WASTE STABILIZATION LAGOONS FOR SCHOOL OF AGRICULTURE FARM

By

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PREFACE

The concept of sewage disposal by waste stabilization lagoons has gained popularity by many people in the world as a means for treating raw sewage. Every year more and more lagoons are being constructed in various parts of the world, including U.S.A., Costa Rica, Saudi-Arabia, Australia, and North Rhodesia. So far, the majority of the lagoons constructed, were reported to have operated satisfactorily. In Lebanon, however, sewage lagoons are not in use.

The present arrangement of septic tanks at the A.U.B. farm is proving unsatisfactory. The study of the sewage collection, treatment and disposal system at the A.U.B. farm, was undertaken as a graduate project. The intention was to correct and improve the existing system, or of recommending a new system together with necessary drawings, specifications, and cost estimates of the recommended system, while offering another possible solution.

This presentation includes a discussion of the historical development of sewage lagoons, their theory, design, application, and operation. Although the objective of this thesis is primarily to design sewage lagoons for the agricultural farm of the American University of Beirut, it was felt that such supplimentary information would be of interest to the reader, and of special value in serving as a working basis in the design of this specific project.

The quantitative data for the study was limited and some of it was of doubtful accuracy. The data upon which the design and recommendations were made should be checked, and if found substantially different from the facts given, the design should be revised.

The author wishes to acknowledge the assistance offered by several colleagues and professors. Special thanks are due to Professor Samir El-Khuri for his advice and encouragement; to Professor Edward Hope for his help in the initial stage in the preparation of this thesis; to Professors Khalil Malouf and Gabriel Matta for their helpful suggestions on the hydraulic aspects of the project; to Professors Aftim Acra and Oris Blackwell for supplying some of the literature on the subject; to Mr. George Ayyoub of the Associated Consulting Engineers Office; and to Mr. Van Marais of Northern Rhodesia for their valuable suggestions.

The author also wishes to thank all others who have contributed in various ways to the thesis presented.

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SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Septic tanks, although cheapest of all proposed desigsn, are not recommended for reasons discussed in the text.

With regard to conventional units, the minimum cost of construction would amount to a value equal to that of the designed waste stabilization lagoons if constructed by contract. The main difference, however, is that for the same price, waste stabilization lagoons will serve a higher population. In other words, for the same population, waste stabilization lagoons will be less than conventional units both in initial (construction) cost and the operating costs.

Due to the availability of earth moving equipment on the farm, the actual cost if done by farm personnel under adequate supervision, rather than by contract as estimated, would be reduced by approximately L.L. 7000. This would make the actual cost of the ponds only L.L. 22,000, and the total cost of the proposed design only about L.L. 33,000.

Pending authorization for the construction of this project, the author recommends that more accurate data be obtained (see Chapter VI).

CHAPTER I

INTRODUCTION

It is felt necessary before attempting to solve the existing problem at the farm, to gather some data about the farm and mention some of the various possible solutions for such a problem.

A. Conditions at the A.U.B. Farm.

1. General

The A.U.B. Farm is located in Bekaa Valley (33^{o} - 55' N, 36^{o} - 05' E), in the area called Housh Sneid, near the villages of Al-Nabi Chite, Al-Khodre, Beit Shamma, and Houch Rafqua. The farm area amounts to about 250 acres. The land slopes gently from East to West.

Drawing 1 shows the location of the different buildings at the farm with respect to the general plan. With regard to the area of the farm, no expansion has been made, nor is any expansion contemplated. However, more buildings, other than those shown on Drawing 1 may at some future time be built.

The surface soil (30 cms. deep) of the farm was tested and found to consist mainly of clay. Some holes were dug on the site area, and the results showed an upper surface of 2-3 ft. of soil, and a pervious layer beneath consisting of gravel and small pieces of rock.

Other information about the soil, concerning grain size distribution and percolation rates into the ground, are placed in Appendix C.

Reference to Table C_1 (p.101) gives an idea about the climate of the farm. The data given in Table C_1, is the most important to the problem.

2. Sanitary

a. Population - Residential and Industrial

The residential population is highest in the summer and is lowest in September. In summer (July and August), the estimated population is 190 persons, including students, workers, and staff. In September this drops to a maximum of 50 persons. In winter (October-June) the number will be in the range of 100 persons.

In a slaughter and processing plant, and in a creamery, the population equivalent of the wastes produced are given hereunder for purposes of comparison with domestic sewage (1):

Slaughter house

1 cow	= 36 persons
120 chickens	= 60 persons
10 sheep	= 30 persons

Creamery

100 kgs. of milk = 26 persons

These equivalents may be used in calculating the load of the liquid wastes discharged from the slaughter house and creamery plant into sewers carrying domestic sewage.

Based on data given by the farm personnel, it is said that killing of animals might amount to either 120 chickens, 10 sheep, or 1 cow at a time. The maximum population equivalent as shown above is 60 persons.

With regard to the creamery, about 100 kgs. of milk are processed into cream at a time. Wastes produced by this conversion of 100 kgs. of milk into cream, are equivalent to wastes produced by 26 persons, having in mind that the skimmed milk is disposed of by other means. However in some unusual cases (as was stated by farm staff) some of this is disposed of through the sewer. There is no doubt that in such unusual cases, the population equivalent will be much more than 26 persons.

Considerations of the equivalent population for the wastes produced in the clinic, laboratory, shops, and chicken barn were not possible. However, waste products produced in the above mentioned units contribute to the total population equivalent.

It is clear that the total minimum population equivalent of the farm is at least equal to the sum of 190, 60 and 26, which amounts to a total of 276 persons. In any case, the original design was not based on total population (both human and industrial), but rather on the load derived from the B.O.D. values obtained experimentally, and the amount of swage produced. The B.O.D. load is 285 ppm*, and the amount of sewage is 160 cu.m/day**.

The total weight of B.O.D. produced per day by the above results (sewage flow of 160 cu. m. per day with a B.O.D. load of 285 ppm.) is equal to:

$$\frac{285 \times 160{,}000}{1{,}000{,}000} = 45.5 \text{ kgs.} = 100 \text{ lbs.}$$

The usual B.O.D. production per person per day amounts to about 0.17 lbs. Based upon this value, the total population equivalent will be:

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* See Table C_3, p. 105

** See p. 6

$$\frac{100}{0.17} = 560 \text{ persons}$$

In spite of the fact that this figure of 560 is almost twice 276 persons calculated before, it is still believed that the value of 560 persons is more reliable than 276 equivalent persons, especially since it was shown that the equivalent population of the farm is more than 276 persons.

This assumption of 560 persons will be the basis of the designs that follow in the next chapter, however it is advisable to verify the above data before construction takes place.

b. Disposal and Collecting Systems

i. Refuse: After collection, the refuse is disposed by incineration and buried in trenches near the hill site found on the farm. A visit to the site makes one critisize the situation and proves the ineffectiveness of such method. The reason is that paper and other light matter are carried by the wind to the neighbouring areas, making them ugly and unsightly. The author suggests that a proper incinerator be built to prevent such light matter from being scattered. Other forms on nuisance result from the type of refuse disposal system now being used; but in this report, discussion of such forms will not be included.

ii. Sewage: The present sewage is treated by means of four septic tanks. The results are not satisfactory; this is believed to be due to improper design. The existing spetic tanks have dimensions of about 2 x 2 x 2 m. Good design dimensions for a septic tank serving about 160 persons are 2.5 x 5.5 x 3.0 m (2). Frequent troubles result in bad odors, and surface flooding of the sewage.

The inlets and outlets of the septic tanks have no baffles; consequently short circuiting takes place easily and very often.

The cost of maintenance and operation of the existing sewage disposal system amounts to about L.L. 250 per year for each of the four existing tanks.

Results of the experiments performed on sewage samples from the different septic tanks in the farm are shown in Table C_3. The overall concentration of the sewage produced in the farm is equal to 285 ppm of 5 day B.O.D. incubated at 20° C.

iii. Water: There are two wells on the farm. From the first well in Block 15, water is pumped directly to an elevated tank (lower reservoir) with a capacity of 37 cu.m. From this the water goes to a chlorinator, and is then pumped to the upper reservoir with 43 cu.m. capacity.

Quantitative data of the domestic water used in summer at the farm is given by two people. The first, Prof. Maksoud, gives a figure of 137 cu.m. per day for the year 1961. The second, given by Mr. Khashadourian (farm superintendent) is about 160 cu.m. per day for the year 1962.

In winter time (October-June), the value is estimated at 86 cu.m. per day.

During September no students are at the farm, but a few staff and workers who remain consume about 40 cu.m. per day.

The quantities given above represent the water consumed for domestic purposes, and other culinery uses. Lawns are watered with irrigation water. According to Steel (3), 75% - 125% of the water consumed is converted to sewage. In the absence of any data a value equal to 100% is considered. Based upon this, the sewage flow will be.

- Q Summer = 160 cu.m. per day.
- Q Winter = 86 cu.m. per day.
- Q September = 40 cu.m. per day.

The other well is used for irrigation water. The volume of water pumped is 750 cu.m. per day during the period from 1st of May - 30th October. The cost of this amount is LL. 3022,35* in 1962.

$$3022,35 \div 183 = \text{LL. } 16.51 \text{ per day.}$$

or 750 cu.m. of water cost 16.51

1000 cu.m. of water cost

$$\frac{1000 \times 16.51}{750} = 22.01 = \text{LL. } 22.$$

B. Possible Solutions.

To overcome the existing problem at the farm, several solutions could be attempted; however, the author mentions only three possible solutions. The first one is to make use of the existing septic tanks with any necessary modifications. The second possible solution is the use of waste stabilization lagoons replacing the old system of septic tanks. The third possible solution, is the use of any other suitable treatment plant which is found to be superior to the first or second solution with regard to economy, and performance.

1. Septic Tanks

As was shown earlier, the existing septic tanks are not operating properly. The tanks are obviously inefficient, and there is a clear need for a new system. No doubt, these existing tanks can be

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*This cost does not include depreciation, operation, and maintenance costs. If these are included, the above calculated cost of L.L. 22 to pump 1000 cu.m. of water will be increased.

modified or even demolished and new and better septic tanks based on exact design loadings be placed to serve as a means for the disposal of the farm sewage. However, the author feels that septic tanks should be considered as only a temporary means for sewage treatment for such a large institution. They are good to serve a community for a limited time until enough funds are raised to construct a much better means of sewage treatment. In addition, the existing septic tanks are possible causes in contaminating the underground water that is used by the scattered wells in the neighbouring area in general, and in particular the farm well lying downstream from the septic tanks.

It is believed that such causes will convince the Administration of the American University of Beirut to think of another system that is permanent, especially now that the time for constructing such a project has come.

2. Waste Stabilization Lagoons

Complete discussion about lagoons is placed in Appendix B. This is felt necessary for the better understanding of this process. A summary of the advantages that this process contributes is outlined in what follows:

a. Direct Advantages

Lagoons in general give satisfactory results as compared with other conventional units.

i. From the sanitary point of view, lagoons compare favourably with other conventional methods for treating sewage.

ii. Initial costs range from about 30% - 40% of the cost of conventional units. Since about 50% of this small cost constitutes the initial price of land, a more favourable situation arises since the land at the farm is available.

iii. Simplicity in maintenance, and operation makes the cost comparatively small. About 25% and not higher than 50% of the cost of maintenance and operation of other units have been recorded (4).

iv. Ease of expansion in the future if needed.

v. Sludge problems are absent, which reduces the problems to a great extent.

vi. The long storage time is effective in destroying many kinds of pathogenic bacteria and viruses like the poliomyelitis virus, that pass with the effluents in other treatment units. Because of this, in North Rhodesia ponding is to be installed following any other treatment unit.

vii. The capability of lagoons to absorb shock loads up to 500% of the influent B.O.D. makes it superior to other units. This fact has to be considered carefully, since the farm population and thus the amount of sewage produced is liable to such high changes.

b. Indirect Advantages

i. Fish breeding in ponds - Experiments performed in the United States, Europe, and Asia indicate that crops of fish can be grown economically and successfully.

ii. Well maintained lagoons may form a pleasant body of water. The observer can walk around the dikes and hardly notice any odors.

iii. Algae that grows in lagoons, can be recovered to serve as a good chicken and animal food.

iv. In cases where the land values increase, lagoon construction is very flexible, and thus can be transferred to another site in case of expansion of the city. The real loss is little as compared to the great benefits of high land values.

c. Physical Advantages

Conditions of soil and climate (wind, sunlight, and temperature) that are basic requirements for a successful lagoon are more or less present at the farm. The soil tested contains clay that satisfies the requirements of sewage ponding, which demands a wide spread area of clay. There is enough wind at the farm area to mix the pond contents, and to supply oxygen to the surface layers. Temperatures in summer are ideal for the good operation of such a unit, and in winter are warm enough to produce satisfactory results. In spite of the fact that winter temperatures are not as favourable as those of summer, good results are expected in view of the fact that winter loadings applied to the pond are decreased to about half those of summer.

d. Available Equipment

The use of the available equipment at the farm such as the scraper, the ditcher and the bulldozer will reduce the construction cost a great deal since the major cost in lagoons is for earth moving.

3. <u>Conventional Treatment Plants</u>

The third alternative involves the consideration of a conventional treatment plant, a trickling filter or an activated sludge unit with their other necessary units. However, it is not within the range of this thesis to present a complete design and study of such an alternative method.

Trickling filters or activated sludge plants have on an average shown no better results with regard to stabilization of sewage. B.O.D., suspended solids, and bacterial count reductions are comparable, more or less, with waste stabilization lagoons (4).

The maintenance and operation of such units are much more expensive and complicated than those of a waste stabilization lagoon. Construction costs of conventional units are more than those of waste stabilization lagoons (4).

With respect to this particular problem of the farm, the major cost for construction of the lagoons (the cost of the land) is already solved. Such major advantage is not available for conventional units.

In view of the fact that no detailed analysis will be done with regard to such units the author compares the total cost of the designed waste stabilization lagoon, with that of a conventional unit based on average costs in this part of the world.

The author does not advise the use of the septic tanks, even after repairs or reconstruction. Also he does not recommend the construction of a conventional unit, but instead believes that waste stabilization units are the most suitable process for treating the farm wastes.

C. Scope

In view of the relative unsuitability of septic tanks and conventional treatment plants as indicated above, the scope of this project is limited essentially to the determination of loading and the preparation of designs and specifications for sewage lagoons for the A.U.B. farm.

CHAPTER II

PROPOSED DESIGN

A. Network of Sewers:

The use of the existing sewage system was considered where possible. In the cases where it was considered, it was assumed that the size of the pipes was enough for any future expansion.

Drawing 1, shows the old sewerage system leading to the four different septic tanks (S_1, S_2, S_3, S_4). The proposed system is placed on the same drawing.

Asbestos cement pipes are used for this network except for some pipes in the plant that are exposed to high velocities and pressures. When pipes cross the road with a small cover that is less than 1.20 m, they are to be in concrete, as shown on drawing 3, to avoid any breakage.

For the design of the new system, some assumptions were made related to the distribution of domestic water to each building. These were calculated on the basis of number of people occupying each building. In the shops, slaughter house, laundry, bakery, boiler house, dairy, sheep barn, and kitchen a population equivalent was given with respect to water consumption and not B.O.D. production. On Table 1, it is found out that each person is given an estimated consumption of about 500 liters per day. This value is felt to be rather high, even as compared to maximum used; however, it is used in the absence of more exact data.

Table 1 - Water Consumption and Population Equivalent

Septic Tank	Building Unit	Population Equivalent	Water Consumption cu.m/day	Future Population*
S_1	Administration and Laboratory	15	7.50	30
	Village improvement	15	7.50	30
	Clinic and Dormitory Houses			
S_2	Work Shops	20	10.00	40
	Creamery	--	--	83
	Slaughter House	20	10.00	40
S_3	Workers Dormitory	20	10.00	40
	Students Dormitory	120	60.00	240
	Laundry	40	20.00	80
	Bakery	4	2.00	8
	Boiler House	2	1.00	4
S_4	Kitchen	30	15.00	60
	Staff Houses	30	15.00	60
Not Disposed in Septic Tanks**	Chicken Houses	4	2.00	8
	Dairy and Sheep Barn	6	3.00	12
	TOTAL	326	163.00	735

* Present population doubled.

** In the future will flow in the assumed Secondary 5.

For the proposed design, present values were doubled to take care of any expansion. This factor will not be considered in the design of the lagoons, however, as will be seen later, a similar set of lagoons were designed to take care of any such expansion.

In addition to these values used at the present, and given in Table 1, a value for the creamery building of 83 persons (41.5 m.c. per day)is considered. The creamery is assumed to have a daily capacity of 1000 gallons of milk. Also an additional value of 20 persons is given to sewer secondary 5 that is not present now, but might be installed later.

With regard to infiltration water, the water table is much lower than the level of the sewers, and the only water going into the pipe will be due to rain or irrigation water given to the grass in the gardens where the sewer passes. The amount of infiltration water is estimated as 15,000 gal. per day for a length of one mile of sewer (5). This value amounts to 0.40 liters per sec. per km.

Steel pegs with a red painted head were placed at the position of the manholes. Ground elevations and distances between the manholes were measured. These values can be seen on the profiles of the sewers in Drawing 3.

Table 2 shows the calculations followed to obtain characteristics of the sewers. Columns 14, 15 and 16 were found by the aid of a special slide rule supplied by the Eternit Company S.A.L. To find the ratio of maximum flow to the average (Column 8), a formula given by Babbit (5) is used:

$$Q_{max} = \left(1 + \frac{14}{4 + p} \right) q \text{ av.}$$

Table 2 Sanitary Sewer System

1 LINE No.	2 FROM M.H.	3 TO M.H.	4 Length in m.	5 Additional Population	6 Total Population	7 Av. Sanitary Flow l/sec.	8 Max. to Av. Flow	9 Max. Sanitary Flow l/sec.	10 Infiltration Increment l/sec	11 Commulative l/sec	12 Total Flow in Sewer l/sec.	13 Slope	14 Pipe Diameter cm.	15 Full Flow Velocity m/sec	16 Full Flow Capacity l/sec	17 Actual Velocity m/sec.
PRIM.	P18	P16	48.8	40	40	0.23	4.33	1.00	0.02	0.02	1.02	0.015	15	1.56	27.8	0.70
SEC.6	S6-1	P16	5.0	40	40	0.23	4.33	1.00	—	—	1.02	0.01	15	1.25	22	0.69
PRIM.	P16	P14	51.2	—	80	0.46	4.28	1.97	0.02	0.04	2.01	0.01	15	1.25	22	0.78
SEC.5	S5-2	P14	—	103	103	0.60	4.25	2.55	—	—	2.55	—	—	—	—	—*
PRIM.	P14	P13	50.8	—	183	1.06	4.16	4.42	0.02	0.06	4.48	0.007	15	1.04	18.4	0.86
SEC.4	S4-1	P13	39.4	124	124	0.72	4.21	3.03	0.02	0.02	3.05	0.0275	15	2.10	38.0	1.23
PRIM.	P13	P12	36.0	—	307	1.78	4.08	7.28	0.01	0.09	7.37	0.004	15	0.78	13.5	0.82
SEC.3	S3-3	P12	112.0	60	60	0.35	4.30	1.51	0.05	0.05	1.56	0.01	15	1.25	21.0	0.69
PRIM.	P12	P11	42.9	—	369	2.12	4.04	8.56	0.02	0.16	8.72	0.004	15	0.78	13.5	0.84
SEC.2	S2-3	P11	98.7	240	240	1.39	4.10	5.70	0.04	0.04	5.74	0.004	15	0.78	13.5	0.75
PRIM.	P11	P8	140.5	—	607	3.51	3.92	13.75	0.06	0.26	14.01	0.02	15	1.85	33.0	1.70
SEC.1	S1-1	P8	21.7	120	120	0.70	4.21	2.94	0.01	0.01	2.95	0.0156	15	1.58	28.0	0.98
PRIM.	P8	M.H-T5	393.85	-	727	4.21	3.88	16.35	—	0.27	16.62	0.0078	15	1.10	19.5	1.30
Treatment Plant Sewers	T1	POND	35.50	-	—	-	—	—	-	-	8.31	—	15	—	—	—
	T2	POND	20.0	-	-	-	-	-	-	-	8.31	-	15	-	—	—

*. Kept for future design if needed.

Table 2 . Sanitary Sewer System

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
									Infiltration					Full Flow		
LINE No.	FROM M.H.	TO M.H.	Length in m.	Additional Population	Total Population	Av. Sanitary Flow l/sec.	Max. to Av. Flow	Max. Sanitary Flow l/sec	Increment l/sec.	Commulative l/sec.	Total Flow in Sewer l/sec.	Slope	Pipe Diameter cm.	Velocity m/sec	Capacity l/sec.	Actual Velocity m/sec.
PRIM.	P18	P16	48.8	40	40	0.23	4.33	1.00	0.02	0.02	1.02	0.015	15	1.56	27.8	0.70
SEC.6	S6-1	P16	5.0	40	40	0.23	4.33	1.00	–	–	1.02	0.01	15	1.25	22	0.69
PRIM.	P16	P14	51.2	—	80	0.46	4.28	1.97	0.02	0.04	2.01	0.01	15	1.25	22	0.78
SEC.5	S5-2	P14	–	103	103	0.60	4.25	2.55	–	–	2.55	–	–	–	–	–*
PRIM.	P14	P13	50.8	–	183	1.06	4.16	4.42	0.02	0.06	4.48	0.007	15	1.04	18.4	0.86
SEC.4	S4-1	P13	39.4	124	124	0.72	4.21	3.03	0.02	0.02	3.05	0.0275	15	2.10	38.0	1.23
PRIM.	P13	P12	36.0	—	307	1.78	4.08	7.28	0.01	0.09	7.37	0.004	15	0.78	13.5	0.82
SEC.3	S3-3	P12	112.0	60	60	0.35	4.30	1.51	0.05	0.05	1.56	0.01	15	1.25	21.0	0.69
PRIM.	P12	P11	42.9	—	369	2.12	4.04	8.56	0.02	0.16	8.72	0.004	15	0.78	13.5	0.84
SEC.2	S2-3	P11	98.7	240	240	1.39	4.10	5.70	0.04	0.04	5.74	0.004	15	0.78	13.5	0.75
PRIM.	P11	P8	140.5	—	607	3.51	3.92	13.75	0.06	0.26	14.01	0.02	15	1.85	33.0	1.70
SEC.1	S1-1	P8	21.7	120	120	0.70	4.21	2.94	0.01	0.01	2.95	0.0156	15	1.58	28.0	0.98
PRIM.	P8	M.H.-T5	393.85	-	727	4.21	3.88	16.35	—	0.27	16.62	0.0078	15	1.10	19.5	1.30
Treatment Plant Sewers	T1	POND	35.50	-	—	-	–	–	-	-	8.31	–	15	-	-	–
	T2	POND	20.0	-	-	-	–	–	-	-	8.31	–	15	-	—	—

*. Kept for future design if needed.

Where p = population in thousands. Column 17 was found by using graphs given in (3, 5). It is noted that the actual velocity in none of the cases drops below the minimum flushing velocity of 0.61 m. per sec.

For simplicity, ground and invert elevations were not placed in Table 2; but instead are shown in Drawing 3.

Type of Embedment Required:

The following set of calculations are to show what type of embedment to use, and whether the pipe is enough to carry the applied load or not.

Drawing 3 indicates that a minimum invert depth of 50 cms is supplied. Also a critical sewer crossing a street with a depth of 120 cms is taken as a limit for concrete embedment or a gravel embedment as shown in details in Drawing 3.

Case: I: For a depth of 50 cms in gardens.

Procedures followed given by Steel (3).

$$B = \frac{6 \times 3}{2} + 12 = 21 \text{ inches} = 1.75 \text{ ft.}$$

$$d = \frac{50 - 15}{30.48} = 1.15 \text{ ft.}$$

$$\frac{d}{B} = \frac{1.15}{1.75} = 0.66$$

$$C = 0.61$$

$$W = (0.61)(130)(1.75)^2 = 236 \text{ lbs.}$$

Also assume a concentrated load of 600 lbs per ft is applied on the pipe at that section.

fo $\frac{d}{B} = 0.66$, factor $= 0.76$

Hence load = 0.76 x 600 = 456 lbs.

Total load = 456 + 236 = 629 lbs.

Asbestos cement pipes (6 inches internal diameter) can carry 1640 lbs. as required in the specifications.

Since 692 is smaller than 1640, then the sewer is safe.

<u>Case II</u>: When the depth is 1.20 m or more, and the sewer passes under the road.

B = 1.75 ft.

$$d = \frac{120 - 15}{30.48} = 3.45$$

$$\frac{d}{B} = \frac{3.45}{1.75} = 1.97$$

C = 1.60

$$W = (1.60)(130)(1.75)^2 = 620 \text{ lbs.}$$

An additional load (short load) is that of traffic. Assume a tractor weighing 10 tons passes over the road. Width of tractor is 6 feet. For critical cases, assume the tractor to be of 4 wheels, so each will exert a pressure of 2.5 tons or 5000 lbs. applied on 1 foot of sewer.

Ratio of depth to width = 1.97 as before,

K = 0.45

Hence the load = (0.45)(5000) = 2250

Total load = 2250 + 620 = 2870 lbs.

Pipe can carry 1640 lbs and by laying it on gravel, a load factor of 1.9 will result in a total crushing capacity of pipe to be 1640 x 1.9 = 3110 lbs.

Since 3110 is greater than 2600, then the sewer is safe.

Therefore for sewers crossing the road with an invert depth more than 1.20 m, are to be embedded in gravel as shown in Drawing 3, and if the depth is less than 1.20 m, the sewers must be embedded in concrete as shown on the same drawing.

Asbestos cement pipes are to be used as mentioned earlier because of the many advantages they possess over other pipes. Some of the advantages are mentioned in what follows:

a) Local made, not expensive, manufactured in a modern factory with a quality and dimensions in accordance with the International Standard Organization.

b) Sales office with technical service and a special team for installations of pipelines at the service of customers.

c) Reka coupling system - right joints, using sleeves and rubber rings, thus reducing infiltration.

d) Low coefficient of roughness, $n = 0.01$; thus allowing flatter grades and smaller pipe sizes.

e) Light weight that makes installation easier.

f) Length of pipe is 5 meters which reduce the number of joints and hence reduce the infiltration water. This is perhaps, the most important advantage, since the drinking water is obtained from wells.

In spite of the above advantages, asbestos cement pipes have a main disadvantage of being brittle. However, this disadvantage can be eliminated by proper handling of the pipes while installation takes place.

B. <u>Waste Stabilization Lagoons.</u>

The procedures followed in this design are based on Marais' principles for waste stabilization lagoons. The principles are developed in Appendix B. Reasons for the choice of this method are many, the most important one is that this method is simple, practical, and based on rational formulas.

The general data needed in the design is:

1. Sewage Flow (given on p. 6).
    a. Summer flow = 160 cu.m per day.
    b. Winter flow = 86 cu.m per day.
    c. September flow = 40 cu. m per day.
2. B.O.D. of sewage = 285 ppm (p.105).
3. Total rainfall in the month of August = 0 m.m. per day (p.101).
4. Total corrected value of evaporation in the month of August = 252 m.m. (p.101).

It should be noted that the infiltration water which amounts to about 23 cu. m per day (Table 2) is not considered in the design of the lagoons. The reason is that the amount of infiltration water is not a dependable source to the lagoon influent. So by considering such inflows, it would be possible to get non-favorable results in the lagoons. On the other hand, ignoring such inflows, will ascertain better results with respect to the quantity as well as the quality of the effluent in most general cases.

Assumptions:

1. Depth of pond to be 125 cms. (4 ft.).

2. Area of primary unit to be about 3000 sq. m.

3. Seepage flow into the earth will be $\frac{1}{4}$th of an inch = 6.4 m.m.(6). This value is the maximum permissible (6). As time goes on, the percolation will decrease, and hence less losses.

From the above values, it is noted that the B.O.D. in the pond will be:

$$P_o = \frac{750}{0.6d + 8} = \frac{750}{10.4} = 72 \text{ ppm.}$$

$$P = 285 \text{ ppm.}$$

$$R_1 = \frac{\frac{V}{Q_1}}{\frac{V}{Q_2}} = \frac{Q_2}{Q_1} = \frac{d_2}{d_1}$$

$$d_1 = \frac{160}{3000} \times 1000 = 53.3 \text{ m.m. per day.}$$

$$d_2 = d_1 - (d_{evap.} + d_{seep.} - d_{rainfall})$$

Design conditions are to follow critical conditions set by the month of August.

$$d \text{ evap.} = \frac{252}{31} = 8.1 \text{ m.m. per day.}$$

$$d \text{ rainfall} = 0 \text{ m.m. per day.}$$

$$d \text{ seepage} = 6.4 \text{ m.m. per day.}$$

Hence d inflow = 53.3 m.m.

and d outflow = 53.3 - 8.1 - 6.4 + 0 = 38.8 m.m.

$$\frac{R_1}{R_2} = \frac{d_2}{d_1} = \frac{38.8}{53.3} = \underline{0.73}$$

Using the formula developed by Marais.

$$P_o = \frac{P}{KR_1 + \frac{R_1}{R_2}}$$

$$72 = \frac{285}{0.17\ R + 0.73}$$

Attention at this point is to be made for the value of K used (K = 0.17). This value is that used by Marais in tropical climates. For Lebanon (A.U.B. Farm), this value is not known. It must be smaller so as to give a longer detention period, and thus a smaller loading per unit area. In the design, this value of 0.17 is used in hope that the proper value can be estimated later on from operational results. Care will be considered in relation to this point in the design.

So $72 \times 0.17 \times R = 285 - 72 \times 0.73 = 285 - 53 = 232$

Hence $R = \dfrac{232}{72 \times 0.17} = 19.0$ days.

According to Marais's method, for an influent B.O.D. concentration of 285 ppm, and a depth of four feet, a minimum retention time of 17.5 days is required. Since 19.0 is greater than 17.5, the value is safe. Also a retention of 19.0 days will give a B.O.D. loading of:

$$\frac{285 \times 160 \times 1000}{1000{,}000} \times 2.204 = 100 \text{ lbs.}$$

$$\text{Area} = \frac{19.0 \times 160}{1.25} = 2438 \text{ sq. m.}$$

$$\text{So a loading} = \frac{100}{\frac{2438}{4047}} = 166 \text{ lbs. of B.O.D. per acre per day.}$$

Maximum allowable value given by Marais as shown in Fig. B_4, is 175 ppm. Again 166 ppm is less than 175 ppm, so acceptable.

It seems a value of 19.0 days retention is good, however, two ponds each with a capacity of 14 days retention will be constructed. Thus totalling to a 28 days retention period. Operation must be in parallel. Reasons for this last assumption are discussed in the following paragraph.

If the first two ponds designed to operate in parallel are operated in series, the first pond will have a retention time of only 14 days, and a B.O.D. loading of 226 lbs. per acre per day. This is not acceptable according to Marais's method that requires a minimum retention of 17.5 days and a B.O.D. loading of 175 lbs. per acre per day for an influent B.O.D. of 285 ppm.

In spite of the above fact, series operation is permissible later on if more exact value will be found to prove the validity of series operation.

Effluents from the ponds should be only used to irrigate trees. For trees secondary units are enough, while for vegetables, tertiary units must be added.

For the secondary unit, a retention of seven days is necessary.

Based on an inflow of 160 cu.m. per day, the volume of each of the primary ponds* is to be:

$$14 \times 160 = 2240 \text{ cu. m.}$$

For a depth of 1.25 m. the area will be equal to:

$$\frac{2240}{1.25} = 1790 \text{ sq. m.}$$

Rectangular areas give a length of 60 m, and a width of 30 m.

Area provided = 60 x 30 = 1800 sq. m.

For the secondary pond, with retention of seven days, Area will be half as much, since the depth is kept constant.

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* Present value of water consumption is used, having in mind that any future expansion will be solved by a similar set of lagoons shown in dotted lines on Drawings 1 and 2.

Thus dimensions of pond will be 30 x 30 m.

For the water surface area, each of the above dimensions is to be increased by 6.45 m. This is explained by the following, and a look at Drawing 2 will facilitate the explanation.

- Slope of the dikes to be followed is 3 horizontal to 1 vertical.
- Freeboard is 0.9 m.
- Depth of water is 1.25 m.
- Width of enbankment is to be 2.50 m. everywhere, except where the sewer line passes where it becomes 3.0 m.

Hence the total height of the dike will be 2.15 m, and the horizontal distance will be 3 x 2.15 = 6.45 m.

Surface Area of primary lagoon to give the proper retention time will be equal to:

66.45 x 36.45 = 2420 sq. m.

The two units = 2 x 2420 = 4840 sq. m.

Assumed value was 3000 sq. m.

In spite of this difference, no changes in the design will take place, as shown down below:

$$d_1 = \frac{160}{4840} \times 1000 = 33.1 \text{ m.m. per day.}$$

Loss = 14.5 m.m. per day, so $d_2 = 33.1 - 14.5 = 18.6$ m.m.

and $\frac{R_1}{R_2} = \frac{d_1}{d_2} = \frac{18.6}{33.1} = 0.56$

Using the formula developed earlier:

$$R = \frac{285 - 0.56 \times 72}{0.17 \times 72} = \frac{245}{0.17 \times 72} = 20 \text{ days.}$$

20 days are less than 28 days, so it is still safe, and the value is not changed.

Surface area of the secondary lagoon = 36.45 x 36.45 = 1330 sq.m.

Hence the total water surface area of the whole pond = 6170 M.S.

The following flow balance (Table 3) is derived from inflow, evaporation, rainfall and seepage rates:

Inflow:

Summer = $\frac{160 \times 1000}{6170}$ = 26.0 mm per day.

Winter = $\frac{86 \times 1000}{6170}$ = 13.90 mm per day.

September = $\frac{40 \times 1000}{6170}$ = 6.5 mm per day.

(Total Loss) Example of August

= d evap. + d seep. - d rainfall

= 8.1 + 6.4 - 0 = 14.5 m.m. per day.

Table 3 - Flow Balance

Month	Inflow mm per day	Total Loss mm per day	OUTFLOW mm per day	OUTFLOW m.c.per day
January	13.90	3.80	10.10	62.30
February	13.90	5.10	8.80	54.30
March	13.90	7.50	6.40	39.50
April	13.90	9.45	4.45	27.40
May	13.90	11.15	2.75	16.90
June	13.90	13.40	0.50	3.08
July	26.00	14.30	11.70	72.20
August	26.00	14.50	11.50	71.00
September	6.50	12.80	- 6.30	- 38.80
October	13.90	10.70	3.20	19.80
November	13.90	7.35	6.55	40.40
December	13.90	4.40	9.50	58.60

It is shown on the flow balance sheet Table 3 that the maximum effluent which could be used for irrigation is that of the month of July - 72.20 cu.m. As time goes on, the pores in the soil of the lagoons will be clogged and infiltration water will decrease, thus a larger effluent would be expected. Outflows during the winter season are not needed for irrigation, and are to be disposed of in a suitable manner discussed briefly as follows.

With regard to winter conditions, some of the effluent may be kept in the pond by increasing the depth to a maximum of 1.50 m. The excess may be disposed by the two following methods.

a. Effluent may be spread over the land by passing it in the irrigation channels used to irrigate the land in summer.

b. Dispose the effluent into the neighbouring canal (Canal Tel Chassil).

Any other method found suitable might be used. In no case it is to be indiscriminately disposed of on the ground, because the water might erode the soil.

The field to be irrigated by effluents is shown on drawing No. 1, and is bounded with red color.

Assuming irrigation is run daily, since the field is very big about 30 acres in area, and taking a safety factor of two, the volume of the irrigation pool will be:

$2 \times 72.20 = 144.40$ cu.m.

Assume a depth of 2 m, and a freeboard of 0.50 m.

The surface area will be:

$$\frac{144.40}{2} = 72.20 \text{ sq. m.}$$

For a width of 6 m, the length will be about 12 m.

$6 \times 12 = 72.0$ sq. m.

Further looking at the flow balance, Table 3 it is noted that trouble comes only in September, when the students leave the farm, and the flow will be a minimum and less than the losses. To overcome this, many solutions are possible. Two such solutions are:

1. In August accumulate more volume and thus raise the depth; $\frac{6.3 \times 30}{10} = 19$ cms. Thus the total depth of water = $1.25 + 0.19 = 1.44$ m. This will leave a freeboard of $0.90 - 0.19 = 0.71$ cms that is still acceptable for small lagoons. This method will decrease the irrigation water originated from the outflow by about 39 cu.m. per day.
2. Leave the operation of the pond in August to a depth of 1.25 m, and in September the losses will decrease the height to $1.25 - 0.19 = 1.06$ m, which is still acceptable (greater than 3 feet). This lowered water level will then increase in winter to the original designed depth of 1.25 m.

Which method to use is left for the operator to choose. Other methods might be more valuable if irrigation water is needed. Storing of water in the pond by increasing the depth up to 1.50 m, may be done to increase the irrigation water for use in summer.

C. Auxiliary Features:

1. Parshal Flume:

This is to be ahead of the grit chamber since it controls a fixed velocity for any range of discharge. The design followed here is according to the procedures stated in (7).

Maximum flow = 16.62 liters per sec.

Average flow = 4.48 liters per sec.

Minimum flow = half the average (assume) = 2.24 liters per sec.

Since most of the factors are approximated, use a safety factor of 1.5 for the maximum flow so:

Maximum flow = 16.62 x 1.5 = 24.93 liters per sec.

24.93 = 0.88 c.f.s.

and 2.24 = 0.08 c.f.s.

Hence design a flume that can measure a range of flow between 0.08 c.f.s. and 0.88 c.f.s.

Assume that for maximum flow, the depth of water in the ditch is 0.65 ft., and the freeboard is 4 inches. Allow for 3 inches friction loss.

Flumes with a throat width of 6 and 9 inches satisfy the given range of flow.

Throat Width	Ha	0.40 Ha
6 inches	0.585	0.235
9 inches	0.445	0.178

The flume with width 6 inches is the smallest flume to satisfy the given conditions.

Hence the required depth upstream will be = 0.235 + 0.650 = 0.885

Crest of the flume should be set at

0.885 - 0.585 = 0.300 ft. above the bottom of the ditch.

More complete details are shown on the drawings, and values are changed to cms.

2. Grit Chamber:

According to Steel (3), for every inch of depth, one foot length of grit channel is to be used. In this particular case, the depth is 0.885 or 10.5 inches. So the minimum length of grit channel is 10.5 feet = 3.20 m, use a channel 4 m. in length.

Width of channel, assuming all flow to be in one chamber at a time.

Velocity = 30 cms per second

Discharge = 24930 cm^3 per sec.

Depth = 0.885 x 30.48 = 27 cms.

$\frac{24930}{30x27}$ = 30.8 cm.

Use a width of 30 cms.

In the channel after the grit chamber, the width is 40 cms, and the velocity in it is $\frac{24930}{40x27}$ = 23 cm per second.

Make two channels, and use each one at a time. Supply gates as shown on the drawings.

3. Screens:

They are placed at the entrance of each grit chamber. Width of section is to be reduced to 20 cms, thus making a velocity of about 45 cms per second. Actual velocity is little more than this because of the reduced section of channel due to the iron bars. Thus the velocity approaches the required of 60 cms per second.

Assuming a friction loss of 3 cms head in the screens, the depth in the channel after manhole T_4 and before the grit chamber will be 30 cms.

Screens are to be inclined to the horizontal at an angle of 45°. The screenings are to be ground, dumped or burnt with the refuse. Cleaning of the screens is to be done manually as necessary.

4. Channel from Manhole T_4 to Grit Chamber:

Depth = 30 cms.

Width = 20 cms.

Flow = 24.93 liters per second.

Velocity = $\frac{24930}{30 \times 20}$ = 42 cms per second.

This velocity is rather low, to secure a higher velocity of at least 0.60 m per second, the channel must slope down.

Hydraulic Radius = $\frac{A}{P}$ = 0.280

and n for concrete = 0.013

So slope = 0.0015, say 2 per thousand.

5. Manholes and Distribution Box:

Complete details are shown on the drawings. For MH.T_1' and MH.T_2, an overflow channel to the pond is designed in case of any clogging of the influent pipes.

6. Effluent Pipes:

Since the pond is to be operated at different depths for reasons mentioned earlier, peak flows take place when going from a high level to a lower one, by removing planks designed in Section 7. Maximum value of flow will be 71.5 liters per second as calculated in the next section.

A pipe section of 25 cms, with an area of 493 sq.cm. will give a velocity of $\frac{71,500}{493}$ = 146 cms per second.

This is an acceptable velocity. However the velocity in this pipe at normal conditions will be very small = 2 cms per second as calculated down below.

Assume uniform flow of 80 cu.m. per day

So $\frac{80 \times 1000}{86,400} = 0.92$ liters/sec.

For a diameter of 25 cms, the area is 493 cm^2

So the velocity $= \frac{920}{493} \cong 2$ cms/sec.

For this reason, a smaller pipe diameter should be installed at the expense of higher velocity.

A pipe of 15 cms internal diameter, with area $\cong$ 177 cm^2.

Velocity will be $\frac{920}{177} = 5.2$ cm/sec.

This is still small, but since the water flowing is practically clean, no troubles should take place due to clogging. In any case, the pipe is given a small slope.

For maximum conditions:

$$V = \frac{71,500}{177} = 404 = 4.04 \text{ m/sec.}$$

7. Outlet and Planks:

Planks are to act as gates with a variable height. Pond is designed to operate at four different depths. Mainly 1.50 m, 1.25 m, 1.05 m and 0.90 m (not more than 1.50 m and not less than 0.90 m). Normal operation designed for is 1.25 m.

The maximum drop in elevation is from 1.50 to 1.25 and is equal to 0.25 m. For this height, and for a weir length of 30 cms, the maximum discharge will be equal to:

$$Q = \frac{2}{3} \times 0.65 \quad 2 \times 9.8 \times 0.30(0.25)^{3/2}$$; assuming no velocity

of approach and $C_d = 0.65$,

$Q = 0.0715$ cu.m. per sec. = 71.5 liters per sec.

As found before, a pipe diameter of 15 cms should be used.

8. <u>Friction Losses:</u>

To calculate the friction losses to see if any reasonable drop in elevation between the ponds is found.

a) Pond:

Equivalent length is 125 m. (8)

Velocity is = 5.2 cms per second.

Assume maximum friction factor of 0.03 for cast iron pipes.

$$h_f = f \frac{L}{d} \frac{V^2}{2g}$$

$$h_f = \frac{0.03 \times 12500 \times (5.2)^2}{15 \times 2 \times 980}$$

3.3 mm - negligible.

The velocity of the pipe is very small, 5.2 cms per second. Although the pipe is under pressure, it is to be sloped to avoid any settling - see drawings.

b) $MH.T_8$ - Irrigation Pool:

i) Loss of friction in the pipe is negligible under normal conditions about 0.2 mm.

ii) Loss over the weir

$Q = 2 \times 0.92 = 1.84$ liters per second.

$L = 30$ cms.

$$\frac{0.00184 \times 3}{0.65 \times 2 \times \quad 19.6 \times 0.30} = h^{3/2} = 0.002$$

$h = (0.002)^{2/3} = 0.021$ m. = 2.1 cms.

Allow a free surface between the crest of the weir and the water level of 6 cms, then the level of the water in the pool will be $2.1 + 6.0 = 8.1 \approx 8$ cms less than the operated depth of the pond, without the flume, placed at the outlet. In the flume, the drop will be equal to about 4 cms, average and 7 cms maximum. Thus totalling to a 15 cms maximum drop in pool elevation.

9. <u>Drainage Channel:</u>

Assume maximum conditions of height, and no friction loss through the gate valve. Discharge = V x A

$$V = 2gh = 2 \times 9.8 \times 1.5 = 5.43 \text{ m/sec.}$$

$A = 177 \text{ cm}^2$ for a pipe of 15 cms diameter.

Discharge = $177 \times 543 = 96{,}000 \text{ cm}^3$ per sec.

Assume a velocity of 100 cms per second in the channel, with a width of 40 cms.

$$\text{The height will be } \frac{96{,}000}{100 \times 40} = 24 \text{ cms.}$$

With a freeboard of 16 cms, the total height will be 40 cms.

Reinforcement is to be made at the outlet of each pipe for a length of 2 meters by 14 $^{m}/_{m}$ at 20 cms c.c.

CHAPTER III

SPECIFICATIONS (STATEMENT OF UNIT PRICES)

Item No.	D e s c r i p t i o n	Cost L. L.
1a	15 cms. (6-in.). Asbestos Cement Pipes, at invert depths less than two meters as shown on the drawings. Asbestos cement pipes must conform with the following specifications: - Wall thickness shall conform to recognized standards specified for the manufacture of asbestos cement pipes. - Average crushing strength by the sand bearing method shall not be less than 2450 kgs. per linear meter. - The pipe shall withstand an average internal hydrostatic pressure of 6 atmospheres. - Length of pipes supplied shall be 5 meters. This item includes all excavation, backfilling (as given in Item 7), carting away and furnishing and installing in place 15 cms asbestos cement pipes as specified. It includes all sheeting, shoring, disposal of water, and placing up to 20 cms. layers of fine gravel underneath the pipe, that is to be well compacted in place so as to provide a firm bearing for the pipe. It should be made of gravel or crushed stone, that is well graded between 6 m.m. and 20 m.m. in size, or as directed by the engineer.	

Item No.	D e s c r i p t i o n	Cost L.L.
1a.	Payment will be made for the number of linear meters actually installed, and depths will be computed as the average between manholes. Lengths of sewers will be measured horizontally along the center line with no deduction for manholes. The unit prices bid per linear meter under this item, shall include all labour, materials and equipment necessary to furnish and install the sewer as shown on the drawings and as specified. Cost per linear meter Lebanese Pounds	
1b.	15 cms. (6-in.) Asbestos Cement Pipes, at invert depths less than two meters as shown on the drawings and as specified in Item 1a, except for the type of embedment which is to be in concrete as shown in the drawings. Full cost of placing the steel reinforcement in the concrete embedment is included in the unit prices bid. Cost per linear meter Lebanese Pounds	
2a.	15 cms. (6-in.) Cast Iron Pipes, at invert depths less than two meters as shown on the drawings. Cast iron pipes to be installed are to be of the centrifugally cast type of approved manufacture.	

Item No.	D e s c r i p t i o n	Cost L.L.
2a.	This item includes all excavation backfilling (as given in item 7), carting away, and furnishing and installing in place 15 cms. cast iron pipes as specified. It includes all sheeting, shoring, disposal of water, using of flanged joints. Selected material of the same soil is to be used. Bends and clean outs as shown on the drawings are to be included in the unit cost. Payment will be made for the number of linear meters actually installed and as given in Item 2a. The unit prices bid per linear meter under Item 2a, shall include all labour, materials, and equipment necessary to furnish and install the sewer as shown and specified. Cost per linear meter Lebanese Pounds	
2b.	15 cms, (6-in.) Cast Iron Pipes, at invert depth less than two meters as shown on the drawings, and as specified in Item 2a, except for the type of embedment which is to be of a channel type between MH-T3 and D.B. shown on the drawings. Cost of constructing the channel and the precast covers is included in the unit prices bid under this Item. Cost per linear meter Lebanese Pounds	

Item No.	D E S C R I P T I O N	Cost L.L.
2c.	15 cms. (6-in.) Cast Iron Pipes, at invert depth more than two meters and less than 2.50 meters as shown on the drawings and as specified in Item 2a, except for the invert depth. Cost per linear meter Lebanese Pounds	
3a.	Manholes at depths less than one meter as shown on the drawings and constructed of plain concrete or precast concrete blocks with poured concrete base, and shall have cast iron frames, covers and steps, with plastered walls, all as specified down below: 1. Plain concrete or concrete blocks. Plain concrete used must contain 250 kgs. of cement per m.c. and to be of a minimum thickness of 15 cms. Cement shall conform with Portland Cement as manufactured in Chekka, Lebanon. Sand shall be clean and well granulated and free of earth and all other foreign materials. The sand used shall pass a 5 m.m. mesh, but in no case be smaller than 0.5 m.m. Gravel used shall pass a 30 m.m. mesh and be held on a 5 m.m. mesh and shall be sufficiently clean for use as concrete aggregate. Concrete blocks might be used in lieu of the plain concrete, and be made of a mix containing 200 kgs. of	

Item No.	D e s c r i p t i o n	Cost L. L.
3a.	cement per m.c. All blocks shall be solid and be curved to the desired radius from the outside and inside. The 28 days compressive strength shall not be less than 3000 p.s.i. (210 kgs. per square cm.). Mortar used must be composed of 1 part of cement to two parts of approved clean sand by volume. The elevation of the top of frame shall be as the elevation of the finished ground. 2. Manhole covers. Should be made of cast iron and as specified and shown on the drawings. 3. Manhole Steps. Shall be made of best merchantable gray cast iron, tought, even-grained and free from all flaws and injuries or unsightly defects. 4. Concrete Bases. Shall be made in accordance with the specification for plain concrete given in Item 3a(1). 5. Plastering. Plastering of the internal and external walls of the manholes is to be done neatly with mortar made up 1 part of cement to 2 parts of fine approved sand to a thickness of 1 to cms. This Item (3a) includes the construction of manholes, including excavation and backfill, concrete bases, manhole steps, cast iron frames, and plastering of manhole walls from the outside and inside. Unit prices bid under	

Item No.	D e s c r i p t i o n	Cost L.L.
3a.	this Item shall include furnishing labour, materials, and equipment necessary to construct manholes as shown and specified. Payment will be done per number of manholes constructed. The unit prices bid under Item 3a shall not include, however, the cost of pipe which is specified and included for payment under other items. Cost per one manhole Lebanese Pounds	
3b.	Manholes at depths more than one meter and less than two meters as shown on the drawings and specified in Item 3a with the exception of the depth. Cost per one manhole Lebanese Pounds	
3c.	Manhole T_3, at depth less than one meter as shown on the drawings and specified in Item 3a, except for the dimensions and manhole frame and cover which is to be 60 x 60 cms. and a total weight of 24.5 kgs. Cost per one manhole Lebanese Pounds	

Item No.	D e s c r i p t i o n	Cost L. L.
3d.	Distribution Box, at depth less than one meter, as shown on the drawings and specified in Item 3a, except for the dimensions and manhole frame and cover that is specified under Item 3c. The unit prices bid under Item 3d shall not include, however, the cost of the steel gates. Cost per one manhole Lebanese Pounds	
4a.	Steel Control Gates, as shown and specified on the drawings, with 25 x 25 x 0.2 cm. The unit prices bid shall include labour, materials, and equipment necessary to furnish and install the gates in place as shown and specified. Cost per one gate Lebanese Pounds	
4b.	Steel Control Gate, as shown and specified on the drawings and as given by Item 4a, except for the dimensions which are: 40 x 25 x 0.2 cm. Cost per one gate Lebanese Pounds	

Item No.	D e s c r i p t i o n	Cost L.L.
4c.	Steel Control Gate, as shown and specified on the drawings and as given by Item 4a, except for the dimensions which are: 40 x 35 x 0.2 cms. Cost per one gate Lebanese Pounds =	
4d.	Steel Control Gate, as shown and specified on the drawings and as given by Item 4a, except for the dimensions which are: 40 x 55 x 0.2 cms. Cost per one gate Lebanese Pounds	
4e.	Steel Control Gate, as shown and specified on the drawings and as given by Item 4a, except for the dimensions which are: 40 x 80 x 0.2 cms. Cost per one gate Lebanese Pounds	
5a.	Fence with a height of two meters from the ground surface as shown on the drawings and specified in what follows: 1. Steel angles to be made of 2 x 2 x 3/16 inch (5 x 5 x 0.5 cms.) and be of best quality. 2. Wire is to be barbed wire. 3. Plain concrete bases, with concrete as specified in Item 3a(1).	

Item No.	D e s c r i p t i o n	Cost L.L.
5a.	The unit prices bid shall include all labour, materials, equipment, excavation, and concrete, necessary to furnish a complete job in accordance with the drawings and specifications. Payment will be made for the number of linear meters placed. Cost per one linear meter Lebanese Pounds	
5b.	Fence Gate, with a height of two meters from the ground surface as shown on the drawings. The unit prices bid shall include all labour, materials including a lock of good quality, equipment, excavation, and reinforced concrete with reinforcing steel necessary to furnish a complete job. Payment will be made on number of gates constructed. Cost per one gate Lebanese Pounds	
6	Screens, as shown on the drawings, and to be made of iron rectangular bars 1 x 5 cm. clear spacing between the bars is to be 2 cms, and the screens supported only at the extreme ends. The unit prices bid shall include all labour, materials, and equipment necessary to furnish a complete job.	

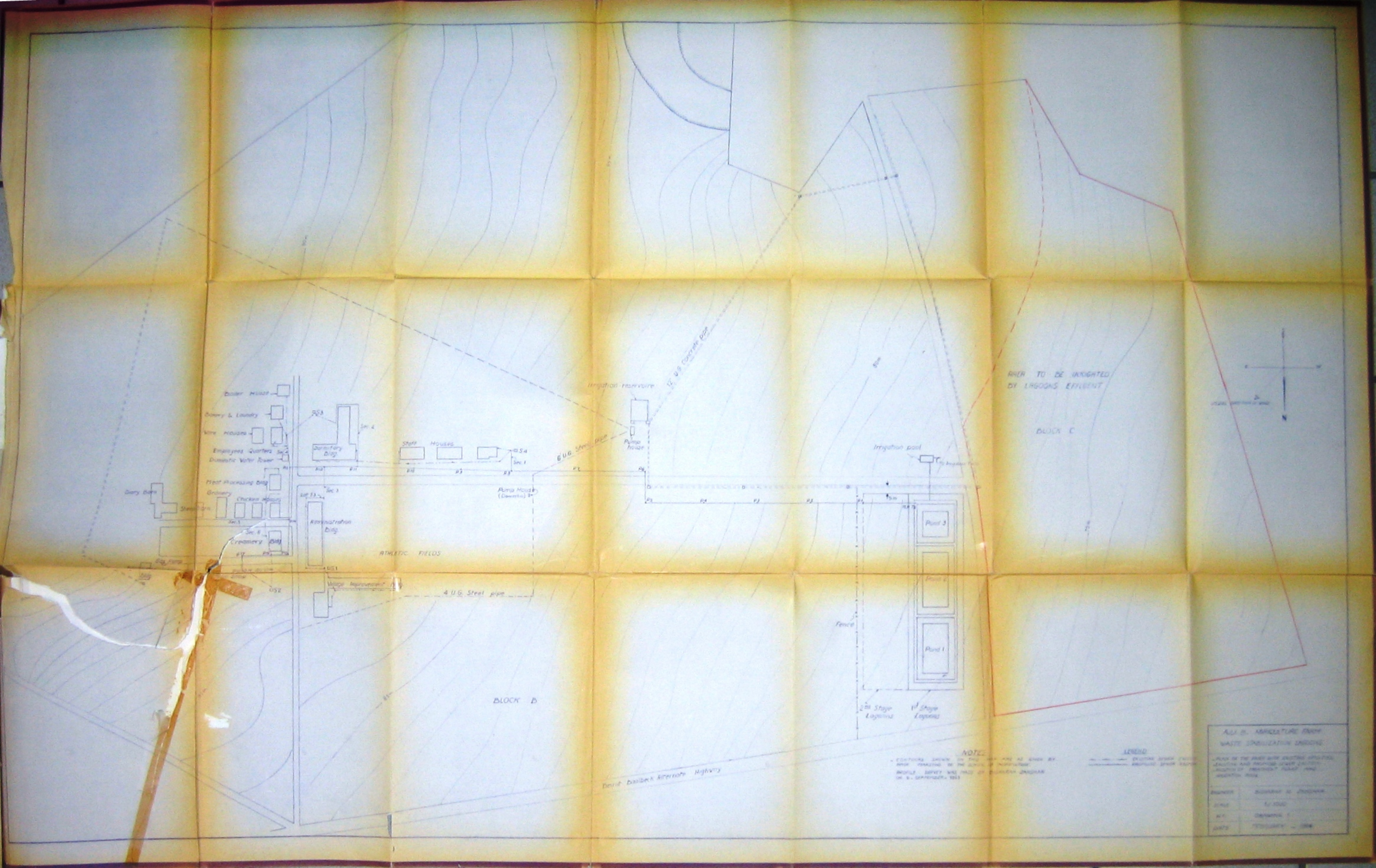
Item No.	D e s c r i p t i o n	Cost L.L.
6	Payment will be made on number of screens constructed as shown and specified. Cost per one screen Lebanese Pounds	
7	Extra Excavation, Enbankment, and Backfill. Item 7 includes all earth excavation, enbankment, and backfill outside the limits required for the installation of the facilities mentioned under items 1, 2, 3, 5 and 6, which are necessary for the area of the lagoons and the road leading to it. The work done under this Item shall include excavation and backfilling an equal volume, or disposing of the extra material in case excavation is more than filling. Also it includes sheeting and shoring, dewatering and protection of existing structures. Backfilling shall be done as soon as possible after the different structures are installed. Selected well graded material, free from large stone (30 cms or above) or lumps shall be used for backfilling. Selected material shall be placed in layers not exceeding 20 cms (8 in.) in thickness and shall be thoroughly compacted by tamping for the trenches, and by a sheeps foot roller for the pond and embankment. Compaction is to be done in accordance with the AASHO specification. Acceptable degree of compaction is 95%.	

Item No.	D e s c r i p t i o n	Cost L.L.
7	Payment will be made for the number of cubic meters of excavation as provided for in the detailed drawings, and as specified. The unit prices bid shall include all work mentioned under this item. Cost per one cubic meter Lebanese Pounds	
8	Extra Backfilling. Under this Item all backfilling in excess of excavation mentioned under Item 7 is considered. Item 8 includes bringing filling material from neighbouring areas, and compacting them as specified in Item 7.	
8	Payment will be made for the number of cubic meters needed as found from the drawings. The unit price shall include all work mentioned under this Item. Cost per one cubic meter Lebanese Pounds	
9a.	Additional Concrete. Item 9a includes all plain concrete outside the limits required for the installation of the facilities mentioned in some of the preceding Items. It is necessary for the concrete mats in the lagoon area. As shown on the drawings and specified in Item 3a(1). The concrete mats are 10 cms. thick.	

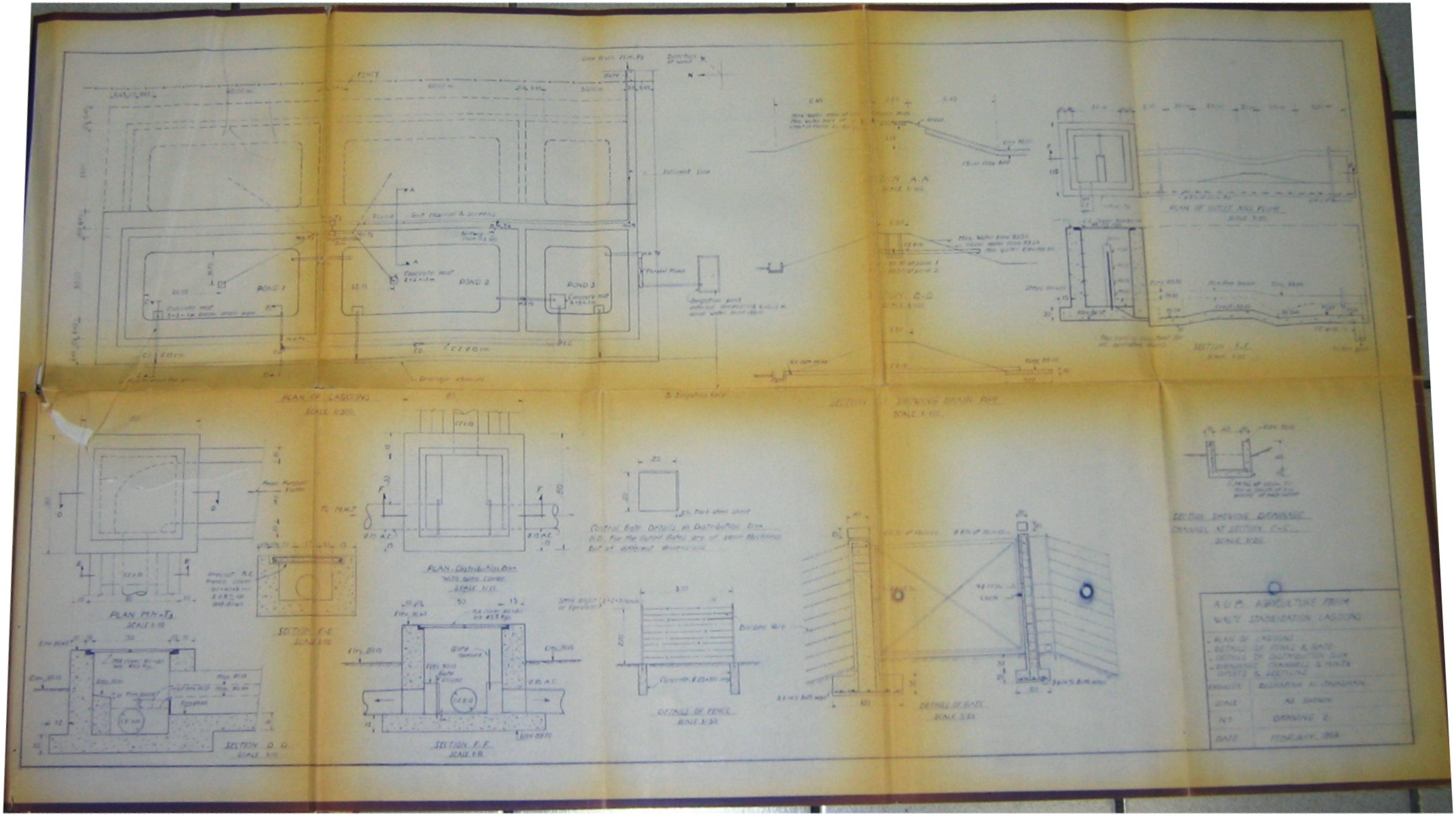
Item No.	D e s c r i p t i o n	Cost L. L.
9a.	Work under this Item, includes excavation, cart away, and pouring the concrete to a thickness of 10 cms. The unit prices bid shall include all labour, materials and work as specified above. Payment will be made on the number of square meters finished. Cost of one square meter Lebanese Pounds	
9b.	<u>Additional Concrete</u>, as shown on the drawings and specified in Item 9a except for the type of use. Plain concrete under Item 9b is for the construction of the various concrete channels shown on the drawings. Work under this Item includes excavation, cartaway, and pouring the concrete in the required sections. Formwork is included in the unit price also. Payment is done on the number of cubic meters constructed. Steel reinforcement used at some sections of the channels is considered separately. Cost per one cubic meter Lebanese Pounds	
10	<u>Additional Reinforcing Steel</u>, as shown on the drawings and as will be ordered by writing by the engineer; to be placed in addition to that considered in other Items. Reinforcing steel shall be intermediate grade bars and shall be thoroughly cleaned and no reinforcement having	

Item No.	D e s c r i p t i o n	Cost L.L.
10	thick rust or scale shall be used. Work under this Item will include furnishing, bending, and placing the steel in position as shown on the drawings. Cost per one kilogram Lebanese Pounds	
11	<u>Clay Bed</u>. A clay bed of 15 cms. is to be placed and compacted on the lagoon's bottom. Bentonite clay or an equivalent water proof material might be used later if necessary and asked by written from the engineer. Puddled clay is to be used. Work includes furnishing, placing compacting of 15 cms. clay as specified above. Bentonite lining or other equivalent material will be paid separately if needed. Payment under this Item is done per square meter of finished surface. Cost of one square meter Lebanese Pounds	
12a.	<u>Cast Iron Gate Valves</u>. Gate valves are to be of cast iron of approved manufacture. The unit prices bid shall include furnishing and installing in place cast iron gate valves for pipe diameter of 15 cms (6 in.).	

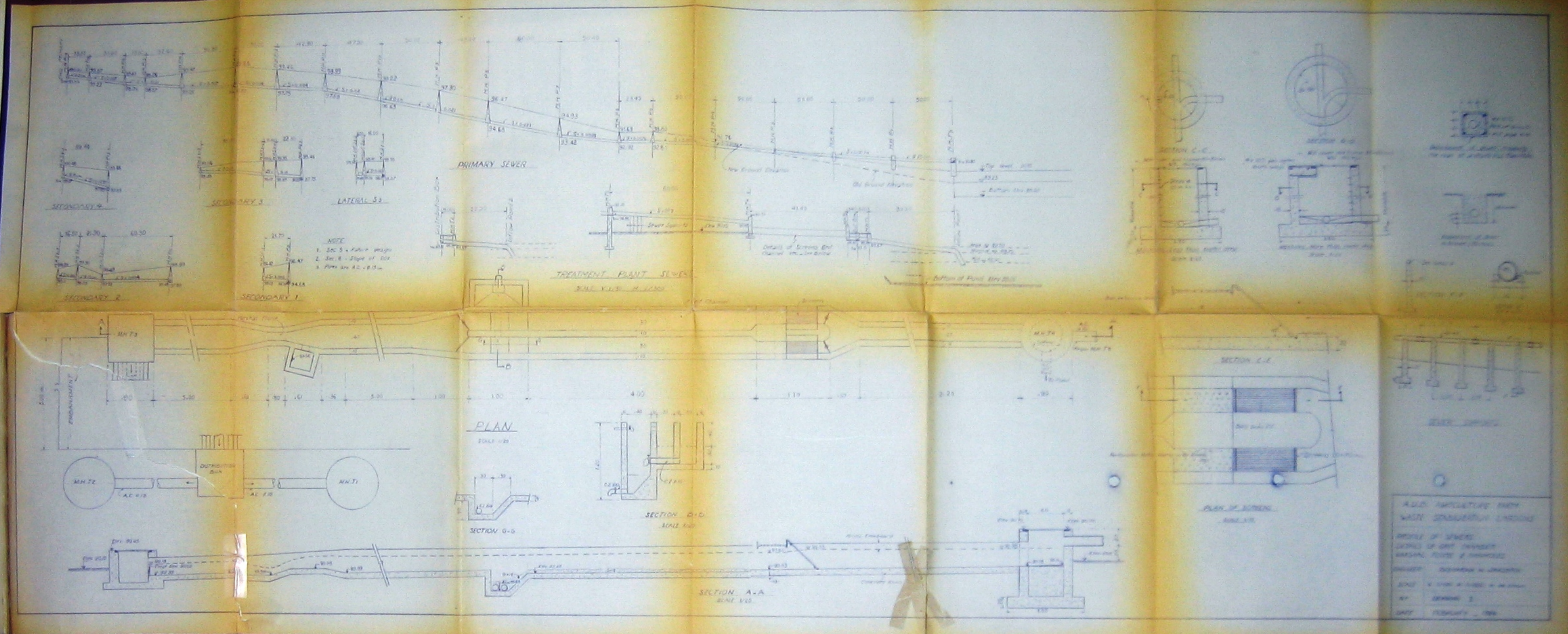
Item No.	D e s c r i p t i o n	Cost L. L.
12a.	Payment will be made for the number of gate valves finished. Cost per one gate valve Lebanese Pounds	
12b.	<u>Cast Iron Gate Valves.</u> As specified in Item 12a, with the exception of the pipe dimension which is to be 10 cms; but in this case the pipe is to be included in the unit prices bid. Cost per one gate valve Lebanese Pounds	



Boiler House
Bakery & Laundry
Employees Quarters
Domestic Water Tower
Meat Processing Bldg
Dairy Barn
Chicken Houses
Creamery Bldg
Administration Bldg
ATHLETIC FIELDS
Dormitory Bldg
Staff Houses
Irrigation reservoire
Pump house
6" U.G. Steel pipe
4" U.G. Steel pipe
Irrigation pool
AREA TO BE IRRIGATED BY LAGOONS EFFLUENT
BLOCK C
BLOCK D
Pond 3
Pond 1
Fence
2nd Stage Lagoons
1st Stage Lagoons
Beirut Baalbeck Alternate Highway
NOTES
LEGEND



PLAN OF LAGOONS
POND 1
POND 2
POND 3
FENCE
Drainage channel
SECTION F-F
DETAILS OF FENCE
DETAILS OF GATE
SECTION SHOWING DRAINAGE CHANNEL AT SECTION C-C
A.U.B. AGRICULTURE FARM
WASTE STABILIZATION LAGOONS
SCALE
AS SHOWN
DRAWING 2
DATE


PRIMARY SEWER
SECONDARY 4
SECONDARY 3
SECONDARY 2
SECONDARY 1
NOTE
TREATMENT PLANT SEWER
PLAN
SECTION G-G
SECTION D-D
SECTION A-A
SECTION C-C
SECTION E-E
PLAN OF SCREENS
SEWER SUPPORTS
DISTRIBUTION BOX
M.H.T1
M.H.T2
EMBANKMENT
A.U.B. AGRICULTURE FARM
WASTE STABILIZATION LAGOONS

CHAPTER IV

DRAWINGS

CHAPTER V

ESTIMATED COST

The following table (Schedule of Prices) shows the different items as specified in Chapter III, with the estimated quantities, the unit prices, and the total estimated cost of the different items.

The quantities were based on dimensions shown on the drawings and/or as specified.

The unit price was based on local prices given by different manufacturers in Beirut, such as Kassarjian for cast iron pipes, and manhole covers; the Eternit Company for the asbestos cement pipes, and other agents and manufacturers for steel, cement, and other necessary materials. The unit price was also based on the specifications given in Chapter III, which specifies the kind of material, as well as conditions on site and quality of work.

The cost shown in the following table has been based on a daily sewage flow of 160 cu.m. and a concentration of sewage equal to 285 ppm, of 5 day B.O.D. That is, an equivalent population of 560 persons.

The sum of L.L. 42,517.83 is the total estimated cost of the whole scheme. That is the cost of construction of the lagoons and that of construction of the proposed sewer system that carries the sewage from the different units of the farm to the lagoon area. For comparison purposes it is necessary to separate the different items in each branch of the scheme.

If only the items directly involved in providing the lagoon as shown in the table (Schedule of Prices) are considered, the cost of construction of the waste stabilization lagoons amounts to L.L. 28,849.61.

So it is concluded that cost of construction of waste stabilization lagoons to treat sewage produced by the equivalent 560 persons in the farm amounts to about L.L. 29 thousand. Also it is obvious that any figure that differs from 560 equivalent will result in a different total cost of construction.

SCHEDULE OF PRICES

Item No.	Description	Estimated Quantities	Unit	Unit Price L.L.	Total L.L.
1a	15 cms. (6-in) Asbestos Cement pipes; depth less than 2 meters.				
	1. Network of sewers	1100.85	m	8.15	9357.22*
	2. Lagoons	55.50	m	8.15	471.75
1b	15 cms (6-in) Asbestos Cement pipes; depth less than 2 meters.	26.00	m	16.00	416.00*
2a	15 cms. (6-in) Cast Iron pipes; depth less than 2 meters.	175.00	m	20.00	3500.00
2b	15 cms. (6-in) Cast Iron pipes; depth less than 2 meters.	3.00	m	31.00	93.00
2c	15 cms. (6-in) Cast Iron pipes; depth more than 2 meters, and less than 2.50 meters.	45.00	m	22.50	1012.50
3a	Manholes; depth less than one meter.				
	1. Network of sewers	11.00	each	125.00	1375.00*
	2. Lagoons	6	each	125.00	750.00
3b	Manholes; depth more than one meter and less than two meters.	14	each	180.00	2520.00*
3c	Manhole-T3; less than one meter.	1	each	30.00	30.00
3d	Distribution Box; depth less than one meter.	1	each	30.00	30.00
4a	Steel Control Gate. 25 x 25 x 0.2 cms.	2	each	1.50	3.00
4b	Steel Control Gate. 40 x 25 x 0.2 cms.	1	each	2.25	2.25
4c	Steel Control Gate. 40 x 35 x 0.2 cms.	1	each	3.25	3.25

* Items directly involved in the network of sewers.

Item No.	Description	Estimated Quantities	Unit	Unit Price L. L.	Total L. L.
4d	Steel Control Gate. 40 x 55 x 0.2 cms.	1	each	5.00	5.00
4e	Steel Control Gate. 40 x 80 x 0.2 cms.	1	each	8.00	8.00
5a	Fence; 2 meters high.	240.00	m	4.25	1020.00
5b	Fence Gate, 2 meters high with a lock.	1	each	200.00	200.00
6	Screens.	2	each	20.00	40.00
7	Extra Excavation-Embankment and Backfill.	5400.00	cu.m.	1.50	8100.00
8	Extra Backfilling.	5245.00	cu.m.	1.50	7867.50
9a	Additional Concrete; Mats 10 cms. thick.	60.00	sq.m.	6.00	360.00
9b	Additional Concrete; for Channels.	34.90	cu.m.	45.00	1570.50
10	Additional Reinforcing Steel.	121.20	kgs.	0.30	36.36
11	Clay Bed; 15 cms deep.	5093.00	sq.m.	0.50	2546.50
12a	Cast Iron Gate Valves for pipes 15 cms in diameter.	6	each	160.00	900.00
12b	Cast Iron Gate Valves for pipes 10 cms in diameter.	3	each	100	300.00
	TOTAL PRICE FOR PROJECT				42,517.83

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

As far as the septic tanks are concerned, modification of the existing units can possibly be made with the least expenditure. It is believed that such modification will not be final since the size of the tanks needs to be increased. Thus the tanks should be replaced by larger tanks or other tanks added. Also to have a proper functioning of the septic tanks, the tile fields for spreading the sewage should be replaced or increased in accordance with a new design. This of course, would be much less costly than the other two possibilities, however, the author still recommends the abandonment of such disposal method for reasons discussed in chapter I. So it is assumed that two possibilities will be left, i.e., the construction of waste stabilization lagoons, or the construction of a conventional treatment plant.

For the lagoons, the cost of construction as shown earlier was L.L. 29 thousand for a population of 560 persons. However, in conventional units, no such detailed estimate has been done for reasons mentioned previously. Instead there follows an average value given by a well known firm* in Beirut for the cost of construction of conventional units in this part of the world. The cost is about L.L. 100 per person or equivalent contributing to the unit for plants of about this size.

The minimum estimated population equivalent (p. 3) is 276. Based on this average value of L.L. 100 person, the minimum cost of

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* Associated Consulting Engineers.

construction of conventional units such as trickling filters, will amount to 276 x 100 = L.L. 27,600. This cost, however, should be more since the population is expected to increase to more than 276 persons equivalent.

In any case, comparison of the total cost of the two different types of treatments show that both types cost about the same price; but in the case of lagoons, they are designed to treat wastes produced by a population almost twice that of conventional units.

If it can be reliably ascertained that the maximum population equivalent will be less than 560, the size and hence cost of the lagoons can be reduced. This means that the cost of construction of waste stabilization lagoons would be less than 29 thousand Lebanese pounds.

Hence if the two methods are compared on the basis of the same load, the lagoons will be much less expensive to construct.

Operational and maintenance costs are also much less in lagoons than in conventional units, which further justifies the recommendation for lagoons.

These two points are on a strictly cost basis. However, it must be remembered that lagoons have other advantages over conventional units, as mentioned in Chapter I.

Pending the authorization for the execution of the work, the author suggests the verification of the amount of domestic water consumed at the farm, and the B.O.D. concentration of sewage produced. The information can be found by the following means:

1. Determination of Water Consumption.

The amount of water consumed at the different buildings in the farm is required in order to determine the total sewage flow into each septic tank. To accomplish this, it would be necessary to install several meters at strategic points in the plumbing system. This method is the best and most accurate; however, if found costly a minimum of one meter should be installed directly after the upper reservoir that feeds the whole farm. In this case, water consumed by each building would have to be estimated as closely as possible.

In each of the above two procedures, readings of the meters should be taken at the beginning and at the end of each month; hence, the daily average water consumption can be found for that given month. This is to be repeated throughout the year, with special care in July, August, and September.

2. Determination of B.O.D. Load.

Representative composite samples of sewage should be routinely collected from the influent of the four septic tanks on different occasions and times throughout the year, and tested to find out the 5 day B.O.D. load of the sewage.

Knowing the water consumption and hence the sewage flow, and the B.O.D. load of the sewage influent of each septic tank are known, the overall B.O.D. load of the total sewage can be calculated.

APPENDIX A

Symbols and Abreviations

A.U.B.	American University of Beirut.
B.O.D.	Biochemical oxygen demand incubated at 20° and for a period of 5 days.
L.L.	Livre Libanaise (Lebanese pound).
lat.	Lateral sewer.
P. or PRIM	Primary sewer.
Sec.	Secondary sewer.

APPENDIX B

Historical Development and Theory of Waste Stabilization Lagoons

Literature on this subject has been written by many authors over the world. During the last few years, an increased interest has been developed in the use of ponding as a good, efficient, and economical method for the treatment of many wastes.

The increased popularity in the use of this method, goes back to many factors. Such factors that yield better results in relation to efficiency, economy, and simplicity for small units where land is cheap.

Although different people have different opinions in different countries about this subject, yet all believe that ponding is to be the first treatment method to be thought of in the future.

Having this in mind, the following discussion is placed in here, since it seems necessary to give a clear and better understanding of this method of sewage treatment.

1. Terminology, History and Classification

a. Terminology. The terms lagoon and oxidation pond have been used by many people to mean the same thing, that is, all types of earthen ponds used to treat sewage wastes. A more developed definition is to define as oxidation ponds all those that are regular in shape, depth, and marginal area; in other words, artificially made ponds specially designed for sewage treatment. While the term lagoon was given to those ponds that are not controlled with respect to depth, shape, and marginal area, that is natural low lands that are used for sewage treatment.

Both of the above two types showed good and satisfactory results, but the use of oxidation ponds is more common, since they are easier to maintain, and are more sure to produce good results(9).

A still more developed definition used these days, and accepted by many people is considered as the basic terminology to this type of treatment. A waste stabilization lagoon is that pond that receives into its body of water raw sewage; thus acting as a full treatment for the wastes (10, 11). On the other hand, an oxidation pond receives into its body of water effluents from primary units; thus acting as a secondary unit. Both types are artificially made with a controlled depth, shape, and marginal area.

It is noted that the terminology is now effected by the type of sewage, raw or partly treated, while before the trend was to distinguish the regularity and control of shape.

b. <u>History</u>. Although it is reported that the use of sewage lagoons goes back to many centuries (4), yet the first recorded sewage lagoon dates back as only to 1905, when the city of San Antonio, Texas, started the construction of a ditch, dam, and reservoir on Mitchel Lake for the proper disposal of the sewage of San Antonio (12).

During the year of 1925, a student at the University of Texas was engaged in a survey for the State Department of Health, in order to investigate why the sewage of Palestine town that was discharged into a swampy area appeared to look like fresh water a few miles away.

The result of this investigation led Mr. Vic Ehler, Texas State Sanitary Engineer, to the probability that the aquatic plants growing in the swampy area are responsible for the stabilization of the sewage. In about 1929, Mr. Ehler recommended the use of sewage lagoons for Abilene City. Results at Abilene City encouraged the Texas A and M College to construct an experimental lagoon, 14 acres in area.

Other lagoons at this early stage were also constructed in California, North Dakota, and other states. Lagoons that were built between 1905 and 1948 were constructed haphazardly, not being based on any engineering principle, or standards. This period was transitional and used mainly for investigation (12).

In 1948, the end of this transitional period came about, and the first lagoon built on sound engineering principles, and according to the modern concept of design was that of Maddock, North Dakota (12). The area of this last lagoon was 10 acres. It served a population of 1000; with a mean depth of 5 feet. Results turned out to be successful.

The success of this lagoon at Maddock encouraged many other towns in North Dakota to install such treatment units. Other states later followed, and by 1955, net less than a 100 lagoons were constructed in what is called the Missouri Basin States.

It is concluded that the lagoon at Maddock has the honor of being the first modern built lagoon (1948), as does the lagoon in San Antonio in being the first recorded sewage lagoon (1905).

c. Classification. Waste stabilization lagoons can be classified into three different classes, anaerobic, fucultative, and aerobic lagoons. A discussion of each of these classes is given hereunder.

(i) Anaerobic Lagoons: As the name implies, they are lagoons where anaerobic reactions predominate. The major part of the B.O.D. is decomposed by the methane producing bacteria (methane fermentation). Such lagoons could be as deep as 10 ft. or even more, since anaerobic reactions take place. Loadings can be in excess to 200 lbs of B.O.D. per acre per day, and even up to 2000 lbs of B.O.D. per acre per day for a depth of 5 ft., and 1200 for a depth of 3 ft. (10).

Care must be considered in such units, since they produce bad odors, and are thus prohibited in or near dwelling areas. They are good in areas far from cities where odors thus formed are tolerated.

Anaerobic ponds are to be followed by aerobic ponds. To show the importance of this type of ponds, follow through the following example.

City X with a population of 6000 persons. Each person produces 0.17 lbs of B.O.D. each day. Anaerobic lagoon's loading = 2000 lbs of B.O.D. per acre per day.

Aerobic lagoon's loading = 40 lbs of B.O.D. per acre per day. If only aerobic lagoon is to be used, the area will be:

$$\frac{6000 \times 0.17}{40} = \frac{1000}{40} = 25 \text{ acres.}$$

However, if a combination is used

$$\text{Area of anaerobic lagoon} = \frac{1000}{2000} = \frac{1}{2} \text{ acre}$$

$$\text{two ponds} = 2 \times \tfrac{1}{2} = 1 \text{ acre}$$

$$\text{B.O.D. left} = \frac{1000 \times 30}{100} = 300 \text{ lbs of B.O.D.}$$

$$\text{Hence area of aerobic lagoon} = \frac{300}{40} = 7\tfrac{1}{2} \text{ acre.}$$

So comparing the two values of 8.5 acres and 25 acres, the importance of the combination of anaerobic -aerobic lagoons can be noticed.

These anaerobic lagoons are to be designed in pairs, so that one is under operation, while the other is not. It should provide for one year of sludge accumulation, with enough room left for detention time. Lagoons of this type produce 1 ft. of sludge per year for a detention of 2 days, and an average B.O.D. reduction of 70% (13). For smaller detentions, more frequent collection of sludge is to be made, otherwise the depth allowed should be more than one foot.

(ii) Facultative Lagoons: In facultative lagoons the B.O.D. is removed as a result of both aerobic reactions take place at the surface and upper layers, and anaerobic reactions take place at the bottom layers due to the settle sludge.

These lagoons are the ones of greatest interest in relation to the one proposed in this thesis. Hence details of this type of lagoons will be presented and discussed in the following text.

(iii) Aerobic Lagoons: Organic matter in this kind of lagoons is stabilized solely by aerobic bacteria. Such lagoons are designed with a shallow depth of about 6 inches to a foot, and thus possess a bigger ratio of surface area to volume than do other ponds. Oxygen here is introduced by either mechanical means, or by growing a large quantity of algae. When mechanically aerated, loadings as high as 500 lbs of B.O.D. per acre per day can be used. On the other hand, when algae is supplying the oxygen by photosynthesis, a lower value of 200 lbs of B.O.D. per acre per day can be used.

It should be noted that only the facultative lagoons, used in the proposed design will be discussed fully. The reason is that they are the most important and the ones serving the purpose of lagoons (economy and efficiency). The first, although more economical, produces odors and nuisance, while the third uses a bigger area for a fixed retention time and needs more maintenance.

2. Theory of Waste Stabilization Lagoons

a. Application. The process that takes place in stabilization lagoons occurs naturally, to a greater or lesser extent, in most localities of the world. Hence some type of stabilization lagoon might be utilized satisfactorily anywhere (14). Based upon this fact, it is reasonable to assume that lagoons will be applicable in Lebanon with a certain set of conditions.

In general, the applicability of lagoons is confined to small communities with a range in population from 20 persons to 25,000 persons. Most common are between 200 and 5000. Lagoons are not good for large cities. The reason behind this lies in the fact that lagoons demand a large surface area that is rare to find in large cities (and if found, is expensive).

The fact that many factors such as water quality, rainfall, wind speed, sunlight intensity, evaporation rates, soil conditions, and temperature influence the design, restricts the application of lagoons to some areas (10).

A cold area, with little sunlight is not good for proper operation of lagoons. Also areas with high percolation rates are not recommended, although methods of reducing such losses are now available. Modification of the soil by such methods will involve extra expense that should be avoided unless such an action is absolutely essential.

With respect to the type of sewage, lagoons have gained popularity not only in stabilizing domestic sewage, but also in treating many non-toxic industrial wastes. Laundry, coffee, creamery and poultry processing plants, canning, abbatoirs, dairies, refineries and sugar beets wastes are being treated by lagoons (9, 10, 15, 16, 17).

Lagoons are used for the removal of suspended solids and the biochemical stabilization of colloidal and dissolved organic solids (18). They are used also to regulate waste discharges, to conserve the water for irrigation, and fertilize the water for fish propagation. Sewage ponds, moreover, are used for cultivation and harvesting algae which can serve as an excellent source of animal food.

b. Principles of Stabilization. The sewage treatment taking place in sewage lagoons is fundamentally that of any other biological treatment method. However, all reactions, physical, chemical and biological take place in one single unit. That is sedimentation of organic matter, precipitation of organic salts (flocculation), and finally the organic matter stabilization.

A waste stabilization lagoon is an aerobic treatment device. Anaerobic zones might result and often become a combination of aerobic anaerobic unit.

The stabilization of the organic matter is brought about by many micro-organisms. The most predominating organisms are bacteria which are unicellular microscopic organisms. The reactions are mainly oxidation and reduction. The oxidation reaction is the important one since it is done by the aerobic bacteria. Aerobic bacteria oxidizes the organic matter to a final inorganic stage of carbon dioxide (CO_2), and water. Anaerobic bacteria reduce the organic matter to organic acids that are in turn a B.O.D. load. Still further, the organic acids are reduced to the organic form of methane.

Together with oxidation and reduction reactions, photosynthetic reactions take place by the help of a green form of plants called algae.

Bacteria and algae live in the same surrounding peacefully without competition, and with natural benefits. This phenomenon is termed symbioses. This symbiotic effect is essential to lagoons; since without it they would not operate successfully.

The following schematic diagram Fig. B 1, shows this symbioses between bacteria and algae.

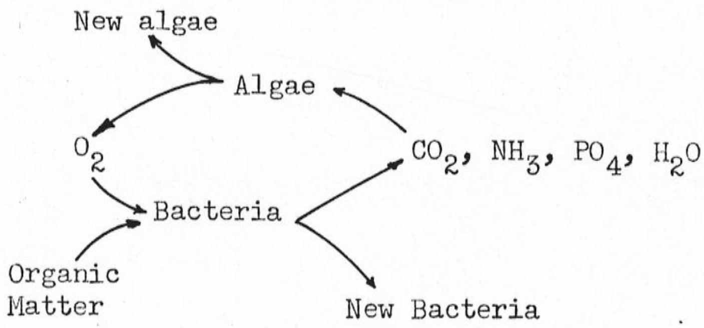

Fig. B 1.

Bacteria-Algae Symbiosis (19, 20)

The bacteria metabolize the organic matter to form carbon dioxide, water, ammonia, and others, while using oxygen. The presence of oxygen is important to insure aerobic reactions. At the same time, and only in the presence of sunlight, direct or diffused, algae utilizes the by-products of bacteria mainly carbon dioxide to form new cells of algae and produce oxygen as a by-product. The oxygen liberated in turn, is absorbed by the bacteria, and the cycle is repeated.

The following chemical reactions will help in further explanation:

a) As given by Oswald et al in 1958 (21)

$$C_{11}H_{29}O_7N \;+\; 14O_2 \;+\; H^+ \dashrightarrow 11CO_2 \;+\; 13H_2O \;+\; NH_4$$

or (22)

$$\text{Organic Matter + Other nutrients in sewage + Oxygen} \xrightarrow{\text{bacteria}} CO_2 + NH_3 + H_2O$$

$$\text{Green Plants} \;+\; \text{Sunlight}$$

c) Considering carbon, hydrogen, and oxygen, it follows the reversable reaction: (22)

$$nCO_2 + nH_2O \underset{\text{Bacteria}}{\overset{\text{Algae + Light}}{\rightleftharpoons}} (CH_2O)_n + nO_2$$

It is to be noted that the biologic reactions given above are considered to be reversable:

$$\text{Organic Matter} + O_2 \xrightarrow{\text{bacteria}} CO_2 + H_2O \xrightarrow{\text{algae}} \text{Organic Matter} + O_2$$

The organic matter represented by $(CH_2O)_n$ is of two types. It represents dead organic matter in sewage which is oxidized and the live organic matter of cell material which is synthesized.

So in essence, there is little or no decrease in the organic content of sewage that passes through a stabilization lagoon. Since the putrescible raw wastes are converted to relatively stable algae cells.

Algae are discharged with the effluent, and may act as a good source of food to fish in streams, or recovered as animal food.

Millions of bacteria, aerobic and anaerobic come with raw sewage and the lagoon. In the presence of oxygen, aerobic bacteria will dominate, while in the absence of dissolved oxygen, anaeorobic bacteria will start working. In the upper surfaces of lagoons, plenty of oxygen is present that insures a complete aerobic reaction. This oxygen is produced by the algae in day time, and the action of rearation at night. In cases the oxygen concentration in the pond drops below the average value of dissolved oxygen. Usually oxygen in the pond will be in the supersaturated state and will last till the second day. Oxygen regenerated from the atmosphere is not enough to keep aerobic reactions; it is equivalent to less than 40 lbs of B.O.D. per acre per day (12). This is for sure obvious that another source is to supply oxygen - it is the algae.

In the lower layers, facultative bacteria takes over, and the settled sludge is stabilized by the anaerobic bacteria at the bottom. The following schematic diagram Fig. B 2, shows the above discussion.

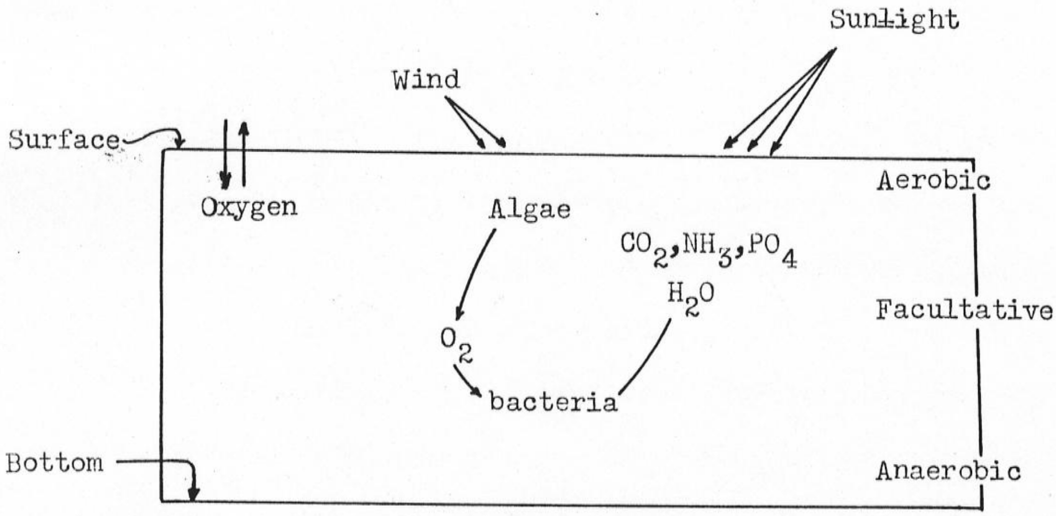

Fig. B 2 - Schematic Diagram of Oxidation Pond Operation (19).

This is a brief description of what takes place in a pond. However, it is felt incomplete to stop at this point without discussing some characteristics of bacteria and algae in relation to waste stabilizations.

Bacteria:

They are microscopic unicellular micro-organisms. Bacteria, fungi, and protozoa are responsible for the biological decomposition of the organic matter. Bacteria, although the smallest of them, metabolize the fastest.

Bacteria is found practically everywhere. They are found in the atmosphere, in the food, and in the soil. They are found in and outside our bodies. They go out from the body with the excreta and thus are found in the sewage.

With regards to resistance, bacteria act as follows:

1) Some toxic materials stop the action of bacteria and may lead to a poor stabilization lagoon (23). This is one reason why toxic wastes cannot be treated by ponding.

2) Temperature has a great effect on the reactions taking place. Higher temperatures will yield a faster reaction and hence faster stabilization. Low temperatures as of freezing, might kill or at least stop the biochemical activity.

3) Oxygen requirements. Available dissolved oxygen will insure aerobic reactions, while the absence of dissolved oxygen will let anaerobic bacteria take over, on condition there are compounds containing oxygen such as nitrates and phosphates.

4) pH. Most bacteria grow best between a pH of 6.0-8.0.

5) Food Supply. All living organisms, including bacteria need food for their growth.

Algae:

Algae are classified as plants and they convert carbon dioxide and water in the presence of sunlight to starch and oxygen. The oxygen thus produced is utilized by the bacteria for the aerobic stabilization of the organic matter in the stabilization lagoons.

There are many species of algae that grow under various conditions, some flourishing during certain localities with optimal climatic and environmental conditions.

The growth of algae is governed by many factors.

1. Temperature has an effect on the type of species of algae. Some prefer high temperature (flourish in summer), and others prefer low temperature (flourish in winter). This is why they change with the season of the year.

2. Type of food. Algae prefer phosphates and nitrates as a source of food. Characteristics of wastes determine the type of algae to live in the lagoon.

3. Sunlight. In the absence of direct sunlight most algae die, since they are plants requiring solar energy for the photosynthetic process. Algae are not found in deep waters where sunlight cannot penetrate. This is the reason why ponds are restricted to a maximum depth of 5 ft.

A more detailed discussion on the growth and recovery of algae is placed at the end of this appendix.

c. Methods of Design:

The following design methods are mainly concerned with waste stabilization lagoons; that is ponds receiving raw sewage. Other types of ponds were discussed briefly early in this chapter.

Until very recently, no rational formula was presented for the design of lagoons. Design criteria was mainly based on actual results of existing pilot plants, and an intelligent guess on the part of the designer. It is very natural to expect differences in view on such a subject that lacks a fixed rational formula. Different opinions based on experimental observations were developed. In the United States, as well as some other countries, design criteria varies between 20 and 60 lbs of B.O.D. per acre per day. Most acceptable value is 40 lbs of B.O.D. per acre per day.

The most efficient loading cannot be known exactly for a given country or part thereof. However, countries with warm winter climates can safely use the upper limit, while countries with cold winter climates, are advised to use the lower limit. High values up to 100 and even 170 lbs of B.O.D. per acre per day have been practiced on pilot plants and produced satisfactory results.

Recently several theories for the design of lagoons have been developed by different authors (4). In this report only two will be discussed, namely that developed by Oswald and Gotaas (24), and that developed by G. Van R. Marais (25, 26). The approach followed by each is totally different from the other.

Oswald and Gotaas use the availability of sunlight energy and thus the photosynthetic efficiency of algae in producing the required oxygen for stabilizing a certain load of pollution, as a basis of their design theory. G. Van R. Marais bases his theory on the mono-molecular theory that is expressed as:

$$dS = -KSdt$$

Since all these theories are recent, no practical proof has yet been found to distinguish the best theory.

1. <u>Oswald and Gotaas:</u>

In order to develop practical equations for the design of oxidation ponds, it is necessary to assume that ponds are operated in such a way that all the oxygen required by bacteria comes from the development of new photosynthate. To safeguard such an assumption, it is taken for granted that optimum conditions for the survival of algae are present.

The weight ratio of the released oxygen to the organic matter produced is expressed as a factor P:

$$P = \frac{\text{Wt. of Oxygen released}}{\text{Wt. of Organic Matter produced}} = \frac{Wo}{Wom} \quad \ldots\ldots\ldots\ldots \text{1-a}$$

The available stored energy (H) of the organic photosynthate is equal to the product of its unit heat content (h) and weight (Wom).

$$H = h \times Wom; \text{ or } h = \frac{H}{Wom} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{1-b}$$

The original solar energy (Es) is equal to the total heat energy divided by a factor (F) that represents the efficiency of energy conversion.

$$Es = \frac{H}{F}\text{ ; or } F = \frac{H}{Es}\text{ ; or } H = FEs \ldots\ldots\ldots\ldots \text{1-c}$$

This means that H is proportional to the solar energy and is of the same dimensions.

$$Es = S.A.D. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{1-d}$$

Where:

S = insolation in langleys

A = surface area exposed to light in sq. cm.

D = retention time in days

$$S.A.D.F. = h.\ Wom$$

$$\text{or} \quad F = \frac{h.Wom}{S.A.D.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{1-e}$$

From the above equation, H = S.A.D.F., and this energy stored in algae cells can be found from the product of the heat of combustion of algae (h), by the concentration of the algae (Cc) expressed in ppm rather than Wom.

Considering one liter with a depth of d cms, then the surface area will be

$$A = \frac{1000}{d}$$

$$\text{Hence} \quad H = h.\ Cc. \quad = \quad S.\ \frac{1000}{d}.D.F. \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{1-f}$$

$$\text{or} \qquad D = \frac{d.h.Cc}{S.1000.F}$$

The biochemical oxygen demand expressed as (L_t) in any time (t); is equal to Wo as assumed. Also Wom = Cc.

$$P = \frac{Wo}{Wom} = \frac{Lt}{Cc}$$

$$\text{or} \qquad Cc = \frac{Lt}{P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{1-g}$$

Combining equations 1-e and 1-g

$$D = \frac{d.h.Lt}{1000.F.S.P.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{1-h}$$

The efficiency (F) in the above equations is affected by many environmental factors. Quantitative effects of the many environmental factors are generally unknown.

For example, the temperature affects both algae and bacterial growth.

So equation 1-h becomes

$$D = \frac{d.h.Lt.}{1000.S.P.Tc} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 1\text{-}i$$

Before applying the equation 1-i, for the design of ponds, the inter-relation between the several factors of the equation should be known.

It is to be noted that such factors are influenced by many environmental conditions.

Depth: Experimental data have shown that a suspension of algae cells absorbs light within close limits set by the Beet-Lambert Law.

$$\frac{I}{I_1} = e^{-Cc \propto d} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 1\text{-}j$$

Where I_1 = incident light intensity

I = measured light intensity at depth d.

$\propto$ = specific absorption coefficient

Taking the natural logarithm of both sides of equation 1-j;

$$\log_e I - \log_e I_1 = -Cc \propto d$$

For a practical design, it is assumed that all the light is used; thus I at the bottom will be approximately zero. So equating I = 0, and solving for d.

$$d = \frac{\log_e I_1}{Cc. \propto}$$

The value for is usually given as $1.5 \times \overline{10}^3$.

The value for h is given as 6 K. cal per gram of sewage grown algae.

The value for P is generally given as 1.25 - 1.75.

Efficiency (F): It is a variable, dependent on many environmental factors such as light duration, intensity, and temperature. No firm value is given for the maximum efficiency although it may be more than 50% under normal conditions (24).

Temperature Coefficients: These coefficients (Tc) vary with temperature as in the following Table (Table B_1).

Temp. C°	Tc
0	-
5	0.26
10	0.49
15	0.87
20	1.00
25	0.91
30	0.82
35	0.69
40	-

Table B_1 - Values of Temperature Coefficients Used in the Design of Oswald and Gotaas (24).

Insolation: Values for the factor S, are given in Table 2. This is dependent on astronomical, geographic, and meteorological phenomena. Corrected values are to be used in the design.

For the intensity of incident light, means of its calculation have been approximately found.

INSOLATION

Probable Average Values — Direct And Diffuse — On A Horizontal Surface At Sea Level In Langleys² Per Day As A Function Of Latitude And Month

North Latitude (Degrees)	Range	Jan (vis³ tot⁴)	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0	max⁵	255 408	268 700	271 708	258 650	249 546	236 428	238 930	232 468	269 490	266 694	226 588	255 667
	min⁶	210 390	219 363	208 534	198 442	182 450	108 274	137 288	147 482	207 533	203 130	202 548	199 527
1	max	232 490	268 698	271 707	247 593	232 652	238 638	241 638	254 670	269 692	194 591	264 678	251 656
	min	208 370	216 370	209 535	188 445	158 482	105 281	139 370	168 438	207 538	108 527	200 534	193 516
2	max	250 670	265 668	271 706	267 697	255 665	241 642	244 646	255 673	269 695	242 666	251 566	240 646
	min	204 580	213 680	204 534	188 464	184 484	108 288	141 375	159 442	208 531	200 523	198 526	188 508
3	max	244 590	261 690	270 705	258 700	258 660	244 648	247 652	256 678	229 694	261 684	249 598	246 658
	min	203 560	210 560	203 533	187 465	156 487	110 296	144 380	170 444	205 330	198 519	196 516	186 493
4	max	244 660	280 666	270 704	298 701	258 659	247 586	250 687	250 679	259 695	242 666	246 650	244 628
	min	202 540	208 543	202 532	187 466	187 492	113 300	146 385	171 448	204 529	195 513	194 510	186 490
5	max	240 640	256 686	289 703	299 703	260 470	250 682	282 663	240 680	269 698	259 679	243 642	240 920
	min	197 530	208 538	201 531	135 466	166 498	116 305	148 390	172 400	203 527	194 507	190 502	180 470
6	max	238 630	254 678	288 702	270 708	262 675	288 690	288 619	261 682	269 697	236 670	240 634	238 610
	min	193 520	199 530	200 530	184 467	159 500	115 310	150 395	172 452	202 524	191 500	188 494	176 450
7	max	234 620	252 470	268 701	270 707	264 680	285 673	258 476	262 685	268 597	254 666	237 624	236 600
	min	190 508	198 520	198 528	185 487	190 502	122 315	152 400	173 454	201 521	189 494	185 486	173 460
8	max	230 610	249 668	287 700	270 708	298 665	258 678	260 680	263 666	267 696	252 660	234 616	231 590
	min	187 495	192 510	196 523	188 467	191 504	124 320	154 405	174 456	200 516	186 486	182 478	169 440
9	max	226 600	247 660	266 696	270 710	268 690	260 583	262 685	264 690	267 694	250 656	231 606	228 580
	min	184 488	188 500	195 519	184 465	192 509	126 325	156 410	176 458	198 514	184 490	179 470	166 430
10	max	223 586	244 655	264 694	271 711	270 694	262 688	260 690	258 693	268 693	248 650	228 600	225 570
	min	179 475	184 480	193 514	183 464	192 512	129 330	158 414	176 480	195 510	181 474	176 462	162 420
11	max	220 588	241 680	263 692	271 711	271 698	268 698	267 695	254 698	266 692	246 648	224 594	221 590
	min	175 468	181 480	191 507	182 448	193 515	131 335	160 418	176 482	129 500	179 468	172 454	158 410
12	max	216 572	238 646	262 690	271 710	273 702	267 700	269 700	267 697	264 691	244 640	221 585	217 550
	min	172 456	176 470	189 500	181 462	195 518	133 343	161 421	176 464	193 502	176 482	169 446	154 400
13	max	212 588	236 640	260 689	271 710	274 706	270 708	271 704	266 598	263 690	242 634	218 578	213 544
	min	168 446	172 460	187 492	180 461	194 520	136 350	163 424	178 466	191 489	173 457	136 438	150 390
14	max	208 558	238 630	259 680	271 709	278 710	272 710	273 708	269 700	262 688	240 627	214 587	208 536
	min	163 430	167 480	184 487	179 460	194 524	137 384	164 428	177 487	189 496	170 449	182 410	146 380
15	max	204 548	230 620	256 675	272 708	279 714	274 715	275 712	270 702	250 686	237 620	210 560	206 524
	min	159 420	163 440	182 480	178 458	194 526	140 360	166 432	177 468	187 492	151 442	138 420	142 370
16	max	200 530	226 610	255 670	272 707	279 718	276 720	277 715	270 703	259 684	234 615	206 554	200 520
	min	154 400	158 430	180 475	177 456	194 528	141 343	167 435	177 468	165 489	154 434	154 410	138 360
17	max	196 528	224 600	253 668	272 708	280 720	278 724	278 718	271 704	257 692	232 610	202 545	196 510
	min	149 390	158 420	177 466	175 454	194 529	143 369	168 437	177 470	133 414	161 428	150 400	134 350
18	max	192 515	220 590	250 684	272 708	282 723	280 728	280 723	272 705	256 680	229 605	198 530	192 500
	min	144 380	150 410	174 459	174 452	194 530	145 375	170 442	177 471	180 479	157 418	146 390	129 340
19	max	188 505	216 580	248 688	271 704	283 727	282 732	281 724	272 706	254 679	226 600	194 530	187 490
	min	138 370	145 400	171 453	172 450	194 532	146 380	171 447	177 471	178 473	154 410	142 380	125 330
20	max	183 500	213 575	246 632	271 703	284 730	284 738	282 729	272 706	252 674	224 596	190 520	182 480
	min	134 360	140 390	168 440	170 447	194 532	148 383	172 450	177 472	176 467	150 400	138 370	120 320
21	max	178 490	210 570	244 648	270 702	285 732	285 742	284 732	272 707	250 671	221 599	186 510	177 470
	min	129 350	136 380	165 430	168 443	194 632	150 388	172 452	176 472	173 461	147 390	133 360	115 310
22	max	174 480	206 560	241 644	270 701	286 734	286 747	285 736	273 707	248 669	219 582	183 500	172 490
	min	125 338	132 370	162 426	167 440	193 530	152 392	173 454	176 472	170 455	145 380	128 350	110 300
23	max	170 470	203 585	239 630	269 700	287 736	288 750	286 730	273 708	246 663	216 576	179 490	167 450
	min	118 320	128 355	159 418	166 438	192 528	153 398	175 458	175 472	168 449	140 370	124 338	106 290
24	max	166 460	200 548	236 628	268 697	288 738	290 758	287 742	273 708	244 658	212 588	175 480	161 440
	min	111 310	123 340	156 410	164 433	191 525	155 403	178 469	174 471	166 443	136 360	119 326	101 280
25	max	160 450	198 540	234 620	267 693	288 740	291 767	287 746	273 707	245 656	208 560	171 470	156 430
	min	105 298	119 323	153 400	162 429	190 522	154 406	177 461	173 470	163 436	132 344	114 313	78 272
26	max	156 440	192 530	230 618	266 690	288 741	292 780	288 749	273 708	240 462	208 588	166 460	149 420
	min	99 280	114 310	149 390	160 428	188 518	158 409	177 483	172 469	160 429	128 328	109 300	98 280
27	max	150 430	188 520	228 608	265 687	289 742	293 782	289 752	272 705	238 646	202 548	162 460	144 410
	min	93 268	110 300	146 380	158 420	188 612	159 412	179 468	170 468	157 423	124 320	104 289	98 247
28	max	146 420	184 510	224 603	264 683	289 743	294 784	288 756	272 704	236 638	198 537	157 440	138 400
	min	87 250	109 299	142 375	158 416	187 506	161 418	179 457	169 466	154 418	120 319	99 278	80 234
29	max	141 410	180 500	222 595	262 680	290 744	295 766	289 757	272 703	234 630	198 531	153 430	132 390
	min	81 230	100 280	138 366	158 410	186 498	162 422	175 458	168 464	160 407	116 300	98 287	76 223
30	max	136 400	176 490	216 587	261 675	290 744	298 768	290 720	271 702	231 626	192 524	148 420	126 380
	min	79 220	98 260	134 348	151 408	184 490	163 425	175 460	168 462	147 399	113 290	90 256	70 210

North Latitude (Degrees)	Range	Jan (vis tot)	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
30	max	136 400	176 490	216 587	261 675	290 744	298 700	290 754	271 702	231 626	196 534	148 420	126 380
	min	79 280	98 260	134 362	151 400	184 490	163 427	178 480	166 462	147 399	113 290	90 256	70 210
31	max	130 380	173 480	216 580	260 680	290 740	296 770	290 760	270 701	228 620	190 517	143 410	120 370
	min	80 200	92 250	130 350	140 400	185 402	164 428	179 471	164 462	144 388	108 290	96 240	60 198
32	max	128 380	169 470	212 570	258 653	290 744	296 775	290 761	269 700	226 610	186 616	138 400	114 300
	min	63 180	87 240	126 340	146 399	181 475	166 484	179 472	163 480	140 388	104 270	90 224	60 184
33	max	120 370	164 460	208 560	256 660	290 748	297 774	299 760	269 690	224 607	182 500	133 390	107 380
	min	58 168	83 230	122 330	146 390	178 460	167 438	179 472	161 498	137 377	100 288	75 213	64 169
34	max	114 360	160 480	204 583	284 667	290 743	287 775	299 768	257 690	221 602	176 490	128 380	101 338
	min	53 158	78 215	116 320	141 385	176 382	148 480	178 472	159 448	134 366	96 280	79 203	47 150
35	max	108 390	159 440	200 547	282 653	290 742	298 776	280 764	266 690	218 697	174 480	125 370	96 328
	min	48 145	73 210	115 310	130 300	174 458	169 441	178 471	187 400	130 390	92 240	54 191	43 146
36	max	103 339	130 430	196 530	280 650	288 761	290 776	299 790	264 680	215 580	170 470	118 360	98 314
	min	46 138	72 200	111 300	130 375	172 444	170 448	177 470	188 438	127 380	90 230	90 180	39 134
37	max	97 320	140 429	193 530	240 645	287 740	298 777	288 746	253 680	213 580	146 460	112 340	82 300
	min	40 128	56 190	107 290	134 370	166 436	170 446	176 400	84 434	123 340	64 280	56 180	36 132
38	max	90 310	140 419	189 520	246 640	287 730	298 779	288 790	262 684	210 378	162 450	106 330	77 290
	min	36 120	52 180	103 280	131 365	166 428	171 446	175 464	132 429	120 330	90 215	50 180	30 111
39	max	86 295	138 405	185 510	244 636	288 736	298 778	288 765	260 682	206 500	157 440	101 326	71 280
	min	33 110	57 170	98 275	128 360	164 420	171 480	174 480	180 428	116 320	76 109	48 146	27 102
40	max	80 280	130 390	181 500	241 630	286 732	298 778	290 768	298 680	203 582	152 430	96 313	66 270
	min	30 108	53 160	96 270	125 355	162 418	173 480	172 438	147 418	112 310	72 202	42 134	24 94
41	max	78 270	124 380	179 490	238 624	284 730	298 777	287 763	256 675	200 598	147 420	90 302	64 260
	min	27 98	49 130	92 260	123 349	159 470	175 [illegible]	170 450	149 420	109 300	68 198	37 123	22 86
42	max	68 255	119 370	172 485	236 618	283 728	298 777	287 761	254 670	196 547	144 410	86 288	56 244
	min	24 90	45 140	88 250	120 344	157 408	174 [illegible]	167 442	145 403	106 290	65 187	34 112	18 79
43	max	62 240	113 350	169 480	233 612	282 724	298 777	298 730	252 668	192 539	138 400	79 276	52 280
	min	22 84	42 135	84 240	117 337	155 400	174 452	165 435	141 398	103 280	62 180	31 106	17 70
44	max	55 228	106 340	165 470	230 607	280 722	298 777	285 755	250 660	189 530	132 380	72 263	47 218
	min	20 80	37 130	80 230	114 325	153 395	175 453	164 430	139 389	98 270	58 173	28 90	18 62
45	max	50 210	100 330	161 460	227 602	279 720	296 778	284 752	248 658	186 520	128 380	67 250	42 207
	min	18 78	34 120	75 220	111 320	151 390	175 434	162 425	137 388	94 260	55 168	28 92	13 58
46	max	45 200	94 315	156 450	224 590	278 716	298 775	294 749	242 650	81 542	122 370	61 238	38 184
	min	16 74	30 110	72 210	108 315	150 388	175 455	161 420	134 374	90 250	52 158	23 86	11 48
47	max	40 190	38 300	153 440	221 593	276 713	296 778	293 748	242 649	178 504	116 300	58 225	34 182
	min	14 70	26 104	68 200	105 310	146 380	175 456	159 445	132 368	86 240	48 151	21 78	10 48
48	max	35 190	32 290	149 430	218 582	274 710	294 776	282 740	241 640	174 496	111 390	50 210	32 170
	min	12 64	25 99	64 190	102 307	146 379	176 430	153 410	129 358	81 230	45 144	18 70	9 37
49	max	32 172	76 280	144 420	214 578	272 702	297 778	281 737	238 639	170 488	106 340	44 196	29 188
	min	11 60	22 98	61 180	100 303	145 373	176 430	156 407	127 350	77 280	42 137	17 48	8 33
50	max	28 164	70 265	141 410	210 568	271 703	297 776	280 733	234 625	166 480	100 329	40 163	26 144
	min	10 54	19 90	55 173	97 300	144 371	176 450	152 408	125 342	73 210	40 130	15 90	7 50
51	max	24 152	64 255	137 400	206 562	269 700	296 776	279 729	233 620	162 470	93 318	38 170	23 138
	min	9 50	15 70	54 165	96 298	142 308	176 440	156 400	123 335	69 200	37 125	13 57	5 28
52	max	22 140	60 240	134 390	202 558	267 696	296 776	278 725	232 615	156 460	87 307	32 160	21 121
	min	8 45	14 62	51 158	92 295	141 366	176 460	153 398	120 328	65 180	34 120	12 53	4 27
53	max	18 130	55 225	130 380	198 550	265 640	296 776	277 723	227 610	153 480	82 296	29 130	18 110
	min	7 42	12 53	48 152	90 292	140 342	176 460	151 366	118 319	62 180	31 113	14 48	4 25
54	max	16 120	50 215	126 370	194 542	263 687	296 776	276 720	224 602	150 440	75 285	26 140	16 99
	min	6 40	11 45	46 145	88 289	139 380	176 460	150 394	116 312	58 170	29 154	9 45	5 26
55	max	14 110	46 210	123 360	190 534	260 684	296 775	274 718	221 590	146 430	70 273	22 130	14 88
	min	5 38	9 40	44 138	87 284	138 358	176 460	148 392	113 308	54 160	26 100	8 30	2 20
56	max	12 108	44 200	120 350	188 528	258 680	295 775	272 714	218 587	141 420	64 264	20 120	12 76
	min	4 35	8 36	41 132	85 283	136 382	175 460	148 390	116 297	51 150	24 95	7 38	2 19
57	max	10 88	40 190	116 340	186 522	256 675	295 776	272 712	216 582	138 410	58 250	17 110	10 84
	min	4 30	7 30	39 126	84 280	138 330	175 460	147 388	108 280	47 141	22 90	6 32	1 16
58	max	9 80	37 170	113 330	182 516	254 670	294 776	270 710	212 576	134 402	52 230	16 102	9 58
	min	3 28	6 20	37 118	82 277	134 344	175 460	146 388	108 280	44 132	19 83	5 27	1 14
59	max	8 70	34 160	110 320	179 508	251 668	294 773	260 709	208 580	130 390	48 229	13 94	8 88
	min	2 24	5 23	38 110	81 273	133 342	174 460	145 382	103 278	41 124	17 88	4 22	0 12
60	max	7 64	32 130	107 310	176 500	249 660	294 773	268 708	206 384	126 386	43 218	11 88	9 40
	min	2 20	4 20	33 108	79 270	132 340	176 460	144 380	100 270	38 119	15 78	3 17	0 10

Notes

1. Calculated from data published by the United States Weather Bureau.
2. Langley is the unit used to denote one gram calorie per square centimeter.
3. 'Visible' is for radiation of wave lengths of 4000Å to 7000Å penetrating a smooth water surface.
4. 'Total' is for radiation of all wave lengths in the solar spectrum.
5. Maxima are firm (will not normally be exceeded).
6. Minima are based on or extrapolated from lowest values observed for indicated month and latitude during 10 years of record.

Corrections

1. Approximate corrections for elevation up to 10,000 ft
   - a Total radiation: tot (1 + 0.0185 EL)
   - b Visible radiation: vis (1 + 0.00985 EL)

   Where EL is in thousands of feet

2. Correction for cloudiness (approximate):

   min + [(max—min)αL]

   Where αL is fraction of time weather is clear.

Algae Research Project
Sanitary Engineering Research Laboratory
Department of Engineering
University of California, Berkeley
June 15, 1954

Table B2 (24)

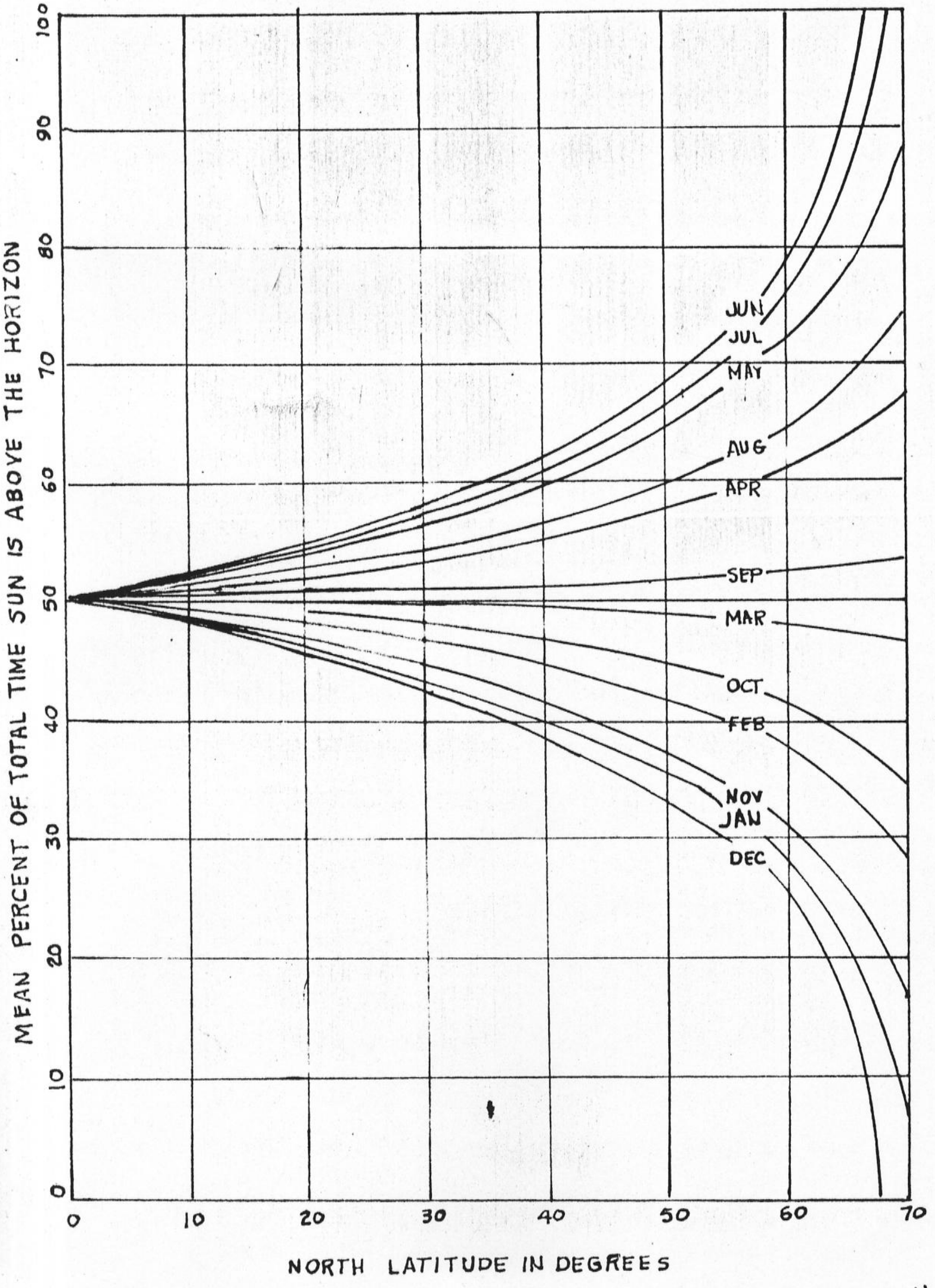

Fig B3 Mean Percent of Total Time Sun is Above Horizon. (24)

i) Obtain the total solar radiation and correct for elevation and cloudiness.

ii) Multiply the result by 10, and multiply also by the fraction of time the sun is in the sky. The latter is found from Fig. B 3.

2. G. Van R. Marais:

He had developed a rational theory for the design of sewage stabilization lagoons in tropical and subtropical areas. His theory is based on the monomolecular theory, that states that the rate of reaction at any one time t, is proportional to the concentration at that considered time. In equation form it is.

$$\frac{ds}{dt} \propto S$$

$$\text{or} \quad ds = -KSdt$$

Only two parameters of pollution, the B.O.D. and the faecal bacteria concentrations will be considered in the development of the following equation. Also it is to be noted that only the equations used in design are developed.

Assumptions that were made are:

a) Monomolecular law applies

$$ds \propto -S\,d\,t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{2-a}$$

or in equation form

$$ds = -KSdt$$

Where K is the monomolecular constant.

b) As the influent enters the pond, instantaneous and complete mixing of it with the pond content takes place. Thus the concentration of the pond content and the effluents are the same.

c) The effect in the concentration of the pond content and thus the effluent due to evaporation and percolation is very small and hence neglected.

For the derivation of the differential equation 2-a, the following terms that will be used, are defined:

t = Time expressed in days, from any arbitrary instant.

Q_1 = Influent Flow to the pond (first pond to be designed in any flow units at time t.

Q_2 = Effluent from the first pond to be designed, at any time t, expressed in the same flow units as Q_1.

V = Volume of the pond in the same units of volume use to express Q_1.

K = Monomolecular constant measured in log S - day units.

So = Concentration of pollution in the influent to the pond at that given time t expressed in ppm.

S = Concentration of pollution in the effluent or the pond content at time t, expressed in ppm.

Considering a small interval of time dt the influent flow Q_1 with concentration So will increase the pond content concentration by a value $= +\left(\frac{Q_1}{V}\right)$ Sodt.

By a similar manner, the effluent leaving the pond will decrease the concentration of the pond content by a value $= -\left(\frac{Q_2}{V}\right)$ S.dt

At the same time a decrease in S (effluent or pond content concentration) takes place due to the stabilization action of the bacteria that is equal to - KSdt.

So the net change in the pond concentration S in a short interval of time dt will be:

$$dS = \left(\frac{Q_1}{V}\right) \text{Sodt} - \left(\frac{Q_2}{V}\right) \text{Sdt} - \text{KSdt} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2\text{-b}$$

This can be written by dividing by dt, and arranging some terms

$$ds + \left(\frac{Q_2}{V} + K\right) S = \left(\frac{Q_1}{V}\right) \text{So} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2\text{-c}$$

With time, the concentration in the pond (S) will reach steady conditions and hence no change in concentration will take place.

Thus $\frac{ds}{dt} = 0$

Also the ratio of $\frac{V}{Q_1}$ is called the influent retention time (R_1), and $\frac{V}{Q_2}$ the effluent retention time (R_2).

Equation 2-c becomes:

$$\left(\frac{1}{R_2} + K\right) S = \frac{\text{So}}{R_1}$$

$$S = \frac{\frac{\text{So}}{R_1}}{(KR_1 + 1)\frac{1}{R_2}}$$

$$S = \frac{\text{So}}{(KR_2 + 1)\frac{R_1}{R_2}}$$

$$S = \frac{\text{So}}{KR_1 + \frac{R_1}{R_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2\text{-d}$$

In cases where the evaporation and percolation are small ($Q_1 = Q_2$), the retention times of the influent and effluent will be equal.

$R_1 = R_2$. This is often for short retention times.

Equation 2-d becomes:

$$S = \frac{So}{KR + 1} \quad \text{..................................} \quad 2\text{-e}$$

Or in cases where the retention time is large, the effluent (Q_2) will be reduced to a great extent, so R_2 becomes very big compared to R_1. Thus $\frac{R_1}{R_2}$ approaches zero and the equation becomes:

$$S = \frac{So}{KR_1} \quad \text{..................................} \quad 2\text{-f}$$

Experimental checking of the equation 2-d showed that the theory is applicable in determining faecal bacteria for a series of ponds. This equation on the other hand, is applicable to the first pond when dealing with B.O.D. For the design of the other ponds (secondary and tertiary), a retention period of seven days is enough. Based on this equation, results must be checked by Fig. B4 for minimum retention period and maximum allowable B.O.D. load per acre per day. Also the ratio of Area and depth must be equal or greater than 1000. Area and depth expressed in sq. ft. and ft. respectively.

The terms in equation 2-d are determined as follows:

- So: Found experimentally by running several B.O.D. tests and then taking their average, or by theoretical means by knowing the total persons contributing, the weight of B.O.D. each person contributes daily. The volume of water consumed by each every day is also needed.

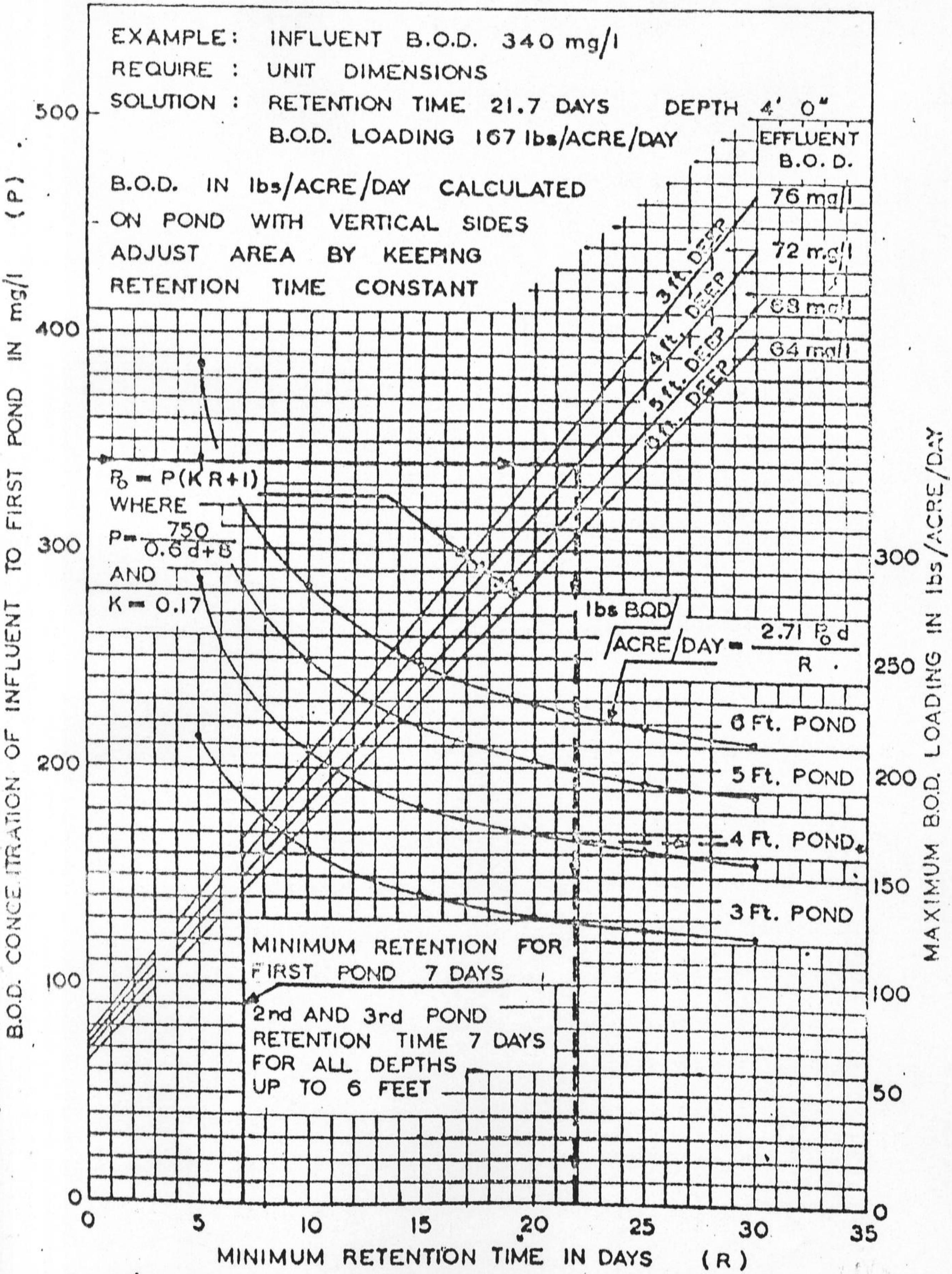

Figure 84. DESIGN CHART FOR 1ST. POND B.O.D. REDUCTION (25)

- Q_1 : Measured or assumed as the product of total persons by the daily water consumption of each.
- Q_2 : This equals to Q_1 less evaporation and percolation losses.
- K : 0.17; a conservative value given for tropical climates.

$$- S = \frac{1000}{0.6d + 8}, \qquad d = \text{depth of pond in feet}, \quad S = \text{ppm.}$$

The above equation was found by application of experimental results that insured total aerobic reaction in the pond. The value found above is the critical value above which anaerobic reaction will take place. For design purposes a safety factor is considered, and the equation becomes:

$$d = \frac{750}{0.6d + 8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 2\text{-g}$$

This equation resulted from experimental data on ponds 2-10 ft. deep, and should be used for pond depths in this range. Recommended depths range from 3-5 ft.

In applying these theories, it is noted that Oswald and Gotaas approached the solution theoretically and thus seems to be more valid. However, in my opinion and due to the many inter-related factors that are in turn affected by other assumed or approximated environmental factors, his method is highly theoretical and difficult to apply.

Marais's method which is a combination of theoretical and practical considerations, is simple, and more practical to use. Although some assumptions were made, yet experiments and actual ponds designed on the bases of this method have proved to be successful.

In the actual design of our project, the theory developed by Mr. Van Marais will be considered.

d. Factors Affecting the Process of Stabilization:

Actually many factors influence the operation of stabilization lagoons. Under this section, only the effects of recirculation, shock load and industrial wastes will be discussed, other factors are discussed within other different sections.

Recirculation: From 0.1 - 1.5 of the inflow may be useful in giving the influent a good amount of oxygen, as well as seeding it with algae. Thus reducing the problems of having anaerobic reactions. Recirculation has been shown to have little effect in increasing the B.O.D. removal, but was found to aid photosynthetic efficiency of ponds (27).

Shock loading: Studies have shown and proved that stabilization lagoons are capable of absorbing and treating shock loads. Shock loads with sewage concentration as high as 500% of the influent sewage, applied for short periods of time, were treated satisfactory without producing any trouble in the lagoons (9, 28).

Industrial Wastes: In general, it is very frequent to have industrial wastes mixed with domestic sewage; however, large industries are usually separated. In many cases, the combined treatment of domestic and industrial wastes offers both economical and technical advantages - less costly and more effective.

Few cases of industrial wastes are harmful to biological activity, and are to receive pretreatment or complete separation from the sewage (23). Such wastes that would have an inhibitive effect on bacteria and hence biological activity are dilute acids or alkalies which would change the pH away from optimum operating range. Wastes producing toxic matter, are to be completely separated from sewage flowing to lagoons. This is because toxic matter might stop or even destroy the action of bacteria and algae from one point, and might contaminate underground water from another.

The effect of some other wastes like milk wastes, and slaughter house wastes are beneficial to biological treatment in a lagoon, provided an account of their presence is considered in the design (23). Milk wastes have all the nutritional requirements necessary for biological stabilization. Slaughter house wastes are rich in nitrogenous matter that serves a good food for both algae and bacteria.

c. Construction Features:

A lot of development has been done with respect to many construction features of lagoons. In spite of this fact, many designers restrict themselves to the values they have found from their lagoons.

This diversity of opinion, is always the case when no rational theory is assigned. Every one uses a certain method, and if being successful, believes that his design is the best. Difficulty is encountered in choosing the criteria for our design, since each country has its own characteristics. The most recent and valid data is discussed down below for each type of feature.

i) Shape: Lagoons should be of such shape that there are no narrow or elongated portions. At corners, the dikes must be rounded, to minimize the accumulation of floating materials.

Square or rectangular (length twice the width) lagoons are most frequent.

ii) Dikes: Proper kind of soil and proper degree of compaction is to be used in order to minimize seepage losses. The top width should be adequate to permit access of maintenance vehicles. 8-10 ft. has been found to be sufficient for normal lagoons (29, 30). Exceptional units might demand a wider top width.

Dike surfaces and excavated embankments above the water line should be graded finely, and seeded with a short stemmed shallow rooted grass. Other form of vegetation should be removed.

Inside slope of 4-3: 1 are good for small ponds (15, 30). Although small slopes are good to minimize the erosion set by wave action, they are still not very much desired, since they require more area, and also will increase the shallow areas conductive to emergent vegetation.

iii) Bottom of the Lagoon: With respect to lagoon bottom many different authors suggest a different manner of construction; each depending on his personal experience. Many however, recommend a flat bottom free of any debris of vegetation. Enough compaction is to be given to lagoon's bottom to minimize seepage. A minimum of six inches of clay lining is to be applied at the bottom, with an average of 1 pound of bentonite clay per square foot is required (31). The cost of bentonite is about 20-50 cents per sq. yard. In many cases of hot and dry climates and porous soils; evaporation and percolation may present a problem. Asphaltic or rubber membranes may be used in place of the bentonite.

Joe Williamson (29) in his recent designs does not recommend a perfectly level lagoon bottom. As well, and during my discussion with Mr. Van Marais, he also does not feel a need for a level bottom, as long as no area is less than 3 ft. deep. Bottoms can be made to slope if felt necessary to go along with the land topography.

iv) Depth and Freeboard: Recommended depth varies from 3 feet as a minimum to 5 feet as a maximum (6, 30, 32, 33, 34). This range is governed by growth of aquatic weeds at low depths, and by the penetration of sunlight rays at higher depths (32). Sunlight is not effective below a depth of 5 ft., so no photosynthetic oxygen is generated. Hence a septic action is formed, and bad odors and nuisance will result.

Means of varying the depth from 3-5 ft. are very much desired. For instance, the depth in summer may be as low as 3 ft., while in winter can be as high as 5 ft. (14), in the cases where a thick layer of ice is formed.

A value of 3 feet is generally accepted as a minimum freeboard (15, 29, 32). Freeboards for larger lagoons must be studied according to height of wave developed. In smaller lagoons, less than 6 acres in area, a freeboard as low as 2 feet can be considered safe (15).

v) Multiple Units: The fact whether one single pond is better than a series of ponds with the same area is still not very clearly understood, proved, or explained. However, Marais (25) very recently has shown that a series of ponds with retention times equal to the retention time of a single pond will deliver an effluent that is superior in quality to that produced by a single pond.

So it is felt necessary that a minimum of two cells and preferably more act as the primary unit. This arrangement will facilitate parallel or series operation as the need arises. Facilities for such operation must be provided to insure flexibility of operation.

vi) Inlets: In general inlet structures are simple, but require much attention as to clogging and production of odors. They must provide a proper mixing and distribution of wastes, and must be designed to avoid any accumulation of sludge.

Multiple inlets have been proved to yield better results for large lagoons in uniformly distributing the entering wastes all through the lagoon area. Thus preventing any relative high concentration of loadings.

A variety of points governing the inlets are numerated below. This difference in opinion goes back to different authors in different countries.

a) Inlets are simply laid on the bottom of the lagoon. This method is not expensive and prevents the odors from raw sewage. Frequent flushing must be provided for such units.

b) In cases of no frost, inlets are to be elevated on the lagoon and thus discharging from the surface.

c) Influent pipes discharging vertically have been successful; but no advantage can be seen. Some say this type of inlet is no good, since accumulation of grit in the line is possible.

d) Submerged pipes, discharging horizontally require a complete splash to minimize erosion.

e) With regard to the position of discharge, inlets must discharge at or near the center. For small and square lagoons center discharge is reasonable. For rectangular lagoons, discharge point is to be at the two thirds point away from the outlet (6, 34). Such inlets will insure dispersion of the settleable solids throughout the lagoon (34).

In cases of large ponds, inlets might discharge within a distance of 200-400 feet from the embankment.

f) Mr. Svore of North Dakota (29) insists that the raw sewage be brought to the lagoons with a submerged inlet, thus avoiding any possible raw sewage odors.

vii) Outlets: Before attempting to design any outlet, a complete study of inflow, evaporation, rainfall and percolation rates should be made. In many cases, there is no need for installing any outlet. Many lagoons in North Dakota are satisfactorily operated on this basis.

Location of outlets must be at the periphery at the furthest distance from the inlet. Many types of outlets have been in use; each type suitable for the existing conditions.

a) Simple outlets with a pipe near the surface going to a manhole in the dike, from which an effluent pipe is then drawn.

b) A similar installation, but the pipe below the surface in countries where the surface freezes.

c) In cases of operating at different levels, a series of planks might be installed to permit operation at different depths.

viii) Miscellaneous Features:

a) Means should be provided to drain the pond completely.

b) Area to be completely surrounded with a fence. This prevents the access of unauthorized persons who might swim or drink from such sewage due to their ignorance of the resulting conditions. Also this method of fencing prevents the tresspassing of cattle and thus damaging the dikes.

c) Frequent signs in the lagoon area must be provided to designate the nature of the installation.

d) Flow measuring devices such as weirs are to be installed at all outlets and inlets of each pond.

f. Purification Efficiency Achieved by Waste Stabilization Lagoons:

The standard measures of efficiency in a lagoon or other sewage treatment plant is the degree of reduction in suspended solids, 5 days B.O.D. incubated at 20° C, and the bacterial count. Other forms such as nitrates, phosphates, carbonates and similar constituents of sewage are also measured; but are not as important as the above mentioned three points which will be discussed in what follows:

1) Reduction in Suspended Solids: A range of 50% - 80% reduction is suspended solids is given by many references. This reduction is mainly due to the settlement and flocculation of the solids in the pond.

2) Reduction in B.O.D.: Values reported on this subject varied a lot in relation to country, number of ponds, loading, and season. Values as high as 98.4% and as low as 43.6% have been reported (4, 31). Average values in B.O.D. reduction from 150 ppm to 20 ppm were given, for a detention period of 15 days. Table 3, gives efficiency values with variation in B.O.D. loading.

It is to be noted that effluents from primary units range in B.O.D. value between 60 and 80 ppm depending on how deep is the lagoon. This value can be found for lagoon between 2 and 10 feet in depth from the equation developed by Mr. Van Marais (Equation 2-g). For further reduction, secondary and tertiary units are to be installed. Exactly what takes place in secondary lagoon is not yet rationalized, however, values of effluents between 10-20 ppm are produced.

3) Reduction in Bacterial Count: Nearly in all the instances of operation, the bacterial count has been lowered to less than 1% of its original value (27). In general, the E coli are reduced from several hundred thousand to less than 100 per ml. At one time, typhoid bacteria were reduced from 41 per ml to negative results (27).

Reasons for this high reduction in bacterial count goes back to many theories. As stated by Caldwell (27), the bacterial count reduction is due to the liberation of toxic material by the algae present in ponds. Another theory says that the reduction is due to long storage and extreme competition.

It could also be believed that such reduction is a result of the high concentration of oxygen produced by algae during the day. As well, due to the relatively high pH values produced in the day time as a result of absorption of carbon dioxide by algae. Another factor may be the effect of the ultraviolet rays in the sunlight. It is still not known which of these theories is valid. It might be that the effect is due to a combination of causes. Regardless of the cause, high reductions in bacterial counts of about 99.9% have been reported (13, 18). Table 3 gives values of bacterial reduction for different degrees of loading.

Table B 3 - B.O.D. and Bacterial Count Reductions (18)

	POND NUMBER				
	1	2	3	4	5
Loading lbs of B.O.D. per acre per day	20	40	60	80	100
B.O.D. Reduction %					
Range	84-90	82-89	72-87	72-89	72-87
Average	87	86	82	81	82
Colliform Organisms in Effluent Average per ml.	147	247	243	340	402
% Samples 930 per ml.	0	7.9	8.9	18	19.2

g. Modifications and Best Conditions for a Lagoon:

The best conditions for a lagoon might not be all known to start with for a given country, however, general conditions that apply are as stated below.

a) Sunshine: Countries where sunshine is abundant are suitable for the development of stabilization lagoons. Countries like England, where direct sunshine is infrequent, waste stabilization lagoons are not recommended. The energy requirements is a minimum of 1.5-2 langleys per day on a monthly bases for each pound of B.O.D. applied per acre.

Due consideration must be given to the maximum utilization of the incident solar energy in designing stabilization lagoons. High freeboards will decrease the number of direct sunshine hours on the water level. Obstacles near the lagoon such as trees might provide shade and thus less sunshine.

2. <u>Consistent Breeze</u>: Wind has an effect on the lagoon's efficiency. It mixes the pond contents and supplies oxygen to the surface layers.

Excessive wind speeds are harmful to the banks, since they produce high waves. In general, average values of moderate speed are felt necessary.

3. <u>Temperature</u>: Has a big effect in the speed of reactions taking place in a pond, as it has an effect on other chemical and biological reactions.

Temperatures as high as 40° C are ideal for a good efficient pond. Tropical climates are the best for lagoons. In cold climates, lagoons designed with a lower loading, have also showed success.

4. <u>Clay Areas</u>: Wide open areas of clay or other water holding material is perfect for lagoons. This point is clear since we are interested to keep a constant depth of water in the pond.

5. <u>Evaporation Rate</u>: High evaporation rate to the extent of appreciably decreasing the water depth are not desirable. In such places, care must be followed to insure no overdesign. A sufficient water flow, and a relative small surface area are to be adopted in such cases.

h. <u>Operation and Maintenance</u>:

The required maintenance for a waste stabilization lagoon is rather simple; but very important since it governs the success or failure of such installation.

As compared to other treatment units, lagoons are much more cheaper to maintain (13). One unskilled laborer can do the job for a small lagoon. An average value for the cost of maintenance and operation is given as $250 - $500 per year for ponds less than 10 acres in area (6).

Reasons for this low cost, and simplicity in maintenance go back to the following:

1) Maintenance problems related to mechanized systems are absent except in some special cases of land topography. There are no pumps, and such mechanical equipment to give rise to a high maintenance cost.

2) Sludge removal is not required in lagoons as it is very necessary and very troublesome in other systems. No case of sludge production had so far given a real problem. Imhoff and Fair (33), gave a figure for the digested sludge produced annually per 1000 persons as 3212 cu.ft. In other words, for an area of 10 acres, a time of 135 years will be required for the building up of one foot layer of sludge at the bottom of the lagoon.

3) Personnel required are very little, and in many cases one is enough.

The major operation features that are highly necessary consist of:

1) Mechanical Units: In the presence of any pumps or other mechanically or manually operated units, frequent checks are to be done as felt necessary and as set by the manufacturer or designer. Careful valve operation is to be considered in order to insure that the proper flows in and out of each pond takes place.

2) Loadings: An important operating feature is to control the loading conditions and retention time set by the designer. This can be insured by operating the pond at the constant depth set by the designer. Any change in such rules, might yield a septic action in the pond. If any such action of increasing the loading and thus septic action, sodium nitrate is to be added to overcome such septic action. Sodium nitrate ($NaNO_3$) is used because it is an oxidizing agent, and the nitrate radicol (NO_3^-) is a good food for the algae. Both mentioned points help the return to good operating conditions. Sodium nitrate dose to satisfy the above condition is from 5% - 15% of influent B.O.D. load (9, 28).

Frequent disposal of high loadings not considered in the design such as nightsoil are to be strictly prohibited to go into the lagoon especially all of a sudden. Allowance might be made, if very necessary, for the disposal of such wastes on condition they are gradually disposed into the lagoon.

3) Flexibility in operation of the ponds in either series or parallel should be insured.

4) Quality of the influent and effluent should be measured frequently by proper sampling and testing. The main tests to be carried out include the B.O.D., the bacterial count, and the solids (settleable and suspended).

5) With regards to the influent, a proper method for its disposal is to be considered. It may be used for irrigation or disposed into a nearby stream or spread on the ground with a proper system.

There is no real need for daily maintenance as is the case in conventional treatment units. However in many cases when a special person is assigned for the lagoon maintenance, it is advisable if he runs the following checks and controls daily.

<u>Erosion of pond banks</u>: This is very often the case in areas of high wind speed, and high rainfall intensity. In large ponds, wind might be enough to form high destructive waves. Any evidence of erosion or deterioration of the banks should be repaired immediately to avoid any possibility of failure of the dikes and consequently the pond. Also it is to be noted that minor repairs at an early stage are of course much less expensive than repairing the whole dike or even the whole pond.

Good methods for the protection of dikes have been used by many designers. Some plant grass on the sloping sides of the dikes while others use a concrete mat over the slopes. The latter method is much more effective in preventing failure due to wind, and also decrease the seepage losses into the embankment and hence out of the pond; but is more expensive to construct, although cheaper to maintain. A still good method to use, a mean of the above two methods, is to put a concrete mat little above and below the water level, and plant the upper parts by grass.

<u>Vegetation</u>: Emergent plants in waste stabilization lagoons or oxidation ponds must be controlled by maintaining a minimum uniform depth of 18-24 inches. A depth as great as 3 feet is recommended. Vegetation near the periphery and the shore line is to be controlled. These types of plants interfere with the action of sunlight and wind (10).

Additional trouble is brought when these plants die and thus increase the organic load in the pond. It must not be forgotten too that these plants give rise to mosquito breeding.

<u>Mosquito Control</u>: Mosquitoes are rarely found in good and well maintained lagoons. Lagoons with a lot of weeds and vegetation are abundant with mosquitoes. It has been shown - experience wise, that mosquitoes will grow best in calm areas where wind and wave action are absent. Such conditions are set by the growing vegetation. There is no real need to control mosquitoes, as long as a good control of vegetation is available (10).

Insecticides might be used but it was stated (35) that their use is not recommended because they are expensive and might interfere with the biological activity that is responsible for the waste stabilization.

<u>Floating Matter</u>: Floating matter such as masses of decomposing solids and dead algae that accumulate in corners are to be broken up as promptly as possible, or dumped away after accumulation.

These are important to remove because they make an ugly site and worse than this, they decompose anaerobically thus producing bad odors.

<u>Burrowing Animals</u>: Bank rats, and snakes are such animals that undermine and ultimately break down the dike structure (31). Immediate repairs should be done to the dikes, and control methods which are mainly by trapping are to be insured and followed.

<u>Mechanical Units</u>: Maintenance of all mechanical units if any, is to be set and run according to the manufacturer's recommendations. Inlets, and outlets should be clean always. In cases of clogging, control of the depth must be insured or else erosion of the dikes might take place. A safety outlet, that is used for dewatering the pond is to be used in such emergency.

i. <u>Algae - Growth and Recovery</u>:

A good definition of algae stated at the conference in Illinois (29) in the year 1956 states that algae is a microscopic single-cell plant. It is one of the earth's early elementary forms of life. It can be eaten (animal food); it contains protein, fat, starch, vitamins and every other food component required to sustain life.

As mentioned earlier in this chapter, there are many different species of algae. Most active of these in stabilization of sewage are, Chlorella, Euglena, Scenedesmus and Chlamydomonos (20, 21).

Each of these cells of algae can be considered as a factory by itself. It absorbs carbon dioxide gas either from the atmosphere, or that dissolved in water through its body wall and takes non-living chemicals such as nitrogen and phosphorous from the surroundings. Being plants, they absorb the sunlight energy and by the help of the chlorophyl they contain, convert these non-living organic substances to proteins, fats and carbohydrates in the synthesis of new algae cells. In addition to synthesis, they liberate oxygen as a by-product.

As to the growth of algae, the yield of algae per unit area increases directly with the loading of organic matter to a limit where sunlight energy becomes the limiting factor. An annual algae yield of about 40 tons per acre of area is reasonable (29).

The extent that algae will live in cold climates is not yet known, however, the growth is small at freezing temperature (22). For the proper growth of algae, enough quantity of nitrogen is to be available in the sewage. From 8% - 10% of the dry weight of algae is nitrogen (9, 21).

Algae grow most in the upper surfaces of the pond where light energy is maximum, and decrease as depth increases.

The concentration of dissolved oxygen produced by algae can be as high as 39.2 ppm, at 25° C (9), which serves a high degree of oxidation. The use of algae to produce oxygen is not restricted in ponds, but is thought of to remove carbon dioxide and produce oxygen in the new atomic submarines.

Being restricted to lagoons, the algae living in great concentrations in the lagoon can be recovered and used as food for many animals (29). Thus stabilization lagoons serve a dual purpose, they treat the sewage, and reclaim the organic matter in the sewage to an animal food. So far, the recovery of algae is not yet economically solved, although many methods are in use.

Methods for harvesting the algae are many, some of which include settling, filtration, centrifugation and flocculation (36) as will be discussed hereunder.

Settling did not show good results in concentrating algae for harvesting. This goes back to the long detention time needed for settling, and the poor quality of the supernatant obtained.

Separation of algae from the effluents may also be done by filtration through a deep layer of fine sand. This method is actually not economical. Also vacuum filtration up to now is not economically feasible.

Centrifugation has shown good results in the separation of algae from the effluent-treated sewage.

Flocculation carried through by adding lime to raise the pH to 11.3 gave excellent results. This method has the advantage of low operational cost, and the disadvantage of producing an effluent with a high pH. Another disadvantage to this method may be shown in the following paragraph.

Although chicken tolerate as much as 25% of some samples of algae in a meal, this value is reduced to about 10% due to the presence of aluminum produced by flocculation procedures (37).

A value of \$35 - \$40 per ton or more may be spent for harvesting and processing the algae to a condition suitable for feed stuff (36).

In the cases where the algae is not recovered, it can be left in the effluent with no real harm. In any case, the algae concentration in the secondary lagoons is reduced a lot, and if tertiary units are supplied, the value will be even smaller. Any algae left in the effluent will either serve as a food for fish in streams, or might serve for further purification in the stream itself if conditions are favorable. A third possibility is that algae die slowly thus producing a tolerable condition in the stream.

APPENDIX C.

Supplementary Data.

Table C1. Meteorological Data

Average Monthly Values (1953 - 1962) - KSARA OBSERVATORY

MONTH	Jan.	Feb.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Temp.* C°	5.9	6.9	8.3	12.4	16.9	21.2	23.0	24.2	20.9	13.9	11.6	7.0	—
Rainfall* m.m.	124.3	83.6	47.0	22.6	11.8	0.5	0	0	0.1	6.2	52.9	115.0	463.2
Sunlight Intensity Langleys **	212.0	282.4	414.5	539.4	650.0	747.0	739.0	667.0	567.0	413.1	281.0	196.0	—
Evaporation$ m.m.	67.6 (44.0)	71.1 (46.1)	124.2 (80.6)	176.1 (114.2)	243.0 (158.0)	325.0 (211.0)	376.0 (244.0)	388.0 (255.0)	295.0 (191.0)	260.0 (169.0)	125.0 (81.0)	70.0 (45.5)	2523.0 (1636.4)

*. Values at Housh Sneid (FARM) - Elevation 995 m. above sealevel.

**. Values at Ksara Observatory - Elevation 920 m. above Sealevel.

$$. Values at Riyak – Elevation 931 m. above sealevel.

N. B

1. Correction factor for evaporation is 0.65. Corrected values are placed in parenthesis.

2. Direction of the wind at the farm is NE. Wind speed is about 3 m. per second.

3. 1 Langley = 1 calorie per sq. cm.

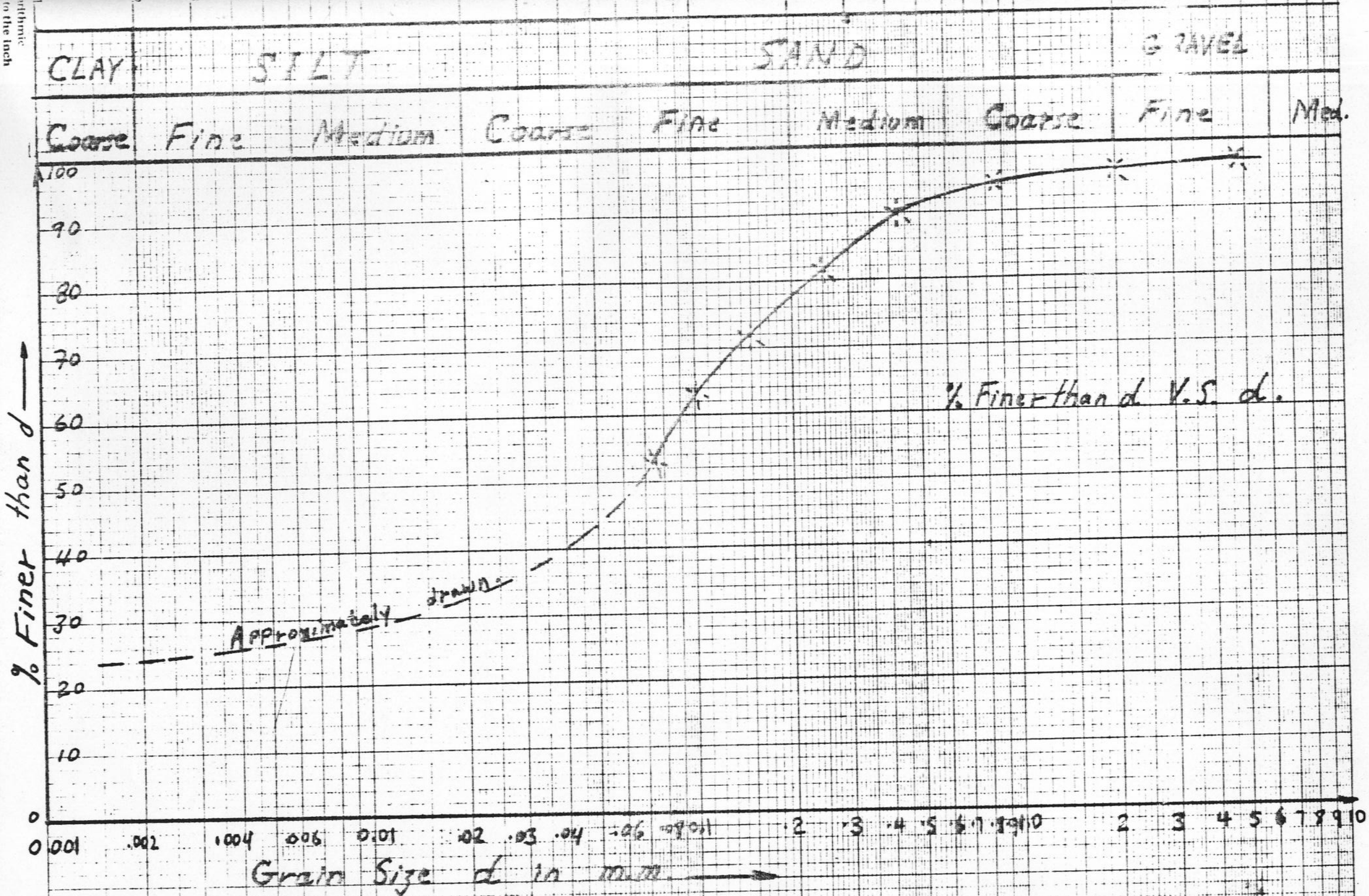

Fig. C1 GRAIN SIZE DISTRIBUTION For A.A.E. FARM SOIL

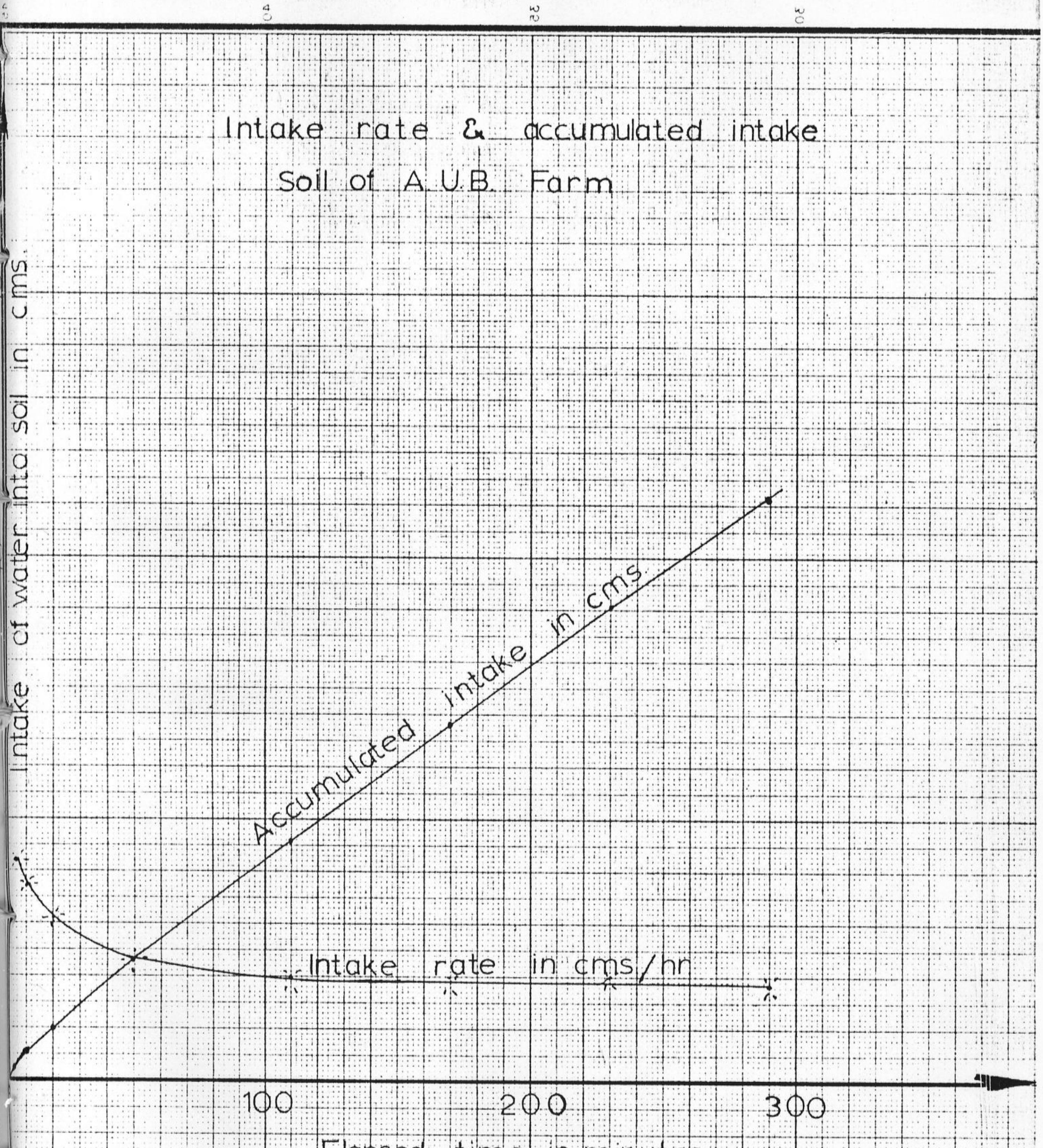

Fig. G2 (Given by Prof. Maksoud.)

Table C2 . Chemical Analysis of Farm Water

Chemical Test	Tests Performed		Results by Prof.
	14-JUNE-1963	31-JULY-1963	S. Maksoud
Total Hardness as $CaCO_3$ - p.p.m.	144	142	—
Calcium - p.p.m.	—	—	65.0 ± 10.0
Magnesium - p.p.m.	—	—	10.7 ± 1.8
Sulphates - p.p.m.	—	—	24.0 ± 9.6
Chlorides - p.p.m.	15.0	7.1	14.2 ± 2.1
Ammonium - p.p.m.	Negative.	0 < Value < .05	Negative.
Nitrite - p.p.m.	Negative.	.001 < Value < .004	Negative.
Potasium - p.p.m.	—	—	0.58 ± 0.20
Sodium - p.p.m.	—	—	0.70 ± 0.12

Table C3. Chemical Analysis of Farm Sewage.

TRIAL →	I * (31-JULY-1963)							II ‡ (4-SEPTEMBER-1963)				
TEST →	Acidity	Alkalinity	NITROGEN		B.O.D	SOLIDS		B.O.D	SOLIDS		Acidity	Alkalinity
SOURCE ↓	p.p.m.	p.p.m.	Ammonia p.p.m.	Nitrite p.p.m.	p.p.m.	Settleable p.p.m.	Volatile p.p.m.	p.p.m.	Settleable p.p.m.	Volatile p.p.m.	p.p.m.	p.p.m.
S1	10	210	—·—	0	100	600	160	—	—	—	—	—
S2	—	—	—	—	—	—	—	370 (278)	560 (420)	360 (270)	64	250
S3	22	224	—·—	2.2	80	350	120	170 (128)	380 (285)	110 (83)	20	190
S4	30	208	—·—	·05	300	680	500	1040 (780)	990 (743)	920 (690)	120	180
ALL MIXED ‡	—	—	—	—	—	—	—	380 (285)	615 (460)	328 (245)	40	198

*. Sample was taken at one time (11:00 A.M.). At this time the B.O.D Production was minimum. Hence Values were not used.

† Sample was taken for a period of 8 hours (6:00 A.M. - 2:00 P.M.). Corrected Values are placed in parenthesis.

‡ Ratio was 2:9:3. as given in Table 1

— No sample could be obtained.

—·— Values obtained were very small

N.B. pH ranged from 6.5–8.0, and that of the mixture is 7.2

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<u>NOTE</u>: Meteorological Data was Supplied by Ksara Observatory, Ksara, Zahle, Lebanon.