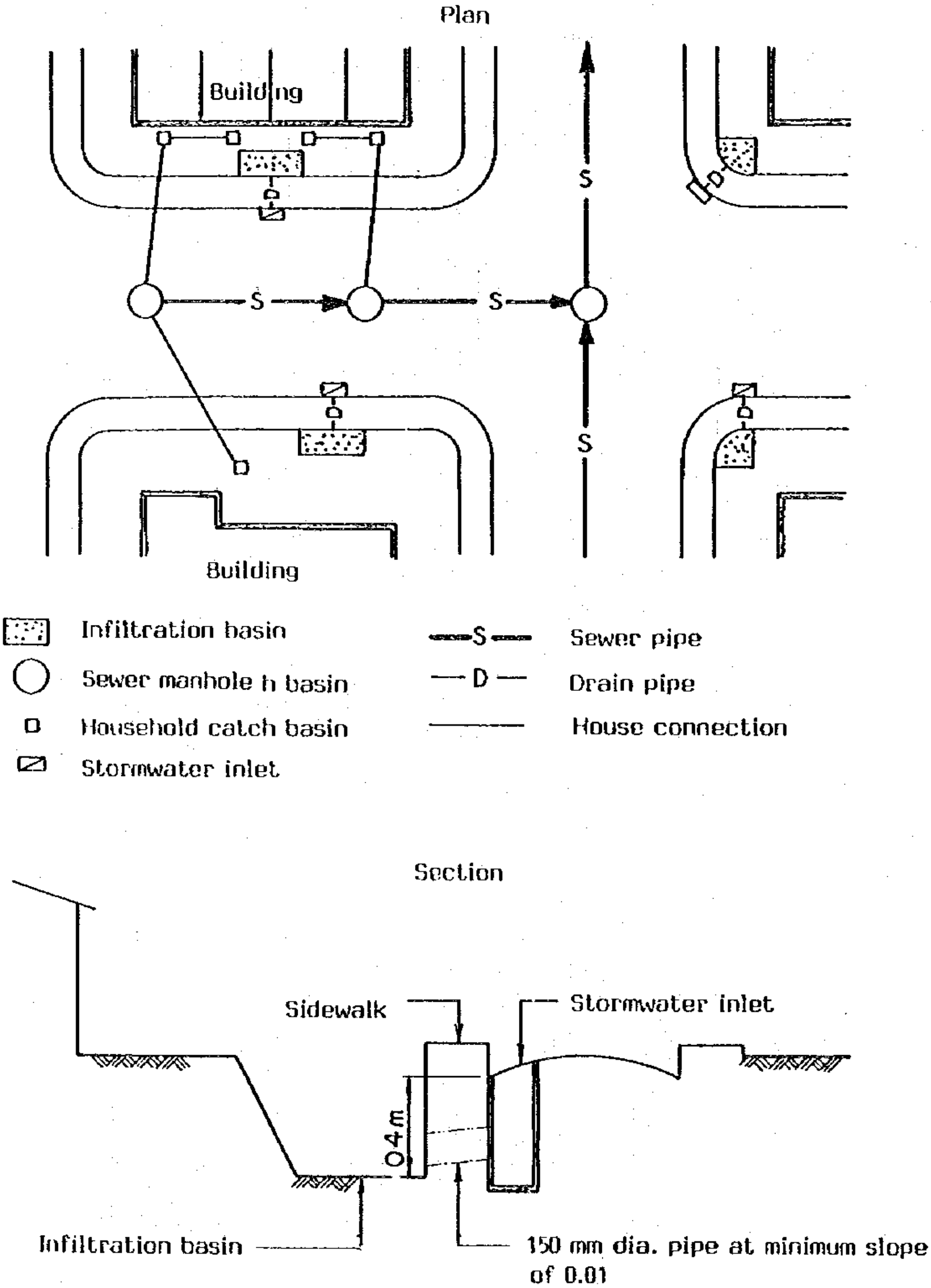


Figure 3.6.27. Typical Layout of Sanitary Sewerage and Stormwater Drainage Facilities

3.7. DESIGN CRITERIA FOR FACILITIES

In general, and except for special reasons, the sewerage and drainage system shall be planned and designed on the basis of the following design criteria:

3.7.1. Sewers and Drains

(a) Design Period

In general, sewer systems should be designed for the estimated tributary population in the year 2005, except in considering parts of the systems that can be readily increased in capacity.

(b) Design Factors

In determining the required capacities of sanitary sewers the following factors should be considered:

- Maximum hourly sewage flow.
- Additional maximum sewage or waste flow from industrial or other facilities where necessary.
- Groundwater infiltration.
- Topography of area.
- Location of the sewage treatment plant.
- Depth of excavation.
- Pumping requirements.

(c) Design Basis

- Sewers shall be not less than 200 mm in diameter except for house connection pipes.

- The Manning's equation shall be used for gravity sewers and stormwater drains in the form:

$$V = 1/n\ R^{2/3}\ S^{1/2}$$

where

V = velocity of flow, in m/sec

n = coefficient of roughness

R = hydraulic radius, in m

S = slope

Table 3.7.1. Coefficient of Roughness for Various Pipe Materials

Type of Pipe	Coefficient of Roughness (n)
Vitrified clay pipe (V.C.P.)	0.013
Plastic pipe (P.P.)	0.010
Concrete pipe (C.P.)	0.013

(d) Velocity of Flow

All sewers shall be so designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/sec, based on Manning's formula using an "n" value as shown in Table 3.6.1.above. The velocity shall not exceed 3.0 m/sec in any type of sewers to protect sewer erosion.

(e) Slope

The following are the minimum slopes which should be provided; however, slopes greater than these are desirable:

Table 3.7.2. Minimum Sewer Slopes

Sewer Size (mm)	Minimum Slope in Metres per Metre
150	0.01000 (House connections)
200	0.00330
250	0.00245
300	0.00192
350	0.00157

(to be continued)

Table 3.7.2.(2)) Minimum Sewer Slopes (continued)

Sewer Size (mm)	Minimum Slope in Metres per Metre
400	0.00131
450	0.00112
500	0.00097
600	0.00076
700	0.00062
800	0.00052
900	0.00044
1000	0.00039
1100	0.00034
1200	0.00030

(f) Alignments

Sewers shall be laid with straight alignment between manholes.

(g) Increasing Size

When a smaller sewer joins a larger one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the crown of both sewers at the same elevation.

(h) Joints and Infiltration

Sewer joints shall be designed to minimize infiltration and to prevent the entrance of roots or other obstacles.

(i) Manholes

Manholes shall be installed at the end of each line; at all changes in grade, size, or alignment; at all intersections; and at distances as shown in the following table:

Table 3.7.3. Manhole Spacings

Sewer diameter (mm)	Maximum manhole spacing (m)
200 or smaller	30
200 to 450	45
450 to 600	60
600 to 1000	80
1000 to 1500	120
Larger than 1500	200

The minimum diameter of manholes shall be 900 mm; larger diameters are preferable.

3.7.2. Pumping Stations

Sanitary sewage pumping stations shall be designed on the basis of the peak flow rates. All pipings and conduits shall be designed to carry the expected peak flow rates. The following items should be given considerations in the design of sanitary sewage pumping stations.

(a) Type and Structure

For large pumping stations to lift the sewage from submain or main sewers should in general be of the dry well type. Wet and dry wells, including their superstructure, shall be completely separated. Provision shall be made to facilitate removing pumps and motors. Suitable and safe means of access shall be provided to dry wells of pumping stations and shall be provided to wet wells containing either bar screens or mechanical equipment requiring inspection or maintenance.

For intermediate sewage pumping stations required for lifting the sewage of branch and/or submain sewers should be of submersible type provided in a manhole. Submersible pumps shall be readily removable and replaceable without dewatering the wet well and with continuity of operation of the other unit or units. Submersible pumping stations shall meet the applicable requirements for the dry well type pumping stations unless otherwise specified.

(b) Screening

For dry well type pumping stations, protection for pumps and other equipment shall be provided by installing bar screens. All facilities should be readily accessible for maintenance. Clear openings between bars for mechanically cleaned screens should be 25 mm. For mechanically cleaned screens, maximum velocities of flow through the screen should 0.8 m/sec. The velocity shall be calculated from a vertical projection of the screen openings on the cross-sectional area between the invert of the channel and the flow line.

Channels shall be equipped with the necessary gates to divert flow from any one screening unit. Provisions must also be made for dewatering each unit. Amply-sized facilities must be provided for removal, handling, storage, and disposal of screenings in a sanitary manner. Suitable drainage facilities shall be provided both for the platform and for storage areas.

(c) Grit Removal Facilities

Grit removal facilities should be provided in principle for sewage pumping stations. Where it may be necessary to pump the sewage prior to grit removal, the design of wet wells should receive special attention and the discharge piping shall be designed to prevent grit settling in pump discharge lines of pumps not operating.

Grit removal facilities should have at least 2 units. Channel-type chambers shall be designed to provide controlled velocities as close as 0.3 m/sec, with the detention period of 30 seconds for the average daily flow rate. The overflow rate of the channels should be of 1,880 m^3/m^2/day. The design should take into consideration undesirable turbulence and velocities at inlets and outlets.

Grit facilities located in deep pits should be provided with mechanical equipment for pumping or hoisting grit to ground level. Such pits should have a stairway, adequate ventilation, and adequate lighting. Impervious surfaces with drains should be provided for grit handling areas. It grit is to be transported, the conveying equipment should be designed to avoid loss of material. An adequate supply of water under pressure shall be provided for clean up.

(d) Pumps

At least 2 pumps shall be provided for all the types of pumping station. Pumps should be designed to fit actual flow conditions and must be of such capacity that with any one unit out of service the remaining units will have capacity to handle maximum sewage flows.

Submersible pumps shall be capable of unsubmerged operation without damage or reduction of service capability or positive provision shall be made to assure submergence.

(e) Controls and Valves

Controll equipment should be so located as not to be unduly affected by flows entering the well or by the suction of the pumps. Float tubes in dry wells, where used, shall extend high enough to prevent overflow. Suitable shutoff valves shall be placed on suction and discharge lines of each pump. Acheck valve shall be placed on each discharge line, between the shutoff valve and the pump.

(f) Wet Wells

The effective capacity of the wet well shall provide a holding period not to exceed 10 minutes for the design average flow. The wet well floor shall have a minimum slope of 1 to 1 to the hopper bottom. The horizontal area of the hopper bottom shall be no greater than necessary for proper installation and function of the inlet.

(g) Ventilation

Adequate ventilation shall be provided for all pumping stations. Where the pump pit is below the ground surface, mechanical ventilation is required, so arranged as to independently ventilate the dry well and the wet well if screens or mechanical equipment requiring maintenance or inspection are located in the wet well.

(f) Flow Measurement

Suitable devices for measuring sewage flow should be provided at the major pumping stations.

(g) Alarm System

Alarm system should be provided for pumping stations where it is practicable. The alarm shall be activated in cases of power failure, pump failure, or any cause of pumping station malfunction.

(h) Emergency Power Supply

Provision of an emergency power supply for major pumping stations should be made. For the emergency power supply, in-place internal combustion engine equipment should be povided with a unit size adequate to provide power for lighting, ventilation and pump unit and such further systems affecting capability and safety.

(i) Overflows

The provision of a high-level wet well overflow to supplement alarm systems and emergency power generation should be considered. Where a high level of overflow is utilized, consideration shall also be given to the installation of storage-detention tanks, or basins, which shall be made to drain to the station wet well.

(j) Force Mains

At design average flow, a cleansing velocity of at least 60 cm/sec shall be maintained. Automatic air relief valves shall be placed at high points in the force main to prevent air locking. Force mains should enter the gravity sewer system at a point not more than 60 cm above the flow line of the receiving manhole.

3.7.3. Sewage Treatment and Disposal System

The following items should be taken into consideration in planning and designing the sewage treatment plant facilities:

(a) Type and Degree of Treatment

On account of the effluent reuse for irrigation and other purposes, and also for the environmental protection of the area, the oxidation ditch treatment process is recommended. The expected qualities of the influent and the effluent are as follows:

Item	Concentration (mg/l)	
	Influent	Effluent
BOD	270	27
SS	250	37

(b) Hydraulic Loadings

In general, the design of the sewage treatment units shall be based on the average daily rate of sewage flow per 24 hours. All pipings and conduits shall be designed based on the peak rate of flow. The following loadings are the recommended design standards for each of the units:

Table 3.7.4. Standard Hydraulic Loadings

Units	Hydraulic Loadings
Screens	
Velocities of flow through bars	0.5 to 0.8 m/sec
Grit chambers	
Overflow rate	1,880 m3/m2/day
Detention time	30 seconds
Velocity of fow through channel	0.3 m/sec
Oxidation ditches	
Detention time	24 hours
Channel velocity	0.3 m/sec
Return sludge	50 to 150 % of inflow sewage
Sedimentation basins	

(to be continued)

(continued)

Units	Hydraulic Loadings
Overflow rate	15 m3/m2/day
Detention time	5.0 hours
Weir loading	120 m3/m/day
Chlorine contact tanks	
Chlorine contact time	20 minutes
Effluent storage tanks	
Detention time	6 hours
Sludge storage tanks	
Detention time	1 day
Sludge drying sand beds	
Sludge depth of application	0.2 metre
Sludge drying time	5 days

(c) Organic Loadings

In general, the design organic loadings shall be computed in the same manner used in determining design flow. Standard organic loadings for each treatment plant unit are recommended in the table below.

Table 3.7.5. Standard Organic Loadings

Unit of Facilities	Organic Loadings
Oxidation ditches	
Aerator BOD loading	0.3 kg BOD/m3 of ditch
MLSS	3,000 to 6,000 mg/l
SRT	20 to 40 days
Oxygen transfer efficiency	1.0 kg O_2/hp-hr
Dissolved oxygen in aeration tank	2 mg/l
Excees sludge solids produced	6.0 x 10^{-5}% of sewage
Concentration of excess sludge	0.8 % of sewage
Excess sludge production	150 m^3/day

(d) Arrangement of Units

Component parts of the plant should be arranged for greatest operating convenience, flexibility, economy, and so as to facilitate installation of future units.

(e) By-Passes

Except where duplicate units are available, properly located and arranged bypass structures shall be provided so that each unit of the plant can be removed from service independently.

(f) Drains

Means should be provided to dewater each unit. Due consideration should be given to the possible need for hydrostatic pressure relief devices.

(g) Construction Materials

Due consideration should be given to the selection of materials which are to be used in sewage treatment facilities because of the possible presence of hydrogen sulphide and other corrosive gases, greases, oils, and similar constituents frequently present in sewage.

(h) Emergency Power Facilities

A standby power sources shall be provided to ensure tha continuous operation of such important equipment as influent pumps, minimum number of aerators, and emergency lighting.

(i) Essential Facilities

Necessary facilities for operation and maintenance of the plant shall be provided, including:

- Water supply facilities
- Drainage facilities

- Plant roads and parking facilities
- Service facilities
- Connecting conduits

(j) Safety

Adequate provision shall be made to effectively protect the operator and visitors from hazards.

(k) Oxidation Ditches

The dimensions of each independent aeration ditch shall be such as to maintain effective mixing and utilization of air. Liquid depth should be in general 2.5 metres. Inlets and outlest for each aeration tank unit shall be suitably equipped with valves, gates, stop gates, or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The hydraulic properties of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration ditch unit out of service.

The mechanism and drive unit shall be designed for the expected conditions in the ditch in terms of the proven performance of the equipment. Multiple mechanical aeration unit installations shall be so designed as to meet the maximum air demand with the largest unit out of service. The design should also provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

If motor driven return sludge pumps are used, the maximum return sludge capacity shall be obtained with the largest pump out of service. A positive head should be provided on pump suction. Pumps should have at least 75 mm suction and discharge openings. Suction and discharge pipings should be at least 100 mm in diameter and should be designed to maintain a velocity of not less than 0.6 m/sec when return sludge facilities are operating at normal return sludge rates. Suitable devices for observing, sampling and controlling return activated sludge flow from each sedimentation basin shall be provided.

(l) Sedimentation Basins

Inlets should be designed to dissipate the inlets velocity, to distribute the flow equally and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 m/sec at one-half design flow. Corner pockets and dead ends should be eliminated and corner fillets or channeling used where necessary. Provisions shall be made for elimination or removal of floating materials in inlet structures having submerged ports.

The liquid depth of the mechanically cleaned sedimentation tanks shall be as shallow as practicable but not less than 2.5 metres. Effective scum collection and removal facilities, including baffling, shall be provided ahead of the outlet weirs on all sedimentation basins. Provisions may be made for discharging of scum with the sludge.

Overflow weirs shall be adjustable. Weir loadings should not exceed 120 m3/m/day.

A sludge well should be provided or appropriate equipment installed for reviewing and sampling the sludge. Provisions should be made to permit continuous sludge removal from final sedimentation tanks when the sludge is returned to the ditches. Each sludge hopper shall have an individually valved sludge withdrawal line at least 150 mm in diameter. Head available for withdrawal of sludge shall be at least 0.9 metre. Sludge hoppers shall be accessible for maintenance from the operating level. The minimum slope of the side walls of sludge hoppers shall be 1.7 vertical to 1 horizontal. Hopper bottoms shall have a maximum dimension of 0.6 metre.

(m) Sludge Drying Sand Beds

The lower course of gravel around the underdrains should be properly graded and should be 0.3 metre in depth, extending at least 0.15 metre above the top of the underdrains. It is desirable to place this in 2 or more layers. The top layer of at least 75 mm should consist of gravel 3 to 6 mm in size.

The top course should consist of at least 150 to 230 mm of clean coarse sand. The finished sand surface should be level.

Underdrains should be clay pipe or concrete drain tile at least 100 mm in diameter laid with open joints. Underdrains should be spaced not more than 6 metres apart. Walls should be water-tight and extend 0.4 to 0.5 metre above and at least 0.15 metre below the surface. Other walls should be curbed to prevent soil from washing on to the beds.

The sludge pipe to the beds should terminate at least 0.3 metre above the surface and be so arranged that it will drain. Concrete splash plates for sludge should be provided at sludge discharge points.

(n) Chlorine Contact Tanks

Disinfection usually is accomplished with liquid chlorine, calcium or sodium hypochlorite or chlorine dioxide. The chemical should be selected after due consideration of waste flow rates, application and demand rates, pH of waste, cost of equipment and the chemical, availability and maintenance problems.

Required chlorinator capacity should be detrmined to have a sufficient contact time between the chlorine compound and the sewage, but in general the contact time should be at least 20 minutes or longer to ensure bacterial destruction. Care should be taken to prevent short-circuiting by the provisions of appropriate baffling or other means.

An ample supply of water shall be available for operating the chlorinator. If a booster pump is required, duplicate equipment should be provided, and when necessary, standby power as well.

(o) Sludge Storage Tanks

Excess sludge will be stored in the storage tanks for about 1 day and then withdrawn by gravity to sand beds. The proportion of depth to width should be such as to allow for efficient sludge handling. The tank bottom should slope to drain towards the withdrawal pipe. To facilitate emptying, cleaning and maintenance, appropriate sludge inlets, outlets, valves and other necessary equipment should be provided. Clearance between the end of the sludge draw-off pipe and the hopper walls shall be sufficient to prevent 'bridging' of sludge solids.

CHAPTER-FOUR

THE PROPOSED PROJECT

Chapter Four

THE PROPOSED PROJECT

4.1. OBJECTIVES

The implementation period for the first stage project is expected to cover eight years 1985 - 1992. The principal social and economic objectives of the project are to provide and maintain adequate sewerage and effluent reuse facilities at affordable prices for all residents plus tourists in El-Arish city and its environs, including the treatment plant effluent supply services to support agricultural development within the Jarada area. This involves improvements in standards of sanitation services throughout the area, augmentation of water resources, and improvements in wastewater disposal.

Concurrently, a major objective is to strengthen the institutional arrangements for providing sewerage system so the residents can be assured of the continuous provision of these basic services.

A complementary objective of the project is to improve the contamination of the groundwater due in large part to the uncontrolled discharge of wastewaters into the ground through the transh system, thus preventing the possible future deterioration in levels of public health.

The principal operational objectives are elaborated hereafter in quantitative terms.

4.1.1. Improvements in Wastewater Treatment and Disposal

Existing systems of wastewater treatment and disposal in El-Arish City are unsatisfactory, as explained in Section 2.6. The transh system has been deteriorating the groundwater which is the source of the city water supply system. The rationalization of sewerage system proposed in the project is intended to eliminate these problems of wastewater disposal.

One indicator of the project objectives is the quantities of wastewater or sludge expected to be treated and disposed of by the proposed system, The present discharge of the wastewater is estimated to be about 15,000 m3/day which is expected to increase to the level of about 20,000 m3/day by the year 2005. If no action is taken to prevent this amount of wastewater to leach into the ground, the contamination of the groundwater will be significant. When the system is completed in the year 1992 as scheduled and commences its operation, the improvement of the groundwater quality and the resulting improved sanitary conditions will be tremendous.

4.1.2. Augmentation of Water Sources for Irrigation

In view of the saving of water resources, reuse of the treatment plant effluent is one of the most important strategies of agricultural development in El-Arish area. Of various schemes, top priority is laid on the agricultural development. It is well known that agriculture in El-Arish area is the most important economic sector, which now accounts for almost 30 per cent of the Gross Domestic Product.

The existing farm land of El-Arish, various crops as olive, dates, watermelon, cucumber, corn, tomatoes and others are harvested annually. Nevertheless, the total output lags behind the rate of population growth. In oder to increase the output, the NSG is now planning to promote the agricultural development using sewage treatment plant effluent as a new water source besides the development of the existing agriculture. According to the NSG's policy, an agricultural developemtn has been planned in the land of about 600 feddans extending within the Jarada area.

The proposed irrigation system aims to grow various kinds of crops throughout the year in the irrigation area of totally 611 feddans, by supplying a total of about 20,000 m3/day treatment plant effluent. The sewage will be treated at the sewage treatment plant located close to the Jarada farm land area from where the effluent will be conveyed through a main pipeline to the water tanks in the farm land. The proposed Jarada area may be allocated to settlers 12-feddan unit each. Leaving aside 12 feddans for the proposed experiemntal farm, the total area available for distribution amounts to about 600 feddans. This means that about 50 families will be settled involving a total farm population of about 300.

4.1.3. Improvements in Sanitation Service Levels

The transh system serves the El-Arish residents. The goal of the project is to provide adequate sewerage facilities for all by the year 2005 and to improve present unsanitary conditions and other various deficiencies caused by the present system. The population to be served by the new sewerage system in the year 2005 is estimated to be about 130,000 in addition to the expected 20,000 served population by the existing Masaid sewerage system, thus, by the year 2005 a total of 150,000 residents will be served by the sewerage system as a whole.

4.1.4. Institutional Improvements

The project is intended to serve as a catalyst in terms of rationalizing the responsibilities for providing such services, particularly sanitation. Furthermore, each organization with a role in providing sanitation services will be strengthened. The overall intention is to assure the continuous provision of these basic services to all residents.

4.2. PROJECT DESCRIPTION

The proposed project comprises several related components, each described separately. Extensive planning and analysis have been undertaken during the feasibility study in order to confirm the technical feasibility and cost estimates for all components. Preliminary engineering designs have been completed for all facilities and detailed designs for facilities to be commenced during the first year can be prepared within several months of a decision to proceed.

This report provides technical information consisting mainly of summary data and drawings. However, considerable engineering data and analyses on which this part of the report is based are available in several Technical Appendices which are included separately in Volume Three of this report.

4.2.1. Sewerage Districts

The sewerage districts encompass a total area of about 1,000 ha, covering El-Arish City, Masaid, Salem and certain precincts. The districts have been examined in view of existing physical and developmental conditions, future development schemes, and topographic conditions. The sewerage districts finally defined are described in succeeding paragraphs. Figure 4.2.1. shows the location of sewerage districts and precincts.

(a) El-Arish District

This district covers most of the builtup urban area of El-Arish City and comprises major residential, commercial, institutional, and industrial districts. Also, small hotels and other facilities for tourists have a large continuous transient population. The present population within this district is estimated to be approximately 51,000. In addition, a total of about 13,000 tourists and other transients stay in El-Arish over short period, particularly in summar season. Thus, the total present population within the district is estimated to be about 64,000.

As may be seen from Figure 4.2.1., this district stretches from the beach in Masaid area to the Wadi El-Arish, serving a total area of 543 ha plus

160 ha of precincts tributary to the district due to topographic feature. The ground slope of the district runs either to the easterly direction towards the Wadi El-Arish, or to the north towards the Sea. The ground elevations range from 2 metres above M.S.W.L.at the beach to 37 metres above M.S.W.L. at the highest location near the grave yard.

Road network is relatively well provided within the district. In the north of the district, a main road connecting El-Arish to Rafah runs from west to east, but other streets within the district are mostly unpaved with the widths ranging from 8 to 10 metres.

This district is suitable for tourim development because of the favourable location for hotel industry. The Government placed the tourism indutry as one of the highest priority programme, and the rapid development is expected particularly along the beach. In summary, this district is the core of the sewerage system development.

(b) Salem District

This district is on the east side of the Wadi El-Arish and south of the Abu Saghal harbour, comprising El-Salem and Abu Saghal areas. The Salem area encompasses a large scale housing area and predominantly residential with small scattered commercial, institutional, and green areas. The ground slope is in general flatter than that in the El-Arish district, gradually declining towards the north direction. The ground elevations range from 15 metres above M.S.W.L. in Abu Saghal and 20 metres above M.S.W.L. in Salem area.

Presently, about 5,000 population resides in the district. There will be some increase in residential use as the housing development scheme is now unerway and the governments' apartment housing complex is expected to be completed in the immediate future, thus a total of about 30,000 population will reside in the district of 191 ha.

(c) Masaid District

This district is newly developed housing area with the planned future population of 20,000. A sanitary sewerage system has been constructed by the NSG to collect all the wastewater discharged from the households in the apartment complexes to the collection system and to convey the wastewater to an extended aeration treatment plant located at the southwest of the district. The plant effluent is planned to be used for the crop irrigation at the nearby farm land. The sewerage is the separate system that carries only sanitary sewage, but the stormwater runoffs will be soaked to soil through stormwater infiltration basins.

The ground elevations range from 1 to 4 metres above M.S.W.L. The ground slope gradually runs towards the coast but is mostly flat.

Because this district has already been covered by the existing sewerage system constructed by the NSG, no additional sewerage and storm drainage system facilities are included in this project, and necessary guidelines to strengthen the existing sewerage and drainage facilities are recommended.

The sewerage districts are listed in the following:

Name of District	Sewerage Service Area (ha)
El-Arish District	
El-Arish sub-district	432
Beach area sub-ditrict	111
Tributary area	160
Sub-total	703
Salem District	
El-Salem sub-district	114
Abu-Saghal sub-district	43
North El-Salem sub-district	34
Sub-total	191
Masaid District	
Masaid district	106
Sub-total	106
Total Sewered Area	1,000

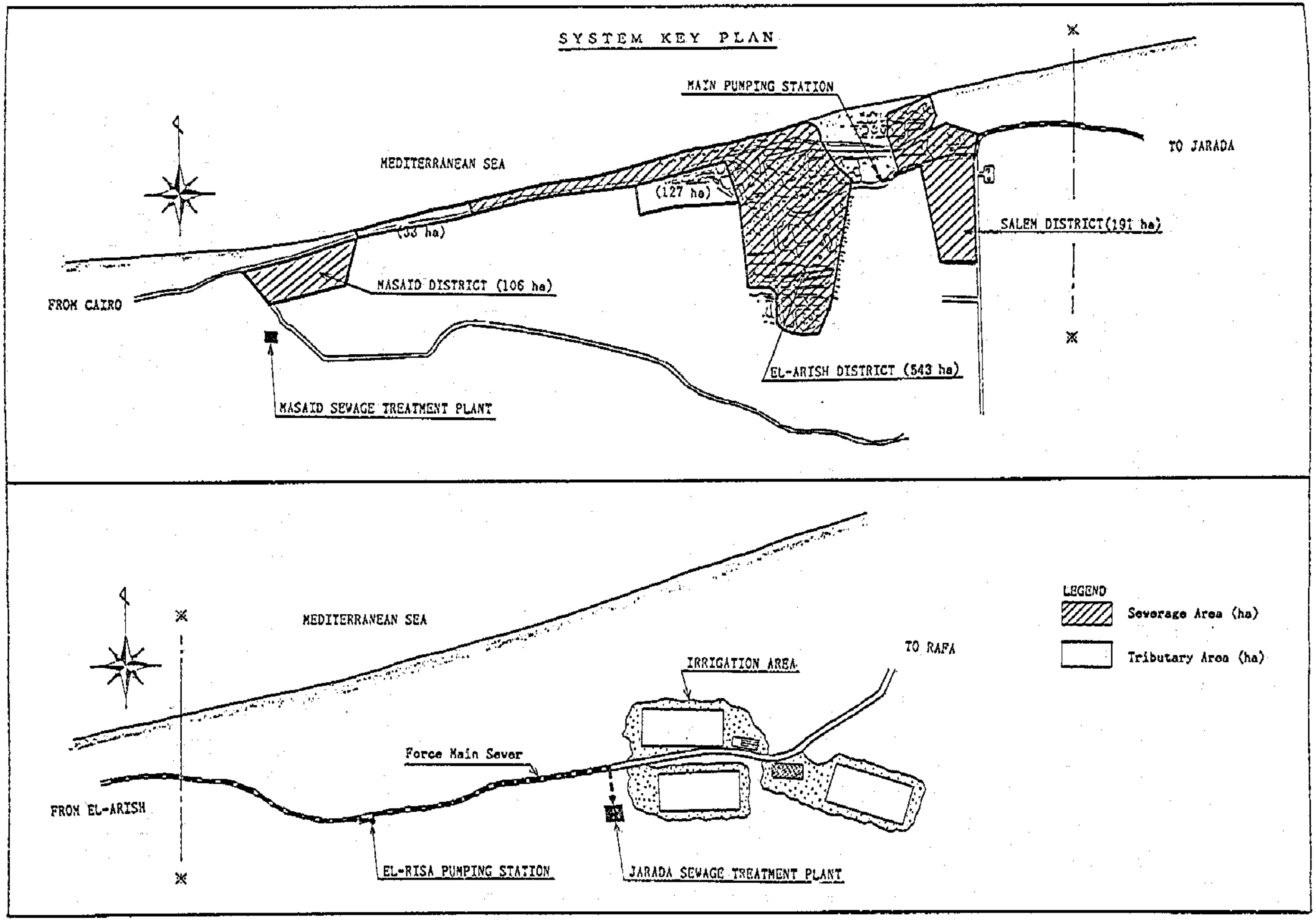


Figure 4.2.1. Proposed Sewerage Districts

4.2.2. Sanitary Sewers

To collect the sewage from the service areas and finally to convey it to the sewage treatment plant to be constructed at Jarada area, the project includes the provision of new sanitary sewer reticulations consisting of main, submain, branch, lateral and house connection sewers. As shown in Figure 4.2.2., the recommended sewerage is the separate system to collect only sanitary sewage, but the stormwater runoffs are collected separately either to transh or infiltration basin to absorb the water into ground.

As illustrated in Figure 4.2.3., the sewage collected from households through the house connections flows by gravity to lateral or branch sewers and then led to submain sewers. When the sewer earth covering becomes 4 metres or more, the sewage is lifted by manhole type submersible pump stations to the elevation sufficient to continue the gravity flow in the sewers. The sewage is further lifted at the Main pumping station located at the right bank of the Wadi and transmitted by pressure to the El-Risa intermediate pumping station that boosts the water pressure sufficiently enough to reach the sewage treatment plant.

Although profiles for small sewers have been not prepared for the purpose of feasibility study, for the major branch and lateral sewers which are influential in determining the invert elevations in submain and main sewers, profiles have been prepared to examine whether main or submain sewers could receive the sewage from the most remote location of the tributary area. Maps used for the planning are those of 1:5,000 in scale, which are adjusted and revised for ground elevations on the basis of the topographic survey carried out under this study to indicate the latest conditions

Housing estate development programmes are currently unerway by governments and private sectors in Ayaiba district, however, for this area road netwrok plan is still at the preliminary planning stage. Due to topographic reasons, this area will have to discharge the sewage into the El-Arish sewerage district in the future when the area is fully developed. In view of these, the capacities of the downstream sewers in the El-Arish district are designed to enable to handle such a future increase of sewage inflow.

Sewer sizes have been determined on the basis of the projected population densities, per capita sewage flow rates and extraneous inflows, for the year 2005, as shown in Figures in Volume Three, 'Drawings.' In planning the sewer profiles, sufficient earth coverings were left between the top of sewer and the bottom of the roadway surface to protect the sewers from traffic loads and to avoide undue interference with other underground facilities, with a minimum covering of 1 metre, except for specific situations where shallower depth is feasible. The maximum earth covering of sewers is generally 4.5 metres.

All sewers are designed to give mean velocities when flowing full or half-full, of not less than 60 cm/sec for all pipes, based on the Manning formula using an 'n' value of 0.013.

Manholes are not indicated on the plans and profiles of the sewers but should be provided at each change in direction, change in grade, and change in sewer diameter, with generally maximum spacings as described in Section 3.7.
Except for very shallow sewers, all manholes are planned to have adequate dimensions for entry and for operation of cleaning equipment.

Standard structures of sewers and manholes are shown in Volume Four 'Drawings,' and cost estimating procedures are described in Appendix - Two of Volume Three.

The proposed sewers range from 200 mm in diameter to 900 mm in dia. for the gravity flow pipes, and 100 to 500 mm dia. pipes for pressure sewers. Sewer dia-length by diameter both for gravity flow and pressure flow are listed in Table 4.2.1. below.

Table 4.2.1. Proposed Sewers by Diameters and Purposes

Sewer Diam. (mm)	Sewer Length (m)	Material	Remarks
200	156,085	Clay or plastic	For gravity flow
250	3,210	"	"
300	3,650	"	"
375	3,920	"	"
450	2,420	"	"
500	1,310	"	"

(To be continued)