PRELIMINARY DESIGN

OF

SEWERAGE SYSTEM FOR HEBRON, JORDAN

By

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A Thesis

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the Degree of Master of Engineering
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Saiyed Hasan Akhtar

TABLE OF CONTENTS

LIST OF TABLES

LIST OF DRAWINGS

SYNOPSIS

The design of a sewerage system for the town of Hebron in Jordan has been undertaken in partial fulfillment of the requirements of the degree of Master of Engineering with major in Sanitary Engineering.

Hebron possesses the problem of having old and modern buildings with narrow streets in most part of the town. It has at present a population of 38,000 persons with different population densities in different areas. The population is increasing at a faster rate and it is anticipated that within the next fifty years it would be doubled. The water supply system is inadequate and provides 30 litres per capita per day only. A new water supply system was prepared by "Brown Engineers International" in 1960 at the rate of 75 l.p.c.p.d. A new distribution network at this rate has been designed by the Central Water Authority and is expected to be executed soon. The town has no proper sewerage system and sewage disposal facilities. The Associated Consulting Engineers, Beirut, were asked by the Municipal Committee, Hebron, to make a preliminary

study for the sewerage system in June 1964. The study has been made by the said firm and report has also been submitted. Fortunately the author of this thesis was associated with the preliminary study of the project while he was under training with the Associated Consulting Engineers in summer 1964. The present study is, therefore, based on the work done by the author for the preliminary study.

The town has a fairly steep topography and, therefore, the sewerage system of the town has been made on gravity flow basis. As the storm water easily collects in the valley and can be disposed of in local depressions out of the town without causing any nuisance, a separate system of sewers for the sanitary and storm water has been designed. This is economical as well as ideal from maintenance point of view.

The per capita water consumption being very low, the sewage is highly concentrated and has a BOD of 2000 p.p.m. It is expected to improve to 975 p.p.m. with the increase in water consumption from 30 l.p.c.p.d. to 75 l.p.c.p.d. An efficient sewage treatment plant which can handle the high concentrated sewage for a population of 75,000 persons at a low operating cost is therefore required. Since the town does not have a proximity to any body of water in which the final effluent may be disposed of, the disposal of the effluent is necessarily to be made on land, and the BOD of the effluent sufficiently reduced so as to make it inoffensive. A conventional treatment plant comprising of grit channels with bar racks, mechanically cleaned primary sedimentation tanks, high rate trickling filters with rotary distributors and final sedimentation tank has been proposed. The sludge will be digested in sludge digestion tank and dried on sludge drying beds. It is proposed to reduce the BOD of the effluent to 50 p.p.m. and utilize it for irrigation of crops as such lands are available nearby. The treatment plant has been proposed at a sufficient distance away from the town (i.e. 2200 meters) so that the odours, if any, from the trickling filter will not reach the town.

The design of the treatment plant units have been worked out for the present population as well as for the anticipated population so that only the required units are constructed initially. This will not place any undue financial burden on the present population as well as forestall poor operating results in the early years because of otherwise oversized units.

The design of the sewers has, however, been made for the anticipated population since the capacity of these cannot be increased. Moreover, the minimum size of the sewer has been adopted as 8 inches which is sufficient for all the main sewers due to the high slopes.

The drawings for the L-sections of all main sewers and the trunk sewer as well as for all the units of the treatment plant have been drawn and are enclosed in the Appendix. The drawing for the layout of the treatment plant, the location of the treatment and the hydraulic profile have also been prepared.

The approximate quantities for the work to be done has been worked out and the cost determined. The whole project is estimated to cost about 379,000 Jordanian Dinars (equivalent to $ 1,010,400) for a population of 75,000 persons, or $ 13.50 per head for the total anticipated population.

CHAPTER I

INTRODUCTION

(a) Historical Background

Hebron is one of the oldest cities of the world. Historians state that it was built in 1700 B.C. and that David governed the city for more than seven years. Later it was burned by the Romans, destroyed by the Persians, and when conquered by the Moslems, it became one of the four sacred cities of Islam and again knew great prosperity. Then came the crusaders and was retaken by Salahuddin in the year 1187, and finally occupied by the Allies in 1917. Today it is the seat of government of the southern district of Jordan's West Bank.

The Prophet Abraham, as mentioned in the Bible, lived in Hebron near the Oak Tree which is known by his name even today. The chief monument of the town is the Mosque of Abraham, still bearing its Arabic name of Haram-al-Khalil, and it is the main attraction for the tourists. Other important touristic and religious features are the Ramah mosque and the Russian church.

The position of Hebron before the year 1948 on main road connecting Jerusalem with Beer Sheba, Gaza and

Cairo made it a main touristic and commercial centre and a pilgrimage station. An indication of the importance of Hebron as a pilgrimage station was the construction by the Sultans of a large pool of about 30,000 cubic meters capacity for the pilgrims to drink water on their way to Hijaz.

(b) Geographical Location

The city of Hebron is located at a distance of 44 kilometers southwest of Jerusalem, and 35 kilometers south of Bethlehem. The asphalted road connecting all these cities passes through rich and rolling country planted with olives and grapes, stands of government-planted pine, orchards of applfes, pears and almonds.

The city lies between an elevation of 850-900 meters above seal level and covers an area of about 3000 dunums.

(c) Living Quarters

The city of Hebron consists of mixture of old and modern buildings. The old city which is disttinguished by their narrow roads, covered alleys and joint dwellings is thickly populated. It has 75 percent of the population in only one-fourth of the area of the city. The modern buildings are scattered in the north-western part of the city. These are surrounded by gardens and connected by wide roads.

(d) Topography

The centre of the city is crossed by a wide valley at which the north-eastern and south-western hills meet. The valley forms the natural water course for the storm water flows as shown in drawing No. 3.

The Hebron municipal area consists of mesozoic rocks which are composed of thin layered lime stone separated by semi-pervious strata. There are faults and cracks through which the seepage of water into the ground may take place.

(e) Climate

Hebron has a moderate climate and relatively low humidity. The average temperature does not exceed 30 degrees centigrade in summer and does not go below 5 degrees centigrade in the winter. The average annual rainfall is about 600 millimeters.

The prevailing winds are western and south-western.

(f) Water Consumption - Present and Future

The city is at present supplied water from the Fawwar well, situated at a distance of eight kilometers southwest of the city with a maximum capacity of 1250 cubic meters per day. Water is at present pumped from this well to the Abu Nuseir Reservoir through a 5-inch and 6-inch pipe which has been proposed to be replaced soon by a 10-inch line. The capacity of the Abu Nuseir Reservoir is 600 cubic meters and its elevation is 965 meters above sea level. This reservoir supplies water to a greater part of the city by gravity. The higher area on the north of the town is supplied water through another reservoir of 600 cubic meters capacity and 985 meter elevation.

Other water resources comprise private deep wells and raw water reservoirs, which are difficult to estimate. However, they are only a few and they do not contribute an appreciable quantity.

The present per capita consumption based on municipal supply of 1250 cubic meters per day after deducting net work losses is thus 30 litres per capita per day.

According to the report of "Brown Engineers International"[1] and the information supplied by the municipality it is believed that the future water consumption would rise to about 75 litres per capita per day. A new distribution network has also been designed at this rate of water consumption by the Central Water Authority Jordan, the execution of which is expected to be started soon.

The future water consumtion rate has therefore been taken as 75 litres per capita per day.

(g) Existing Sewage Disposal Facilities

The existing sewage disposal facilities serve only the old southern part of the city in the northern part, the

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[1] "Brown Engineers' International" were asked by the municipal committee to prepare a study of the water supply of the city. The report was presented in the year 1960.

disposal method consists of private septic tanks or seepage pits.

From the sanitary and engineering point of view most of the existing net work is not suitable. The slopes are inadequate and lack of maintenance has resulted in accummulation of stones and dirt in the sewers. Some of the sewers pass underneath the existing houses and make the maintenance more difficult. A greater part of the existing sewers are broken and uncovered which causes the spread of mosquitos, flies etc., and therefore unhealthy conditions. The dirty water from these sewers is used by some of the inhabitants for irrigating vegetables which helps further in spread of diseases such as typhoid and dysentry.

For the above reasons, the existing network is recommended to be disregarded and a new well designed sanitary sewerage system which is the subject of this study is to be provided.

(h) Industries

There are few industries in Hebron which are very small. Those worth mention include two tanneries, a tomato canning industry, and a slaughter house. The flow from these industries is very small and is not likely to affect the design.

CHAPTER II

POPULATION ESTIMATES - PRESENT & FUTURE

One of the principal factors for determining the estimates of the quantity of dry weather flow is the present and the future population. It is an obvious fact that as the population increases, the use of water and the disposal of used water increases. Thus in order to plan properly for the disposal of used water, sanitary engineers must be aware of present as well as expected future population.

(a) <u>Design Period</u>

For determining the expected future population, a decision is to be made as to the length of time or period the improvement should be made to serve the community before it is abandoned or enlarged by reasons of inadequacy. This decision is known as period of design or design period, and depends on the following factors:

(a) Probable growth of the community.

(b) The useful life of structure and equipment employed.

(c) The initial cost of the work and available funds.

(d) The difficulty of relieving the system when it becomes over taxed and the inconvenience to the public caused by the construction of the sewers in the streets.

(e) The carrying charges of the sewers having surplus capacity and the difficulty of maintenance due to the small flows in large sewers till the system is not loaded to capacity.

(f) Water consumption at the end of design period.

Of the above factors, the probable growth of the community is the most variable one. It depends on many factors such as degree of industrialization, availability of transportation facilities, local zoning ordinances, areas available for development, initiative shown by city agencies in planning for future developments and in attracting new industries, national trends in both birth and mortality, shifts of population between urban and rural communities, and installation of sewerage system or water supply facilities.

The other factors are also difficult to be predicted accurately. Thus the estimates of the future size and the requirements are attended by uncertanity and it is usually not advisable to predict designs upon estimates of conditions which may exist for more than 30 to 50 years.

A design period for 50 years has, been adopted for the city of Hebron for the following reasons:

1. The city of Hebron being an old and densely populated, is believed to reach saturation within fifty years.

The further increase in population, if any, after 50 years will not be accommodated within the existing municipal limits, but an outside development will be required for it.

2. The saving, if any, due to short design period will not be appreciable as there will already be sufficient reduction in the sizes of the sewers due to high slopping grounds.

3. The period of 50 years is reasonable as useful life span period for most of the materials which will be used in the work.

4. Small flows in the initial stages will not reduce the velocity beyond the minimum allowable, as the favourable slopes are available.

(b) Source of Data

The primary source of population data are the past census records. These are available only for the year 1961 when the first census of the town was made. The population of Hebron according to this census is 37,869 persons. Prior to this, the census was made districtwise in the year 1952 which gives the population as shown in table No. I. The figures for the district of Hebron are as under:

1952	125,651 persons
1961	119,861 persons

Table I

GROWTH AND DISTRIBUTION OF POPULATION IN JORDAN

1952 and 1961

Administrative Area	Population in 1952	Population in 1961	% age Increase in 1961 Over 1952
Whole Country	1,329,174	1,691,123	+ 27
Amman District	218,465	414,541	+ 103
Balqa District	64,926	70,431	+ 8
Ajlun District	213,877	271,822	+ 27
Karak District	60,556	65,916	+ 9
Ma'an District	125,651	119,861	- 5
Jerusalem District	301,402	343,964	+ 14
Bethlehem Sub-Div.	56,677	56,344	- 1
Jerusalem Sub-Div.	85,550	110,005	+ 29
Ramalah Sub-Div.	110,076	113,691	+ 3
Jericho Sub-Div.	40,099	63,924	+ 30
Nablus District	315,236	342,155	+ 12
WEST BANK	742,289	805,980	+ 9
EAST BANK	586,885	885,143	+ 51

Data obtained from the Preliminary Report for Sewerage of Hebron Town made by the Associated Consulting Engineers, Beirut, Lebanon.

These figures show that the population in the district has decreased at the rate of 590 persons per year, whereas local authorities and residents of Hebron city think otherwise. The studies were, therefore, made by taking the local statistics for 38 families from different quarters of the city.[1] The results of these counts as entered in table No. 2 show that the number of persons in the 38 families have increased from 222 to 274 within the last ten years. These families being chosen from different quarters, and different standards of living are assumed to be representative for the whole city.

The statistics were also taken for the death and birth rate in the town, and the number of students in the school, and are shown in tables No. 3 and 4. These figures also shown an increase in population.

Thus the decrease of population in the district of Hebron is not attributed to the decrease in the population of the city but to the small villages due to the following reasons.

1. Immigration of population from villages to big towns like Hebron, Amman, Jerusalem, for want of jobs,

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1. The statistics were collected by the engineers of the Associated Consulting Engineers Beirut for making a preliminary study of the project.

TABLE No. 2

TABLE SHOWING NUMBER OF PERSONS IN FAMILIES IN THE YEARS 1954 AND 1963[1]

S.No.	No. of members in year 1954	No. of members in year 1963	Births	Deaths	Immigrat.
1	7	8	2	-	1
2	7	4	-	1	2
3	5	5	-	-	-
4	9	13	4	-	-
5	6	6	2	-	2
6	8	4	-	-	4
7	-	3	1	-	-
8	3	10	7	-	-
9	7	10	3	-	-
10	6	11	5	-	-
11	8	8	2	-	2
12	9	7	-	-	2
13	8	10	3	1	-
14	2	4	2	-	-
15	5	9	4	-	-
16	6	6	-	-	-
17	5	7	2	-	-
18	2	6	4	-	-
19	2	10	8	-	-
20	10	20	10	-	-
21	3	6	3	-	-

Table 2 continued

S.No.	No. of members in year 1954	No. of members in year 1963	Births	Deaths	Immigration
22	10	2	-	-	8
23	4	8	4	-	-
24	2	2	-	-	-
25	3	3	-	-	-
26	11	18	7	-	-
27	9	12	5	1	1
28	10	12	2	-	-
29	3	3	1	1	-
30	7	7	-	-	-
31	8	5	-	1	2
32	5	7	2	-	-
33	3	5	2	-	-
34	3	5	3	1	-
35	5	6	1	-	-
36	5	6	1	-	-
37	8	4	-	1	3
38	8	12	4	-	-
Total	222	284	94	71	27

Average Number of persons per family $= \frac{284}{38}$

$= 7$

1. Data obtained from the Preliminary Report for Sewerage of Hebron Town made by the Associated Consulting Engineers, Beirut.

TABLE No. 3

TABLE SHOWING NUMBER OF BIRTHS AND DEATHS IN AL-KHALIL MUNICIPAL AREA[2]

Year	Number of Births	Number of Deaths	Increase
1954	1763	504	1259
1955	1873	647	1226
1956	1878	416	1462
1957	1920	529	1391
1958	2063	488	1675
1959	1829	436	1393
1960	2303	316	1987
1961	1950	483	1467
1962	2562	385	2177
1963	2376	404	1972
Total	20,517	4508	16,009

Number of births per year = 2052 persons

Number of deaths per year = 451 persons

Increase in population per year = 1601 persons

1. *Ibid.*

TABLE No. 4

TABLE SHOWING THE NUMBER OF SCHOOL GOING CHILDREN IN THE CITY OF HEBRON[1]

Academic Year	No. of Boys	No. of Girls
1954-55	4249	2030
1955-56	4545	2232
1956-57	4929	2462
1957-58	5123	2586
1958-59	4928	2631
1959-60	4865	2674
1960-61	4804	2631
1961-62	4996	2741
1962-63	5207	2872
1963-64	5471	3082

1. Ibid.

education and other amenities of life which are usually not available in smaller places.

2. The available agricultural resources, which are not enough to satisfy the need of the people.

(c) Present Population

The present or the current population is in itself a population projection from the last census.

Because the net population change during any single year usually constitutes only a small percentage of the total population, it is frequently satisfactory when estimating the population for the first two or three years following a census, to add the census year population a sum equivalent to the estimated population increase during the equivalent time period immediately preceding the census. Since only one census figure is available for the city, the population increase preceding the census cannot be estimated, and the method can not therefore be applied.

The other methods such as the "School Attendance" method which is based on the records of number and age of children born prior to, and since the last census and the "apportionment method," which is based on the assumption that the city shares the same ratio in the increase or decrease of state population as it did in the previous year or years, are also not helpful due to the non availability of records.

The present population cannot therefore be estimated accurately by any of the methods. However, since the net population change during the short period of three years after the census will not be appreciable, and the design of works are to be made for the population after fifty years, the present population can justifiably be taken as 38,000 for the design.

(d) Future Population

There are many forecasting techniques which are used by sanitary engineers for predicting the future population, but each of these require sufficient past records of population which are not available in the event of circumstances, therefore, the use of V.B. Stanbery's suggestion that in the absence of data and analysis, the arithmetic and geometric projections might be used as probable minimum and maximum forecast has been made.[1] The population for the past year of 1951 has been estimated with the use of local statistics and the curves for the fixed yearly increase and compound yearly increase have been plotted. Since these curves are based on a constant rate of increase, and do not lead to an accurate result, they have been modified according

1. E.M. Frederick "Population Forecasting by Sanitary Engineers" Journal of the Sanitary Engineering Division, A.S.C.E., Vol. 90, August 1964, p. 44.

to the logistic method. The rate of increase in population while reaching the saturation has been progressively slowed down by assuming that the decline of population after the high rate will start in the year 1980. The following three graphs based on different statistics have been drawn so that a justified figure may be ascertained.

(i) <u>Increase of Population in Relation to Local Statistics</u>

According to the local counts for 38 families in the different quarters of the city as shown in table No. 2 the population of Hebron in 1954 is estimated as under:

Population of the families in 1964	= 274 persons
Population of the families in 1954	= 224 persons
Population of Hebron in 1964	= 38,000 persons
Therefore population of Hebron in 1954	$= \frac{222}{274} \times 38,000$
	= 31,000 persons

Linear annual increase = 700 persons

Compound annual increase = 2.1%.

On this basis the graph No. 1 has been drawn. It will be seen that the population at the end of design period varies between 67,000 and 80,000 inhabitants.

(ii) <u>Increase of Population in Relation to Increase in Number of Students</u>

Table No. 4 shows that the number of students in the school have increased from 6279 to 8553 during the last ten years. This gives a rate of increase of 2274 students. Applying this rate of increase to the whole population, the population in 1954 is worked out as:

$$\frac{6279}{8553} \times 38,000 = 27800 \text{ persons}$$

Linear annual increase = 1020 persons

Compound annual growth = 3.2%

Based on this figure the graph No. 3 was drawn which shows the population at the end of design period to vary between 75,000 and 88,000.

This gives a little high figure which is due to the fact that the education is increasing at a higher rate than the population especially in under-developed countries.

(iii) Increase of Population in Relation to Increase in Population in Jordan

The table No. 1 shows that the population of Jordan has increased from 1,329,174 to 1691,123, thus giving a linear annual increase of 36,200 persons. The compound annual growth is worked out as 2.4%. Applying these rates, the graph No. 3 is drawn which shows the population to vary between 69,000 and 82,000.

It is concluded from all the three graphs that the population of Hebron at the end of design period will vary between 67,000 and 87,000 persons. An average figure of 75,000 inhabitants in the year 2014 will be justifiably used as a basis for the design.

(f) Present & Future Population Densities

According to the records of the Department of Statistics the city has different population densities in different areas, as shown in the drawing No. 1. At the rate of these population densities, the total present population of the city is worked out as 32,000 as shown in table No. 5. The balance population of 6000 persons dwell in the areas surrounding the city.

The future population densities of the town are not expected to follow the same pattern i.e. they will not increase in proportion to the present population densities but will decrease in the areas which are at present most densely populated and rise in the areas which are at present less populated. This will be due to the simple reason that as the time will progress the people will shift from the old quarters to the newly and better developed areas for want of better living.

The forecast of future densities of population and their areas, as shown in table No. 6 and drawing No. 2, have thus been made on the basis of 75,000 persons. This also includes about 12,00 inhabitants who are expected to dwell in the areas surrounding the city.

TABLE No. 5

PRESENT POPULATION DENSITY IN HEBRON[1]

Section	Area in Dunums	Density of Population	Estimated Population
A	1431	4	5,724
B	461	7	3,227
C	723	18	13,014
D	148	25	3,700
E	84	46	3,864
F	15	65	965
G	8	190	1,520
Total	2,870		32,024

Add number of persons living in the areas surrounding the city — 6,000

Total Population = 38,024 persons

TABLE No. 6

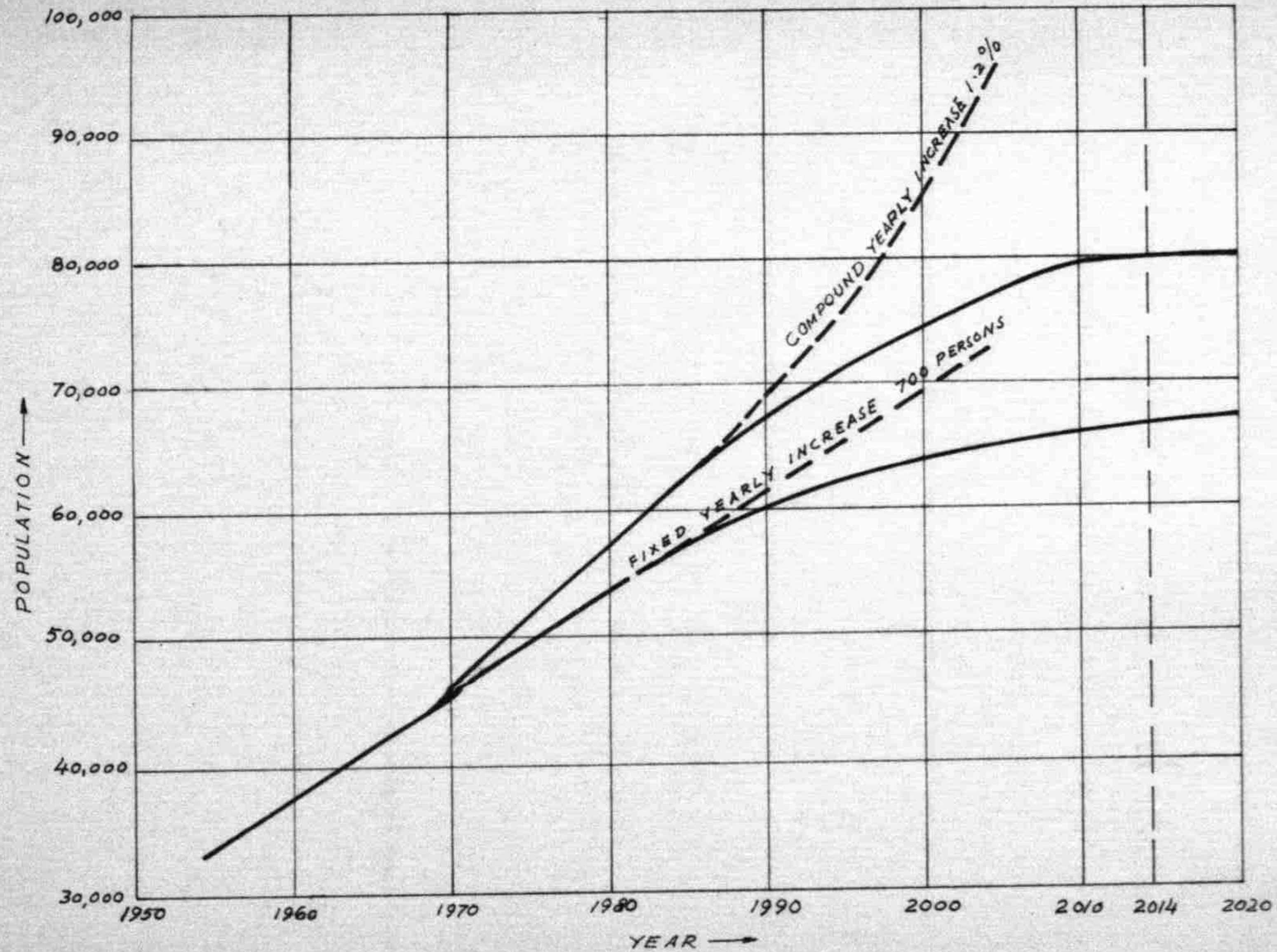
FUTURE POPULATION DENSITY IN HEBRON IN THE YEAR 2014

Section	Area in Dunums	Density of Population	Estimated Population
A	1511	12	18,132
B	314	20	6,280
C	931	35	32,585
D	258	65	16,770
		Total	73,767

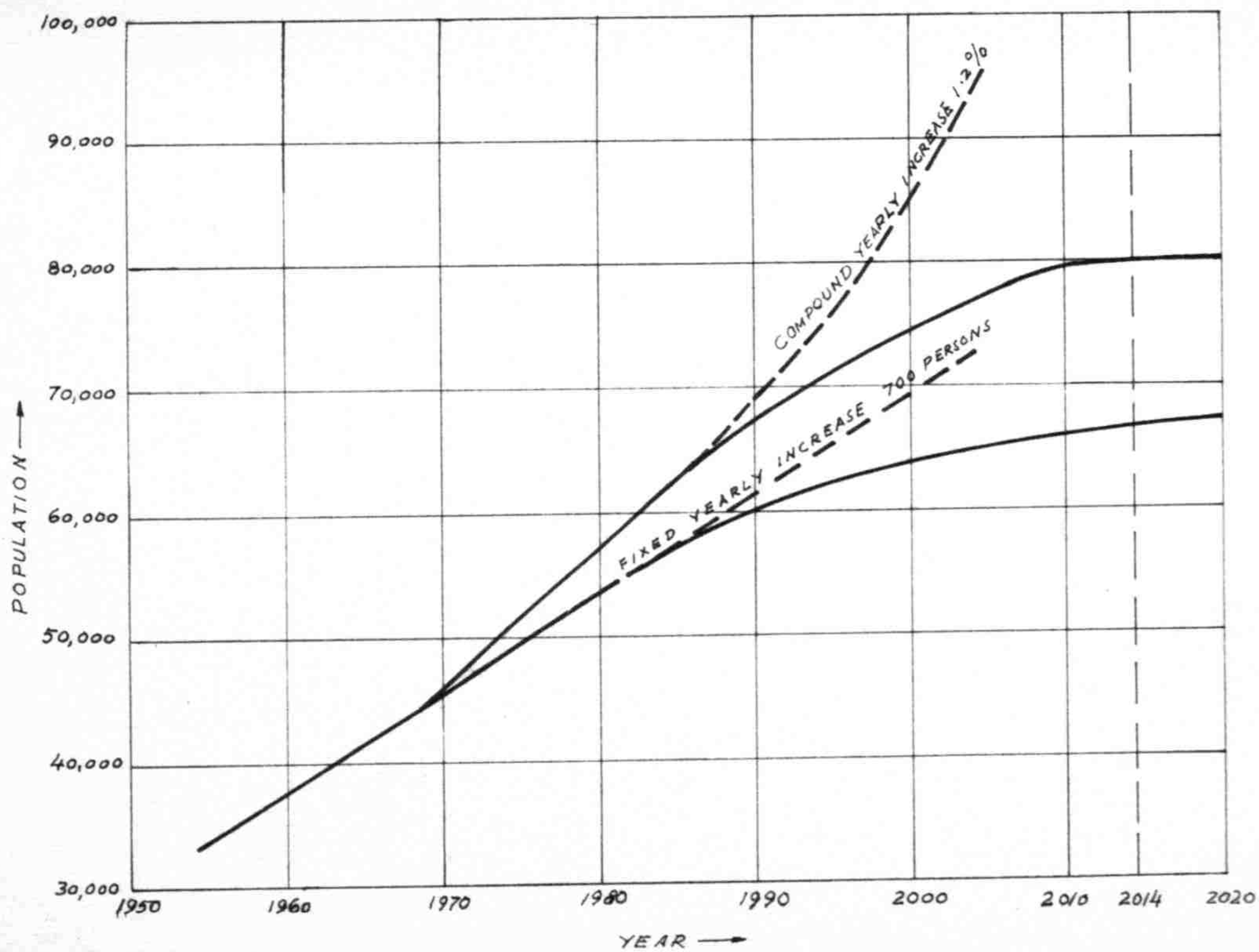
Add number of persons living in the areas surrounding the city — 1,233

Total population 75,000 persons

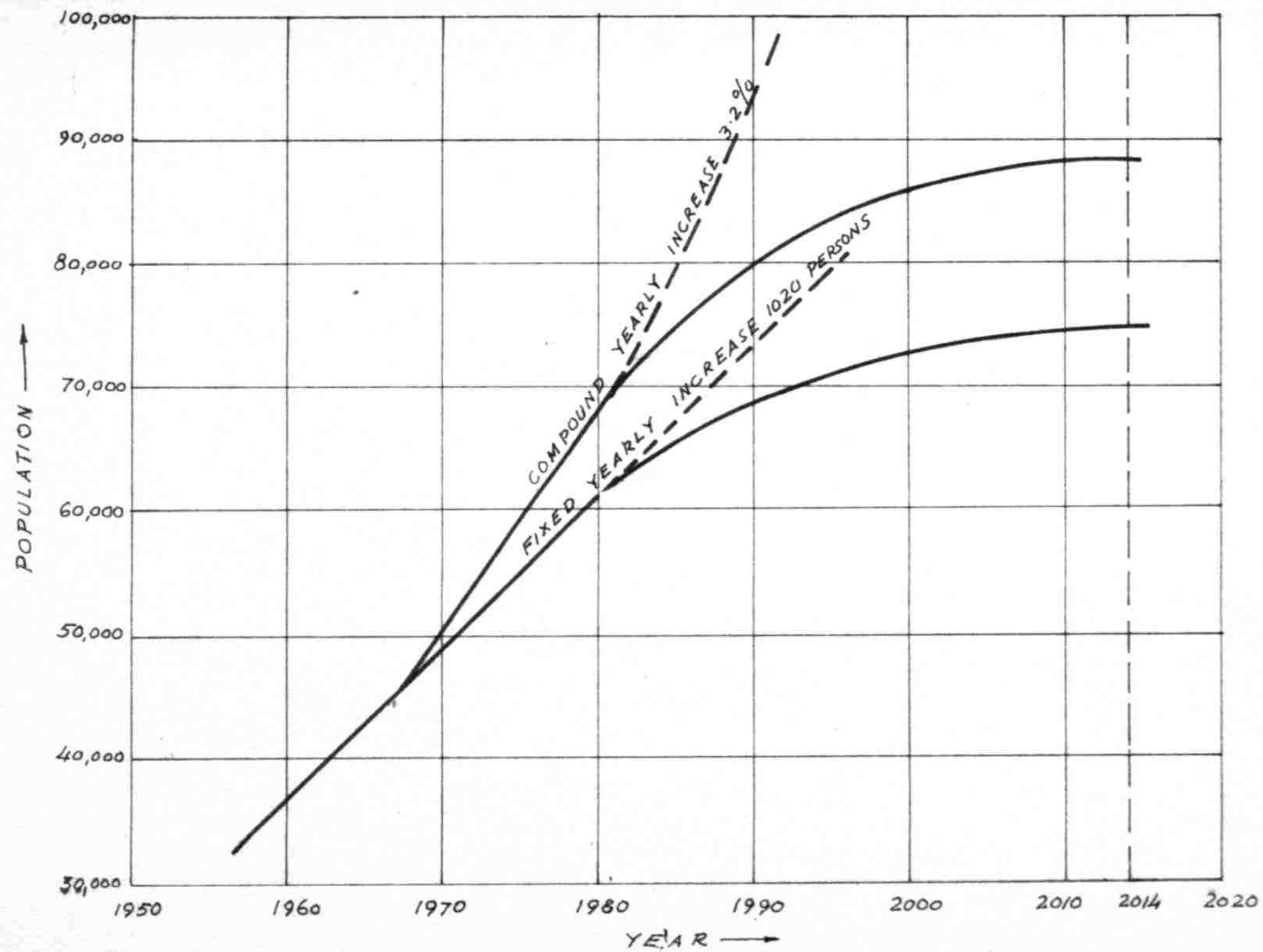
1. Data obtained from the Preliminary Report on "Sewerage System for Hebron Town" made by Associated Consulting Engineers, Beirut, Lebanon.



INCREASE OF POPULATION ACCORDING TO LOCAL STATISTICS GRAPH NO: 1

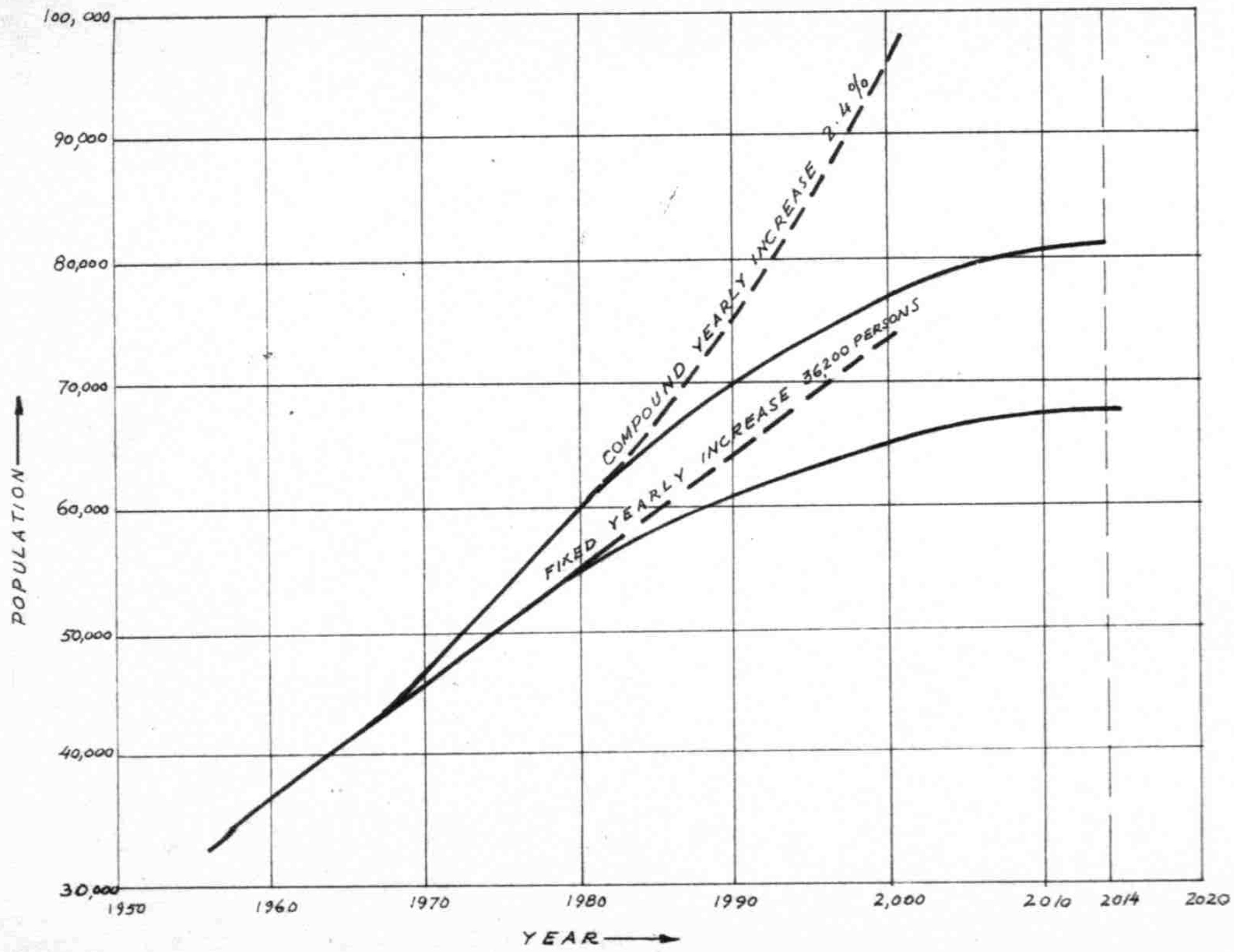


INCREASE OF POPULATION ACCORDING TO LOCAL STATISTICS GRAPH NO: 1



INCREASE OF POPULATION ACCORDING TO INCREASE IN NUMBER OF STUDENTS

GRAPH NO: 2



INCREASE OF POPULATION IN HEBRON ACCORDING TO INCREASE OF POPULATION IN JORDON

GRAPH NO: 3

CHAPTER III

SEWERAGE SYSTEM

A well designed sewerage system is one of the vital public utilities which makes the way of life modern. It performs the function of collecting water-borne wastes of domestic, commercial and industrial origin and of storm water flows for conveyance to point of discharge or disposal. In order to adopt the adequate and most suitable method for the collection, the decision about the type of system should be made.

A. TYPE OF SEWERAGE SYSTEM

The towns are sewered generally according to one of the following two systems.[1]

The combined system, in which the soil sewage and surface water sewage are discharged into one sewer which leads to the sewage treatment works or point of outfall.

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1. A third system which is the partially separate system is a compromise of the separate and the combined systems. In this system, the greater part of surface water for example the water from the road surfaces and the front portions of house roofs etc., is dealt with by the surface water sewers, while a portion of surface water for example that which is discharged by the backyards of houses and back parts of roofs is passed to the soil sewers. This method is not in use now as it is practically difficult to be adopted.

The separate system, in which the soil sewage is carried by an individual system of sewers, while the surface water is carried away either by a number of local systems or by separate sewers.

Each one of the systems has its own merits and demerits and therefore the question whether the sewerage of a particular town should be combined or separate is to be studied on the actual conditions and circumstances which prevail in the town.

Before we decide about the type of sewerage system the possibility of conveyance of storm water by surface drainage should also be studied. Since the topography of the town of Hebron is such that the centre of the town is crossed by a wide valley which forms the natural water course for the storm water flows, there is practically no need to provide for the collection of storm water on all streets and roads. The entire surface run off will reach the valley easily and can be carried through a surface drain or a main sewer from the north to the south of the town. A surface drain will not be suitable in this case as the road is very narrow and passes through congested areas.

The location of the treatment plant has also an important bearing on the decision of combined or separate sewers. The treatment plant has been proposed in the south at a distance of about 2600 meters from the end of town, so that the prevailing wind which is south-west does not bring the foul smell and odour to the town.

Keeping in view the above two points, the separate system is proposed and recommended for the town of Hebron due to the following advantages:

1. Storm water being in-offensive does not require treatment and can therefore be disposed of in local depressions out of the town, thus making a saving in the construction as well as in the operation and maintenance cost of the treatment plant. A further saving in this case will be in the length of storm water sewer required.

It is easier to finance the two systems separately because of less finances required at one time.

3. Some of the storm water flow can be taken care of by the road side gutters.

4. The existing drain which at present carries the domestic sewage can be made use of for carrying storm water for the time the city is incapable of providing funds for the construction of storm water sewer.

5. Self-cleansing velocity can be attained as the size of the sewer in separate sewer needs to be made no larger than necessary to take the peak rates of soil flow, it is usually unpracticable to design a combined system, with

adequate capacity for the storm water run off which will also be self cleansing at the minimum dry weather flows of sanitary sewage.

6. The rainfall in the area is very little. The combined sewer and treatment work if designed, will run underloaded for the entire year except for a short duration of storm. Thus it will be under constant danger of clogging and choking.

(A) MATERIAL FOR SEWER CONSTRUCTION

The selection of material for sewer construction depends on the following factors.

1. Flow characteristics - frictional coefficient
2. Life expectancy and use experience
3. Resistance to scour
4. Resistance to acids, alkalis, gases, solvents etc.
5. Ease of handling and installations
6. Strength to resist structural failures
7. Type of joint - watertightness and ease of assembly
8. Availability and ease of installation of fittings and connections
9. Availability in sizes required
10. Cost of materials, handling, and installations,

It is evident from the above that no one material will meet all requirements and conditions encountered in sewer design. The most suitable material for the particular application is therefore to be selected after giving proper consideration to most of the foregoing factors.

The following materials are used for sewer construction.

1. Concrete pipe
2. Vitrified-clay pipe
3. Asbestos-cement pipe
4. Cast-iron pipes
5. Brick masonary and monolithic concrete sewers.

Out of the above cast iron pipes are not locally available and are costly. Moreover, such pipes are subject to corrosion by acid or highly septic sewage and by acid soils. These are therefore not suitable for use in ordinary gravity sewers.

Brick masonary was used when concrete was not in existence. Because of high cost, lack of durability, it is also not suitable for sewer construction except in special applications.

Cast-in-place sewers of reinforced concrete are used when the required size which is usually very large is more economical than pre cast pipes, or when a special shape is required. Since in our case neither of these conditions exist, such sewers are also not suitable.

The pipe materials which can be considered for our case are therefore concrete, vitrified-clay and asbestos-cement. The vitrified clay pipe and asbestos-cement are available only up to 36 inches diameter (or 42 inches in the case of vitrified clay pipe) and hence can not be used for storm water sewers which will require larger sizes. The concrete pipe will therefore have to be used for storm water sewer.

The other pipe materials i.e. vitrified clay or asbestos-cement pipe though can be used in sanitary sewer but the concrete pipes will also be more suitable for the sanitary sewers due to the following reasons:

1. It has strength to resist sewere compacts and sustain heavy overburdens. The Concrete cradles will therefore not be required.

2. Long laying lengths and the rapidity with which the trench may be opened and backfilling made.

3. Its self healing property as a result of which the small cracks will heal in the presence of moisture and become stronger than before .

4. Many types of joint designs are available depending on the degree of watertightness required.

5. Fittings of the same shapes and for the same purposes that are obtainable in clay are also available.

6. There is no possibility of corrosion as sufficient velocities are available to prevent the sewage from becoming stale and septic, as well as to prevent the sludge deposit. There are also no chances of acid formation because the sewage will not remain in the pipe for a long time.

7. The erosion is also not expected as the velocities are generally within the allowable limits. In the lengths where velocities will be excessive, the protective lining of clay linear blocks will be provided.

8. The concrete pipe are available locally.

C. DESIGN CRITERIA

The function of a sewer is to receive sewage storm water or other liquid wastes and to transport them from one location to the another. It must be therefore deep enough to receive the flow from its source. Its size and slope or gradient must be adequate for the minimum and peak flows to be carried. The slope must be sufficient to avoid deposition of solids. These and other such details form the design criteria for the sewerage system and are very important to be considered while designing the system.

(a) Minimum and Peak Flow of Sanitary Sewage

The first criteria in the design of a soil sewerage system is to estimate the minimum, average and the maximum rates of flow to be expected from the various components areas to be drained.

Since the soil sewerage is primarily a used water supply, the dry weather flow is very similar in quantity to the minimum, average and maximum rates of water consumption. It varies continuously not only with the time of the day, but with the day of the week and the season of the year, with extreme flows occurring between 2 and 6 a.m. in the winter, and peak flows occurring during day light hours in the summer.

Since the records of existing sewerage system or water supply system are not available, the estimates of the sanitary sewage component of the minimum flows or the peak flows, and the exact or definite relationship between them cannot be made. The decision will therefore have to be made according to the standards set by different authorities in these respects.

Many state regulatory agencies (14 out of 38 state boards of health) have set rates of 400 gallons per day per capita for laterals and 250 gallons per day per capita for trunk sanitary sewers as the minimum acceptable design flow rates where no actual measurements or other pertainent data available.[1] The peak flow rate in the trunk sewer is less

1. A.S.C.E. & W.P.F.C. Manual on "Design and Construction of Sanitary & Storm Sewers", Washington D.C. 1960, p. 27.

than in the laterals or mains because the peak is not attained in all the sections of the town at the same time, due to difference in habits of the people in different sections. These rates have been set on the basis of per capita water consumption of 100 gallons per day which is usual in the United States. In our case since the water consumption is less therefore only the ratios will be used. The minimum flow is usually taken half of the average daily flow. The criteria will therefore be as under:

Peak flow in laterals = 4 x Average daily flow

Peak flow in main sewers = 2.5 x Average daily flow

Minimum flow in all sewers = 0.5 x Average daily flow

(b) Selection of Type of Sewer

It is now common practice to use circular cross section for all sewers upto about 8 feet in diameter and will be followed in this design too. This shape has the advantage of giving the maximum of cross sectional area for the amount of material in wall, convenience in manufacturing of pre cast pipes, excellent dydraulic properties and less cost of laying precast pipes as compared to cast-in-place sewers. The egg-shaped sewers, which claim principal advantage of having slightly higher velocity for low flows over the circular sewer of equal capacity will not be of any advantage in our case since the velocities are already favourable due to high slopes available.

(c) <u>Minimum Size of Sewer</u>

The minimum size of sewer will be adopted as eight inches because experience shows that

(i) the smaller sewers are liable to become clogged

(ii) the difference in cost between small sewers of different diameters is not so great as the problem of clogging of sewers and cleanning it.

(iii) it can serve for more period of time.

The minimum depth of sewage in the sewer due to bigger size of 8 inches pipe will not be of concern in this case as the quantity of flow in the mains, which are few will not be so less as to cause the velocity less than that required for self cleansing. Moreover the velocities which are sufficiently high will not reduce beyond self cleansing.

(d) <u>Minimum Velocities</u>

It is a usual practice to design sanitary sewers with minimum slopes which will produce velocities of 2 f.p.s. when flowing full and to design storm drains with minimum slopes which will produce velocities when flowing full of 3 f.p.s. This practice is based on the experience that the velocity of not less than 2 f.p.s. is required in order to prevent settlement of sewage solids when flowing full. Since this is based on the condition of sewer when running full, and it ignores altogether the actual flow which will obtain in a sewer or drain and the corresponding depth, the practice often results in deposit where the actual flow is much less than the full capacity of the sewer. The sulfide build up is also likely to occur due to the low velocities of flow. The condition is aggravated if the sewage is strong and the temperature is relatively high causing serious odour problems, corrosion, increase in chlorine demand and increased difficulty of treatment of sewage. A higher velocity usually 3 f.p.s. or sometimes 3 1/2 f.p.s. is therefore required in such a case.[1]

Since in our case the topography of the area is favourable to allow more slopes, a minimum velocity of 2.5 f.p.s. has easily been adopted.

(e) Maximum Velocity

The maximum velocities of flow in sewers are determined primarily by the destructive effect of grit laden water moving at a high speed. For this reason it is usual practice to limit velocities in pipes running full to not more than 10 f.p.s. In our case the velocities in sanitary

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1. A.S.C.E. & W.P.C.F. Manual on "Design & Construction of Sanitary and Storm Water Sewer", (Washington D.C.: 1960) (Third Printing 1963), p. 132.

sewers have been limited to 10 f.p.s. but the velocities in storm water sewer can not be avoided due to high slopes of the ground and bigger sizes of sewers required. However, since the storm water sewer will have flows only when it rains which is a small percentage of the total time, it is obvious that erosion will be much less than for the sewers carrying high velocities all of the time. The repairs are also not much difficult in case of separate storm water sewer, as these can easily be made when there are no rains.

(f) Minimum Slopes

In the practical design of a sewer, efforts are made to keep the slope of the sewer nearly parallel to the surface of the ground in order to maintain depths not greatly in excess of the required minimum. This has been fully achieved in our case and the sewers have been designed to follow the slope of the ground surface to serve its function at minimum cost. At the places where the ground is flat, a minimum slope which produces minimum velocity of 2.5 f.p.s. when flowing full has been adopted to avoid nuisance from deposits of putrescible organic matter and to reduce the necessity for frequent clearing.

(g) Maximum Slope

The maximum slope to be used for a sewer is determined from the consideration of erosion which is caused primarily by the grit or other unorganic solids which are transported along the invert when the velocity of flow is high. In establishing apolicy for maximum slopes for a particular project not only the velocities which cause erosion are to be considered but also the duration of such velocities.

It is a common practice to limit the velocity in concrete sewers to about 10 f.p.s. where grit erosion is expected to be a problem. In our case since the available ground slopes are excessive there are two alternatives. The first is that the slope of the sewer be kept parallel to the surface of the ground and care for expected erosion due to high velocity be taken by the use of either protective lining and repairs of erroded parts of pipe or its replacement as required. The difficulty here will be that the interior of the continually used sanitary sewer will not be readily accessible for repairs to the invert of damage caused by erosion and thus the repairs will be difficult and expensive. In the case of a storm water sewer, however the repairs can easily be made during the dry weather.

The next alternative is that the sewer be designed at a flatter grade which will require greater depths of excavation, large sizes of sewers and drop manhole structures. This will no doubt be a costly affair.

To decide exactly what way should be adopted, the probable cost of repair to an erroded sewer should be compared with the additional cost of constructing the sewer to a flatter grade. Since the amount of grit expected in the sewer is not known, the erosion expected and hence the cost of its repair cannot be estimated. However since the velocities met in the sanitary sewer are only slightly excessive and that too in few cases, and further that in the case of storm water sewer the velocities will be excessive for a short time during rain storm, for which the repairs can also be made easily, the first proposal (i.e. laying the sewer pipes parallel to the natural slope of the ground) is more economical and hence has been adopted for this design.

(h) <u>Depth of Sewer</u>

The sanitary sewers serves the dual purpose of carrying sewage from one point to another and of providing an outlet for users of the system. The depth of these sewers must therefore be such as to provide an adequate outlet for all connections. Since the houses have no basement, one meter depth of the sewer (top) below the basement of the floor is sufficient and has been adopted.

The depth of a storm sewer will however be sufficient to receive the outlet from the catch basins, and would be at such a depth that it does not interfere with the connections of the sanitary sewer.

<u>Summary of Design Criteria</u>

The following is a summary of the Design Criteria adopted for the project.

1. Maximum flow in laterals ... 4 x Average daily flow
2. Maximum flow in trunks ... 2.5 x Average daily flow
3. Minimum flow in all sewers... 0.5 x Average daily flow
4. Minimum size of sewer ... 8 inches
5. Minimum velocity .... 3 feet per second
6. Maximum velocity ....10 feet per second for sanitary sewer

   No limit fixed for storm sewer
7. Maximum slope .... According to the topography
8. Minimum slope 0.005

In addition to the above the following criteria will also be followed.

9. Invert of mains at a junction shall be above those of trunk by at least half the difference in diameter of small pipe and three-fourth difference in the diameter of large pipe.
10. Manholes will be provided at all changes in alignment, size or grade, at all junctions, and intermediate points and at spacing preferably not more than 75 meters for providing facility in the cleaning of sewers.

(D). <u>DESIGN OF SEWERS</u>

The design of sanitary and storm water sewer is different from each other, and will therefore be considered separately.

I. <u>Design of Sanitary Sewer</u>

The basic factor involved in the design of sanitary sewer is to determine the total amount of sewage flow which is to be taken care of by the sewer. It requires the consideration and determination of the following.

(a) Domestic Sewage Contribution

(b) Commercial Sewage

(c) Industrial Wastes

(d) Ground Water Infiltration

Each of the above is dealt with briefly as under:

(a) Domestic Sewage Contribution

The amount of domestic sewage is based upon the per capita water consumption because domestic sewage is the used water discharged into a sewer. Since the entire water supply does not reach the sewer owing to leakage, lawn sprinkling, irrigation, etc., the amount of domestic sewage is slightly less than the amount of water supplied to the community. It has been experienced that about 70% to 80% of domestic water consumption reaches the sewer. Since there are many open areas in the town where the water will be used for lawn sprinkling and irrigation etc., the percentage of water reaching the sewer will be low in our case. So assuming 70% of the water reaching the sewer, the amount of domestic sewage flow = 0.80 x 0.75 = 52 litres/capita/day.

(b) Commercial Sewage

Commercial areas such as stores, hotels, offices and other business areas also contribute the sewage. Since the city of Hebron is a small town and such areas are scattered all around in the town, the contribution from these is assumed to be amply cared for in the peak allowance for per capita domestic sewage flows.[1]

(c) Industrial Wastes

Hebron is not an industrial town, but it has some small industries like tomato canning, tanneries and a slaughter house. The volume of wastes produced by these industries is worked out as under:

Tomato Canning

The factory produces 60 tons of tomato per year (which is equivalent to 2400 cases of tomatoes per year) taking the average volume of waste produced as 7.5 gallons per case[2], and the season of the factory to last for three months,

$$\text{volume of waste} = \frac{2400 \times 7.5}{3 \times 30} = 200 \text{ gallons per day or } 0.5 \text{ litres/minute}$$

1. W.P.C.F. & A.S.C.E. Manual on "Design and Construction of Sanitary and Storm Sewer", (Washington D.C. 1960), p. 29.

2. Rudolf W., Industrial Wastes, Their Disposal and Treatment (New York: Reinhold Publishing Corporation, 1953), p. 57.

Slaughter House

Average animals slaughtered day are as follows:

Sheep	.....	90 to 100 per day
Camel & cows	.....	5 per day

Taking an average kill of 100 cattles per day and volume of waste per animal as 359 gallons per day[1]

Volume of Wastes = 35900 gallons/day

or 95 litres/minute

Tanneries

There are two tanneries which process 120 hides per day. In the tanneries, the hides are received from the slaughter house, the difference being contributed by the adjoining villages. The volume of waste according to the distribution of hides are as follows:

Volume of waste from 115 sheeps at the rate of 4 gallons per sheep[3] = 115 x 4

= 460 gallons per day

Volume of waste from 5 cows and camels at the rate of 360 gallons per animal = 5 x 360

= 1800 gallons per day

Total volume = 460 + 1800 = 2260 gallons per day

or 6 litres per minute

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1. Nemerow, C.L., <u>Theories and Practices of Industrial Waste Treatment</u> (Addison Wesley Publishing Co., Inc., 1963).

Therefore, Total volume of waste from all industries

$= 95 + 6 + 0.5 = 101$ litres per minute.

Since this amount of industrial flow is even less than 1% of the domestic flow (12,286 litres/minute) it does not need any special treatment and will be easily taken care of along with the domestic flow.

(d) Ground Water Infiltration

Sanitary sewers always carry some amount of ground water infiltration through the joints, craked pipes, or other openings such as manholes etc., and increase the quantity of sewage flow, which further results in the increase in size of pipe, cost of pumping and treatment facilities. It is therefore highly desirable that such flows be kept to a minimum by possible ways and means of strict supervision.

The determination of the amount of infiltration requires the knowledge and study of the factors on which it depends. The important factors are:

1. The level of the ground water with reference to the sewer.

2. The character of the sub-soil. - Sand and Gravel for example permit more water to leak than clay.

3. The watertightness of the joints and the provision to prevent cracking of the sewer pipe.

4. The character of the construction of house connections. This usually relates to the extent and care with which

the house connections are supervized.

The level of the ground water in Hebron is 300 to 400 meters deep and is therefore not likely to contribute at all towards the infiltration. Though subsoil consists of low lime stone, the effect of its low permeability will not be of much concern because the percolation of water into the ground will be very little due to high slopes. The water tightness of joints and construction of house connection will be controlled to the minimum possible extent by exercising thorough supervision over the works. The prevention for cracking of pipe will also be made by proper handling.

However, in spite of all the measures adopted for prevention, some unavoidable infiltration would find the way into the sewer and the allowance for this has been made in the capacity of the sewer at the rate of 20 litres per capita per day. This makes the total flow in the sewer as under:

Domestic flow	=	531.p.c.p.d.
Ground water infiltration	=	201.p.c.p.d.
Total flow	=	73 l.p.c.p.d.

This figure is nearly equal to the per capita water consumption and therefore a rate of 75 l.p.c.p.d. for total flow has been adopted in the design to make the calculations easier.

II. Design of Sewer Net Work

A system of network of sewers in which the sewage may flow from the entire area to the treatment plant by gravity is the most desirable for reasons of economy as well as maintenance. Fortunately the area of Hebron is such that this aim has been fully achieved.

It is apparent from the contours that the city has a central valley on which the eastern and wester hills meet. The general slope of the valley is from north to south. A main trunk 'T' has therefore been provided on Malik Abdullah Road leading to south towards Nasiruddin Road as shown on the drawing No. 3 . All the main sewers meet the trunk sewer easily as the hills slope towards the valley. Since the town is very thickly populated and is very old one, no proper road system exists. The city authorities are however considering the proposal to provide some roads by demolishing some of the areas. Only few main sewers have therefore been proposed in consultation with the local city authorities on the existing roads where no alteration in the alignment is expected. However, since the slope of the entire area is in general towards the valley where the main trunk sewer T is proposed, more main sewers may easily be joined later to the trunk sewer when the new roads are planned and constructed. This will not make any change in their design afterwards because the sizes of the main sewer does not increase beyond 8 inches which is the absolute minimum size adopted for the design. Moreover, since the favourable slopes are available, the velocities will also not be reduced beyond the minimum possible.

The areas, populations served, discharges, slopes, size of sewer, invert levels of the sewers have been calculated and tabulated as per table No. 1 to 13. The calculations have been based on the well known Hazen & Williams formula with the value of coefficient of roughness of pipe C=100.

The minimum depth of flow during lowest discharge period has been checked to be not less than 2 inches, and the velocity, not less than 2 f.p.s. by the partial flow diagram[1]. In order to keep the velocity not less than 2 f.p.s. during the minimum flow, the slope of the trunk sewer from manhole 72 to M.H. 67 has been provided as 0.008 which is slightly more than the minimum allowable. This will not doubt entail a little more depth of sewer in a length of about 375 meters but this excess cost will easily be met by the reduction in size of pipe from 10 inches diameter to 8 inches diameter in a length of 600 meters from M.H. 72 to M.H. 62. A further advantage will be that the excess scouring velocity which otherwise would have caused from M.H. 67 to M.H. 62 will also be reduced.

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1. Steel, E.W. "Water Supply and Sewerage", (McGraw-Hill Book Company Inc., New York, 1960), p. 375.

The design has been made on the basis of maximum designed discharge which will take some time to attain but as the slopes are favourable, the velocities are not likely to be reduced beyond the minimum allowable. However, in some portions where the areas are flat and the slope is less periodic cleaning of sewers will be done by flushing with a hose.

The longitudinal sections of the trunk sewer and all the main sewers have been drawn and are shown in the drawing No. 6 to 10.

II. <u>Design of Storm Water Sewer</u>

The basic factor in the design of storm water sewer is same as that in the design of sanitary sewer i.e. the total amount of surface water to be taken care of by the sewer, but its estimation is however far less straight forward than calculating the flow of soil sewage. There are four factors to be determined none of which is easily decided.

1. Choice of design storm or return period.
2. The extent of drainage area.
3. Impermeability of the surface of the drainage area.
4. Appropriate intensity of rainfall.

A brief discussion of these factors is worth mention here so that a suitable method may be selected.

Choice of Design Storm or Return Period

Storm water sewers are usually designed to dispose of the flow from a storm having a specified return period which is determined by consideration of the damage caused from a flood of specified frequency. Since it is usually difficult to estimate and evaluate the damage which may result from a flood, the selection of the proper return period is often dependant on the designers judgment.

In Hebron since most of the areas are residential and the ground slopes are also favourable, the harm expected is very little. A return period of one year is therefore justified for design and has been adopted.

Drainage Area

The area contributing to any point under consideration is determined by establishing boundaries by field survey and from topograhical maps or aerial photographs.

The drainage areas for each watershed has been marked on the contour map of the city and are shown in drawing number 4.

Impermeability Factor

The proportion of the total rainfall that will reach the sewers will depend on the relative porosity or imperviousness and the slope of the surface. It is a function of the duration of the storm and varies continuously building up from a low to a high value for the area concerned during the period of rainfall. It is thus the most variable one in the design of storm water sewer, and the largest errors made in rainfall run off calculations are usually in over estimation of impermeability factor.

There are many methods of estimating impermeability factor which vary considerably in their way of estimation and accordingly in their accuracy. But unfortunately none of these can be applied here because the town is a very old one and the details of the various components such as total length of roads, paved and unpaved, exposed earth surfaces, paved and roofed surfaces etc., in the city are not known. In the event of the circumstances and in the light of the fact that most of the streets are not properly paved and the exposed plots are frequent, an average value of 0.5 is reasonable and has been adopted.

Intensity of Rainfall

The determination of rainfall intensity will depend on the following factors.

1. Average frequency of occurence which has been taken as one year.

2. Intensity duration characteristics of the rainfall of the adopted average frequency of occurence.

3. Time required for run off from remotest part of the drainage area to reach the point under design (time of concentration).

The intensity duration characteristic which are derived from gage measurements of rainfall, are not available. The intensity of rainfall will therefore be found by using the most suitable formula out of numerous formulas available.

There are many methods which are followed in the American as well as English practices for determining the run off. A discussion of all of these is not of much use here. However to select the most suitable one, a brief discussion of those methods which give accurate results is necessary.

1. <u>Rational Method</u>

The Rational Method translates rainfall into run off by the formula

$$Q = A\ I\ R$$

Where Q = rate of run off in cubic feet per second

I = coefficient of run off of the area

R = average rainfall intensity in inches per hour

A = drainage area in acres.

This method is based on the following basic assumptions:

1. Rate of run off to any point under design is a function of the average rainfall rate during the time required for water to flow from the remotest part of the drainage area to the point, said time of flow being called the "time of concentration".

2. The peak rate of rainfall occurs within the time of concentration.

The application of the Rational Method to a design problem requires determination of the following basic data.

1. Drainage area tributory to point under design.

2. Probable future condition of the drainage area - that is, percentage of impervious surface or character of land use when developed to the extent assumed.

3. Selection of an appropriate run off coefficient for the entire area or for component inlet areas.

4. Rainfall frequency and intensity duration curves for the locality.

5. Time of concentration, including both inlet and conduit flow time to point of design.

Out of the above since the intensity duration curve is developed from rainfall records which are not available, this method cannot be straight away used.

Hydrograph Method

This method is an outcome from science of hydrology, and has made progress in recent years. It is based on application of a definite design storm pattern to the drainage area and the determination of a run off hydrograph from rainfall records. The method requires assumption of a hyetograph of specific shape and size, for which a detailed study of the most severe storm is necessary. Since the rainfall records arenot available, this method can not be used.

Empirical Methods

Many empirical formulas have been developed by the engineers for finding the run off. In general these formulas are expression of the run off in terms of the area drained, the relative imperviousness, the slope of the land, and the rate of rainfall. Comparison of these formulas when applied to the same conditions give a wide range of divergence sometimes even more than 750 percent.[1]

These formulas have therefore become out of use and hence will not be used.

Llyods Davies Method

The method of designing storm water sewers to take when running full the run-off of a storm, the duration of which is equal to the time of concentration of the sewerage system above the point on the sewer, is known as Llyods Davies Method by the British Engineers. According to this method, of all the storms of equal frequency of occurrence the storm which has a duration equal to the time of concentration produces the greatest run off.

This method is very much similar to the Rational method and the run off from any drainage area is expressed by the formula

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1. Babbit, H.E. "Sewage and Sewerage Treatment", *op. cit.* p. 45.

$$Q = 60.5 \times A.p \times R$$

Where Q = Run off in cubic feet per minute

A = Area in acres

P = Impermeability factor

R = Intensity of rainfall in inches per hour.

For the determination of intensity of rainfall, many formulas were advised out of which the Llyods Davies method is based on the following formula:

$$R = \frac{5.9}{t^{0.625}}$$

Where R = inches of rainfall per hour

t = duration of storm in minutes.

This formula is based on once a year storm which is usual in England and is less if compared to the American practices of once in five or once in ten years storm. It is however suitable in our case because the very heavy storms occur infrequently and to build a sewer capable of caring for the storms which occur once in five or once in ten years would involve a prohibitive expense over the investment necessary to care for the ordinary heavy storms encountered annually. The sewers will thus be allowed to overflow on such exceptional ocassions for a small time. This will also result in a more freqeunt use of the sewerage system to its full capacity.

The Llyods Davies method is not strictly accurate except when applied to hypothetical cases in which the impervious area is evenly distributed (in time, not distance). In practice large parts of areas are often bunched in one locality and in such a case the run off from part only of the drainage area and resulting from a storm of duration equal to the time of concentration of thatpart of the drainage area may be greater than the run off from the whole area. Also this method when applied directly to complex problems involving several drainage area may give erroneous results. For example, when two sewers having different time of concentration meet, the flow from the point of junction is calculated according to this method, on the basis of longer time of concentration whereas it may be found that if the calculation is made on the basis of the shorter time of concentration and that part only of the total impervious area that would contribute after that time of concentration, a greater run off results.

This deficiency has been taken care of by drawing the time area graph as per drawing number 5 and applying the tangent method. The most concave point on the curve is selected and is marked "zero time, zero acres". From the new"zero time,zero acres" a point is measured minus twenty minutes zero acres, and from this point a tangent is drawn to the curve. The point at which this tangent meets the time area curve gives the critical time of concentration and effective impervious area which produces the maximum run off.

Since this method can be used in the absence of past rainfall statistics and is also fairly accurate, it will be used for this design.

The contributing areas have been determined by dividing the area into different watershed areas as shown in drawing number 4. The inlet time for the flow to reach from the remotest point in the contributed area has been read from a Nomogram.[1] For this the average slope of watershed has been determined and the character of the ground has been taken as "between paved and bare soil". The time of flow into the sewer has been calculated by knowing the velocity of flow when flowing full and the length of the sewer. By adding the inlet time and the time of flow in the sewer the time of concentration has been calculated.

The intensity of rainfall and the run off has then been calculated by the formulas. The diameter of the sewer, its designed discharge, designed velocity and time of flow has also been calculated and tabulated in table number 15. The designed slopes of the sewers have been provided according to the natural slopes of the ground surface which has resulted in excessive velocities throughout. This could not be helped due to the natural topography of the area. However, since

[1] Seelye Elwyn E. "Data Book for Civil Engineers" Vol. I (New York, John Wiley & Sons) 2nd Edition, p. 500, Fig. H. (Overland Flow time).

the duration of flow into the sewers would be only during rains, the damage expected on this account will be not much and the erroded portions will be repaired when there are no rains.

In the sewer from MH to MH, a small portion is met where the ground is flat. The designed slope in this portion has also been kept higher because the sizes in the earlier portions which are small due to higher slope would have gone very high in a length of 2880 feet thereby increasing in the cost. It would have further increased the depths in the latter portion, making a further increase in the cost.

To economise in the cost of sizes of the pipe,[1] it was considered that some quantity of storm water be allowed to flow in road side gutters. The problem has been well studied and the discharge to be taken care of by the gutters has been calculated by the formula

$$Q = 0.557 \quad S^{1/2} . d^{8/3}$$

Where Q = discharge in cfs.

S = slope of the surface of the road wide gutter

n = coefficient depending upon the roughness and cross sectional shaped of the channel (assumed as 0.13)

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1. Linseley & Franzini, <u>Element of Hydraulic Engineering</u>, (New York, McGraw-Hill Book Co., Inc., 1955), p. 446.

z = transverse slope of the road (assumed as 2%)

d = depth of water allowed at the curb

(allowed as 10 c.m. from the start to the crossing of main road with the Khalid Bin Walid Road and 5 c.m. from the said crossing to the end)

The compuations are shown in table number 16. The size of the pipe required, its slope, velocity etc., for the balance quantity of run off has been calculated. By comparing the two designs (viz. without the road size gutters, and with road side gutters), it is noted that the quantities of run off taken care of by road side gutters are very little and the sizes of the pipes reduce only in the length from MH to MH which are not appreciable. It is therefore recommended that the entire surface run off should be taken care of by the storm water sewers. However, for immediate needs, if the city is not financially in a position to take the full work in hand, the sewer line may be laid only in length from MH to MH as the 1st phase and the earlier portion be joined later.

E. Manholes

Manholes are among the most common and necessary appurtenances to sewage systems and are used to permit inspection, cleaning, and the removal of obstructions from the pipes. These have therefore been located in the system wherever there is a change in alignment, size or grade, as well as at all junctions of the sewers.

Spacing

Practice in manhole spacing varies to some extent. In many parts of the United States a spacing of 300 feet is considered as maximum for the small sized and medium-sized sewers. In this design, a spacing of 75 meters (i.e. about 250 feet) has generally been adopted for providing more facility in cleaning.

Shape

The manholes will be circular in shape with minimum inside diameter of 4 feet which is sufficient to perform inspecting and cleaning operation without difficulty. The circular section has been adopted for reasons of greater durability. A minimum clear opening of 21 inches will be provided so as to enable a man to gain access to the interior without difficulty. The manhole will be widened out rapidly immediately below the opening as shown in the figures.

Material

The material used for the construction of manhole walls include brick and cement concrete. The choice of the material will consider the following:

1. Cost in place including material, labour, and equipment.

2. Durability under all conditions of service

which may reasonably be expected.

3. Adaptability of the material to meet field conditions with particular reference to changes in location, grade or alignment made during construction.

4. Depth of manhole and character of surrounding material.

Since the cement concrete gives more durability under all conditions of service, and standard pre-fabricated steel forms can be easily arranged for depths of one to two meters, the manholes will be constructed of cement concrete. Moreover, as less thickness will be needed, the construction cost will not be uneconomical.

The bottom of manholes will be provided with a base slab of concrete 8 inches thick to support the walls of the manholes and to prevent the entrance of ground water. The flow will be carried away in a smoothly constructed U shaped channel which will be constructed integrally with the concrete base. The side of the channels will be kept high enough to prevent the flow of sewage onto the sloping floor. In manholes where two or more sewers join at approximately the same level, the channels in the bottom will be joined with smooth easy curves. Where the inlets and the outlets are not of the same diameter the tops of the pipe will be placed at the same elevation to prevent back flow in the smaller pipe when the larger pipes are flowing full.

Manholes frames and covers will be made of close grained cast iron. The weights of frames and covers for all streets and roads in the residential area where traffic is not heavy will be 400 pounds. For the main sewer which passes on the main Abdul-Malik Road, frames and covers weighing 600 lbs will be provided. The covers will be roughened to prevent excessive slipperiness but will not be perforated because they allow objectionable odour and entrance of surface water, sand and girt. Circular manholes covers will be used on sewer manholes as they do not fall into the sewer. Manhole covers will be marked so that sanitary sewers may be distinguished from the storm water sewer.

CHAPTER IV

SEWAGE TREATMENT WORKS

A. GENERAL FEATURES

Sewage is treated as a protection to health, to avoid nuisance, to prevent the pollution of natural waters and of bathing beaches, and to avoid damage suits. The treatment is done in a sewage treatment plant whose purpose is therefore to convert the raw sewage into an acceptable final effluent and to dispose of the solids removed in the process. It is fundamental therefore first to determine the basic design considerations such as period of design, the strength of raw sewage, and the required characteristics of the effluent, or the required treatment before proceeding with the design of the treatment works.

In the developed countries such basic design criteria are established by the regulatory bodies, usually the state department of health, and before proceeding with construction of any sewage treatment facility their approval is to be obtained. But in under-developed countries like ours, the case is not so, and deviations are necessitated by such factors as unusual components in the raw sewage, relative cost of treatment facilities required to meet these standards or other justifiable causes.

(a) <u>Period of Design</u>

The design period of the treatment plant will be the same as the period of design established for the estimation of dry weather flow i.e. 50 yards, but its planning will however be so made that the facilities for the present population requirements will initially be installed with provision available for the extension of the facilities as and when they are needed. This is necessary in order not to place any undue financial burden on the present population and to forestall poor operating results in the early years because of oversized units.

The future anticipated population of Hebron has been estimated double that of present population. The number of units in the treatment plant will therefore be so chosen that half of the units may be constructed for the present, leaving the remaining half tobe constructed when the need arises.

The piping, main conduits, channels and other small units will however be designed for the ultimate requirements, because their replacement will not only be costly but practically difficult.

(b) Extent of Treatment

The extent of treatment to be made depends upon the final disposal of the effluent from the treatment plant.

(c) Composition of Sewage

The five-day BOD is the principle yardstick used to measure the strength of the applied sewage. The BOD test requires an incubator to store the diluted sewage for a period of five days at a temperature of 20° C. Unfortunately there is no laboratory at Hebron where the facilities for making such a test are available. Another test known as 4 hour permanganate test was therefore made which does not require an incubator and is quicker.[1] The following results in 4 hour permanganate value of BOD in p.p.m. which is 25 percent of the 5 day BOD were obtained.

Results of 4 hour Permanganate Test

Sample Number	4 Hour Permanganate Value in p.p.m.	5 day B.O.D. in p.p.m.
1	500	2000
2	440	1760
3	450	1800
4	464	1856

To be on the safe side the value of 5 day BOD will be

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[1]The test was made by the engineers of the Associated Consulting Engineers, Beirut, in July 1964.

taken as 2000 p.p.m. The effluent is proposed to be used for irrigation of crops due to the following reasons:

(a) The climate is arid.

(b) Such land is available.

(c) There is acute need of irrigation water.

(d) The ground water level is low enough to permit percolation.

(e) The return of sewage through a sufficient depth of soil will recharge the wells.

The treatment will therefore be such that the BOD of the effluent is sufficiently reduced to make it inoffensive.

(d) Quality of the Effluent

As stated above, the quality of the effluent depends on the effluent utilization. Illinois and New York Standards[1] state that the complete treatment plants employing biological processes should be capable of producing an effluent having a 5 day BOD not greater than 15 p.p.m. and containing not more than 30 p.p.m. of suspended solids.

In our case, since the B.O.D. of the sewage is very high, the reduction of B.O.D. to the extent of 15 p.p.m. will require a very high recirculation and will be very uneconomical. The B.O.D. of the effluent is therefore proposed to be

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[1] H.E. Babbit, Sewerage and Sewage Treatment, op. cit., p. 305.

reduced to 50 p.p.m. which is sufficient to make the effluent inoffensive for irrigation of crops.

(e) Location of the Treatment Plant

The conditions which are to be considered in the selection of the site include topography, soil and underground conditions, danger from flooding, cost and the attitude of the public. Although health-protection requirements place no restriction on the plant location but more isolated site are considered better as it offers a factor of safety against trouble that may arise from improper operation or insufficient administrative support.

The topography of Hebron is such that the sewage can be made to flow by gravity from all parts of the area to the treatment, if the location of the plant is selected to the lowest point of the area served. This will not only be very much economical, as it will save pumping expenditure but will also be dependable from the maintenance point of view in a town like Hebron which lacks technical personnel. Keeping this in view a site comprising an area of 3000 square meters in southwest of the town at a distance of about 2200 meters from the last point on the trunk main has been selected. This site will also have the following additional advantages:

1. The site is well isolated from the residential and commercial areas of the community. Though it is on the

wind direction but the greater distance will not allow the odours etc. to reach the town.

2. Cheap and plenty of land is available.

3. Local disposal of end-product such as grit, sludge or ash is possible.

4. It is accessible to an all weather access road.

5. Possibility of using the effluent for irrigation.

(f) Type of Treatment

The selection of any particular process or combination of processes of sewage treatment is generally based on the study of a number of conditions such as :

(a) the method of final disposal;

(b) the quality of the sewage to be treated;

(c) the efficiency and characteristics of the treatment processes. The criterion must be the quality of the effluent produced;

(d) the skill required in operation, and the availability and experience of operating personnel, and the facilities for maintenance and repair;

(f) head available for the plant and the necessity for pumping of the sewage if there is insufficient natural head;

(g) construction and operation costs, and the amount of money available;

(h) ease of increasing capacity;

(i) quantity and quality of sludge from the processes, its disposal and available areas and possible sites for it;

(j) availability of materials and life of structures and equipment.

The types of treatment process to be selected is dependent on all the conditions and the manner of their combination. In comparing the different processes, however, the operation is to be considered more important, because by proper operation the nature of the effluent can be altered.

The following methods which are generally used and which appear to fit in the local conditions of Hebron are discussed here in brief.

Imhoff Tank

This method of treatment which was used before the development of the sedimentation and the sludge-digestion process has been superseded by modern and efficient processes except in the small installations because the newer methods permit easier control of operation and give better results. The advantage claimed for such a process is that they are cheaper in first cost. This will not be true in this case because these tanks alone are not enough for a complete purification, and would require the use of filter beds and secondary sedimentation units, thus adding to the cost.

Moreover, the maintenance and operation of these tanks is difficult and cumbersome because of odours, creation of unsightly conditions, and foaming problem.

Oxidation Ponds or Lagoons

Oxidation ponds or lagoon are the most simple and cheap treatment processes and are claimed to treat successfully either settled or raw sewage with effluents at least equal to those from sewage treatment plants that provide complete treatment, but these require large areas of land which are not available at low cost. Moreover, the high concentration of sewage due to less per capita water consumption also makes the method unsuitable for use in Hebron. The smell and odours will also be troublesome as the location of the plant is favourable for it. These are therefore not recommended to be adopted.

Oxidation Ditch

It is a novel method developed recently by Professor Pasveer of Netherlands. It is basically similar to the Activated Sludge process but it is much simpler and does not need costly construction work. The other advantages of this method are ease of operation, simple maintenance, low cost of operation and comparatively much less areas required than lagoons. The area required is only 5 percent of the area required for lagoons. This method therefore appears very suitable for the city in question, but the past experience in Europe and even in Netherland has shown that the method has not been used for population of more than 10,000 to 12,000 persons. Such a method is therefore not advisable to be adopted for big cities like Hebron till the experiences at other places prove it worthy for the purpose.

Activated Sludge Process

This method has the advantages of producing a clear, sparkling, and non-putrescible effluent; freedom from odours during operation; degree of nitrification controlable between limits, more than 90 percent of the bacterial removable; and some commercial value in the sludge, but it is not suitable for adoption in cities where constant skilled attendance is not available because of the mechanical equipment that must be operated. Moreover, this process has the following additional disadvantages over the trickling filters.

(a) Greater sensitivity to changes in the quality of the influent.

(b) High cost of operation.

(c) Difficulty in dewatering and disposing of the large quantity of the sludge produced.

Conventional Process

The treatment of sewage in a conventional process involves three stages: preliminary or primary treatment, filtration and final sedimentation. The heart of the process is filter. The other two steps are essential, however, to the successful functioning of the plant.

The advantages which it possess which it possesses over the other methods of treatment are —

(a) the reliability to give a good effluent under wide variations of filter load;

(b) low operating cost;

(c) its ability to function under extreme weather conditions;

(d) the effluent is of high quality.

An outstanding disadvantage to the adoptation of a trickling filter at many plants is the head loss through the filter which varies between 5 and 11 feet, in addition to the depth of the filter. Other disadvantages which it includes over the activated sludge process are —

(a) odour and fly nuisance;

(b) the larger area required;

(c) relatively high construction cost.

These disadvantages are not of very serious nature and will be minimized and overlooked in view of the advantages it possess. Odours are rarely troublesome where the rotary distributors are used, and will be easily checked by proper operation and maintenance. Moreover, even if it could not be checked for a while due to some unavoidable circumstances it will not reach the town, as the treatment plant is at sufficient distance from the town.

The fly nuisance will be minimized by adopting high rate filters or will be controlled by flooding the filter bed for 24 hours every week or two and thus drown the larvae.

The area required for the filter will only be relatively larger than the activated sludge, which will be available with only little additional cost. According to Schroepfer, the cost of the two processes are as under:[1]

Comparative Construction Cost of Trickling Filter
and Activated Sludge Processes
(Cost in Thousands of Dollars)

Plant capacity or sewage Treated m.g.d.	Trickling Filter Plants			Activated Sludge Plants			
	Usual Lower Limit	Estimated in his analysis	Usual Higher Limit	Usual Lower Limit	Average for Aeration 5 hr	Average for Aeration 6 hr	Usual Higher Limit
10	900	1,200	1,500	560	560	700	940
25	2,130	2,850	3,550	1,280	1,600	1,800	2,120
50	4,130	5,500	6,850	2,500	3,200	3,500	4,200
75	6,000	8,000	10,000	3,700	4,750	5,200	6,200
100	7,900	10,000	13,200	6,300	6,300	6,900	8,200

[1] H.E. Babbit, Sewerage and Sewage Treatment, op. cit., pp. 436 and 497.

volume and characteristics of each type of waste produced. As the volume of wastes from the existing industries is very small, no separate treatment, or modification due to these, in the municipal treatment works is required to be made.

B. PRIMARY TREATMENT

The term "Primary Treatment of Sewage" applies to those methods which remove a part of the suspended and floating solids and include the following units:

(i) Screens

(ii) Grit Chamber

(iii) Primary Sedimentation Tanks.

(i) Screens

Screens will be provided for removing large suspended and floating solids which cause the following:

(i) Clogging of sewer pipes, channels and treatment plants.

(ii) Clogging of and injuries to machinery.

(iii) The accumulation of putrifying sludge banks.

(iv) Unsightly floating matter.

The following design criteria will be adopted for the design of screens.

(i) Screens will consist of galvanized steeel bars with a clear opening of 2.5 c.m. between the bars and having a cross-section of 1 cm x 4 cm with long dimensions parallel to flow.

(ii) The velocity of flow through the screens shall be 1.5 ft. per second. The "Ten-State Standard" limits the velocity through the screens to 2.5 f.p.s. but the lower rate is preferred due to greater amount of screening that will be removed.[1]

(iii) The rack will be inclined to the horizontal at an angle of 45 degrees.

The screens will be operated manually in view of availability of cheap unskilled labour and shortage of mechanical personnel. The screenings will be removed at convenient intervals and will be either buried, for which a suitable and sufficient land is available, or will be disposed of along with the general refuse of the city.

The bottom of the screens will be 6 inches below the invert of the sewer toa allow space for collection of large particles which otherwise may cause stagnation and clogging of screens.

(ii) Grit Channel

A grit channel will be provided to remove the grit, which includes coarse particles of sand, gravel, minute pieces of mineral matter, and other materials which are non-putrescible and which have subsiding velocities substantially greater than those of organic putrescible solids. The purpose

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[1]WPCF Manual of Practice No. 8 on Sewage Treatment Plant Design, Third Printing, (Washington: 1963), p. 52.

of removal of these are —

(i) the protection of moving mechanical equipment from abrassion and accompanying normal wear;

(ii) the reduction of pipe clogging caused by deposition of grit particles or heavy sludge in pipes and channels, particularly at changes in direction of the conduit;

(iii) the reduction in frequency of digester and setting-tank cleaning required as the result of excessive accumulations of grit in these units.

The following design criteria will be adopted in the design of grit chamber:

(i) The horizontal velocity of flow in the channel will be kept at one foot per second.

(ii) The settling velocity in the channel for a particle size of 0.2 m.m. is assumed as 0.1 ft. per second.

(iii) The control of velocity within the effective length of the channel will be provided through the use of a partial flume. The partial flume has been adopted in view of its advantages namely, the head loss is kept at the absolute minimum and that the control section can be used conveniently for metering the flow. It is also simple to construct.

(iv) Two channels will be provided so that the difficulty of variable flow of sewage is overcome by placing the channels in or out of operation according to the rate of flow.

(v) The grit would be removed manually in view of cheap available labour as compared to cost of maintenance of mechanical equipment. The two channels will facilitate the grit removal.

(vi) The grit removed will either be utilized for filling the low areas near the treatment plant site or will be buried for which the land is available near the treatment plant.

(vii) A drain of open jointed pipe will be embedded in the bed of grit channel to drain away the sewage collected before the removal of grit.

(vii) The grit will be washed in a separate chamber. The arrangement for wash water to overflow back into the grit channel will be made. Thus the grit free from organic matter will be removed by means of a pipe.

It is proposed to provide only one chamber for the screening and grit removing device due to the following reasons:

1. Saving in cost of construction due to one compact unit.
2. Saving in space at the treatment plant.
3. Facility in operation.

(ii) Primary Sedimentation Tank

The primary sedimentation basin has been provided to remove the bulk of the settleable solids remaining in the sewage, and also to reduce the suspended solids contents of the sewage in order to prepare it for subsequent treatment.

The following criteria will be adopted for the design of sedimentation tank:

1. The sedimentation tank will be designed to operate on a continuous-flow basis, and will be circular in shape. The circular section has been adopted because it is cheap in construction as well as in maintenance cost.[1] The continuous operation has the advantage of less cost, less head loss in operation when compared with fill and draw basin and is more efficient. To facilitate sludge withdrawal and drainage of the tank, the floors will slope radially towards the centre at a rate of one inch per foot.
2. The cleaning will be done mechanically due to the following reasons:
   (i) The mechanically cleaned basins with continuous operated scrapers are not required to be shut down for cleaning and no allowance for sludge accumulation is required to be made in their design.
   (ii) For equal performance they are lower in first cost and in operating cost.[2]

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[1]WPCF Manual on Sewage Treatment Design, op. cit., p.96.

[2]H.E. Babbit, Sewerage & Sewage Treatment, op. cit., p.378.

3. Multiple units will be arranged in group of two and four for taking the present and ultimate discharge. The units will be designed for average rate of flow.
4. The overflow rate will not exceed 1000 gallons per sft of the area per day.
5. The weir loading will not exceed 15,000 gallons per day per linear foot and would remain preferably upto 10,000 gallons.
6. The uniform distribution of the influent will be accomplished by a concentric influent baffle which distributes the flow uniformly towards the effluent. The inlet baffle will have diameter 15 percent of the tank diameter and will extend 4 feet below the surface.[1]
7. The effluent weir will be V-notched type and will extend around the entire periphery of the tank.
8. Baffles will be provided ahead of the overflow weirs to retain floating screen and will extend 8 inches below the water surface.
9. Sludge removal from the hopper will be made by direct connection to pump.

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[1] WPCF, Manual on Sewage Treatment Plant Design, op. cit., p. 100.

10. Scum removal will be done by a radial arm which rotates with the sludge removal equipment. It will be collected at the periphery in a radial trough from it will flow to an outside sump.
11. The sludge hopper will be located near the centre of the tank and will be of sufficient capacity to seal the outlet pipe and avoid entrance of overlying liquid.
12. The inlets and outlets of all the tanks will have exactly the same elevations and capacities so that all the tanks receive a proportionate amount of sewage.
13. The details and dimensions of the supporting structure, and all mechanical equipments etc., will be furnished by the manufacturer.

Design of Grit Chamber

To make a compact unit, screens and grit channel will be provided in one unit. The design of grit channel will therefore be made first, and the design of screens will be checked for the determined dimensions of the channel.

Present population	=	38,000 persons
Ultimate population	=	75,000 persons
Present sewage flow	=	75 l.p.c.p.d.
Future sewage flow	=	30 l.p.c.p.d.

$$Q_{max} = \frac{75,000 \times 75 \times 35.3}{1,000 \times 24 \times 60 \times 60} \times 2.5$$

$$= 6.13 \text{cfs}$$

$$Q_{min} = \frac{38,000 \times 30, \times 35.3}{1,000 \times 24 \times 60 \times 60} \times 0.5$$

$$= 0.233 \text{ cfs.}$$

For velocity control by Partial flume[1]

$$\frac{Q_{min}}{Q_{max}} = \frac{1.1\left(\frac{Q_{min}}{4.1w}\right)^{0.67} - Z}{1.1\left(\frac{Q_{max}}{4.1w}\right)^{0.67} - Z}$$

Where Q_{min} = minimum rate of flow

Q_{max} = Maximum rate of flow

W = Flume throat

Z = Distance as shown in the figure

Assuming the flume with a 6" throat,

$$\frac{0.233}{6.13} = \frac{1.1\left(\frac{0.233}{4.1 \times 0.5}\right)^{0.67} - Z}{1.1\left(\frac{6.13}{4.1 \times 0.5}\right)^{0.67} - Z}$$

$$0.038 = \frac{1.1(0.114)^{0.67} - Z}{1.1\ (3)^{0.67} - Z}$$

$$= \frac{0.246 - Z}{2.3 - Z}$$

$$0.962\ Z = 0.246 - 0.087$$

$$Z = \frac{0.159}{0.962} = 0.165 \text{ ft (5 cm)}$$

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[1]H.E. Babbit, Sewerage & Sewage Treatment, op. cit., p.384.

$$d_{max} = 1.1\left(\frac{Q_{max}}{4.1W}\right)^{0.67} - Z$$

$$= 1.1\left(\frac{6.13}{4.1 \times 0.5}\right)^{0.67} - Z$$

$$= 2.3 - 0.165 = 2.135 \text{ ft (65 cm)}.$$

Volume of Grit

$$Q_{av} = \frac{6.13}{2.5} = 2.45 \text{ cfs}$$

The volume of grit is 4 cft per million gallon.

$$\text{Volume of Grit per day} = \frac{4 \times 2.45 \times 7.5 \times 60 \times 60 \times 24}{1,000,000}$$

$$= 6.35 \text{ cft}$$

For a clearance interval of 7 days,

Volume of grit = 6.35 x 7 = 44.5 cft.

Therefore Depth required for grit collection

$$= \frac{44.5}{2 \times 2.05 \times 20.5}$$

$$= 0.53 \text{ ft (15 cm)}.$$

Therefore depth of grit channel will be

$$= 65 + 15 = 80 \text{ cm}.$$

$$d\ min = 1.1 \left(\frac{0.233}{4.1 \times 0.5}\right)^{0.67} - Z$$

$$= 0.246 - 0.165$$

$$= 0.081 \text{ cft (2.5 cm)}$$

$$\text{Width of Channel} = \frac{Q_{max}}{Q_{max}.v} \text{ or } \frac{Q\ min}{Q\ min.v}$$

$$= \frac{6.13}{2.135 \times 1} \text{ or } \frac{0.233}{0.081}$$

$$= 2.88 \text{ (or 88 cm)}.$$

Provide two channels each 45 cm wide.

Flow should be greater than Qmax in this formula.

$Q_{max} = 130 . W\, N^{3/2}$ $\quad N = 0.375$ ft. for 6" throat

$= 130 \times 0.5 \times (0.375)^{3/2}$

$= 15$ cfs.

Since this rate of flow is larger than the maximum rate of sewage flow i.e. 6.13 cfs, free flow conditions will exist.

For the distribution of velocity in grit channel to be unaffected between minimum and maximum flow,

Area (3) = Area 2 + Area 1

$(88 - 2 \times 2.5) \cdot x = 2.5 \times 2.5$

Therefore $x = \dfrac{6.25}{83} = 0.075$ cm

$B = 1.5\ (Q_{max})^{0.33}$ but should not be less than 2 ft.[1]

$= 1.5 \times (6.13)^{0.33} = 2.73$ ft.

Since it is more than 2 ft, it is OK.

From table

$D = 1.302$ ft

$B = 2$ ft

Ratio $= \dfrac{D}{B} = \dfrac{1.302}{2} = 0.65$

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[1] H.E. Babbit, <u>op. cit.</u>, p. 387.

Ratio of D/B should b e 0.65 in the design.

Therefore D = 0.65 x 2.73 = 1.77 ft = 55 cm.

F = 12 inches = 30 cm.

G = 2 ft = 61 cm

C = 1.302 ft = 40 cm

$Q = 4.1 \, WH_A^{3/2}$

Therefore $HA = \left(\frac{6.13}{4.1 \times 0.5}\right)^{2/3} = 2.09$ ft (64 cm)

At the tail water conditions to prevent submergence

$$d_c + K > dc'$$

$d_c = \sqrt[3]{\frac{Q^2}{gb^2}}$ Where b = breadth at the flume = 0.5 ft.

$$d_c = \sqrt[3]{\frac{6.13}{32.2 \times 0.25}} = 0.91 \text{ ft (or 28 cm)}$$

K for 6" throat = 3 inches = 0.25 ft.

Therefore $d_c + K = 0.91 + 0.25 = 1.16$

$$dc' = \sqrt[3]{\frac{6.13}{32.2 \times (1.302)^2}}$$

$$= 0.485 \text{ ft (or 15 cm)}.$$

Since $d_c + K$ (i.e. = 1.16) is greater than d_c (=0.485), there will be no submergence.

Length of Channel

d max = 2.135 ft

Settling velocity = 0.1 ft/sec.

Time taken for a particle to settle 2.135 depth at the rate of 0.1 ft/sec = $\frac{2.135}{0.1}$ = 21.35 seconds.

For allowing a horizontal velocity of 1 ft/ sec,

Length of channel 21.35 ft (6.5 meter)

Design of Screens

The width of channel adopted = 2.88 ft

C/C spacing of bars = 3 cm = 0.0985 ft.

Therefore number of bars $= \dfrac{2.88}{0.0985} - 1$

Space taken up by 28 bars $= \dfrac{28 \times 1}{30.5}$

$= 0.92$ ft.

Effective width for the screening chamber =

$= 2.88 - 0.92 = 1.96$ ft.

Velocity at peak flows $= \dfrac{Q \text{ max}}{\text{Width of channel} \times d \text{ max}}$

$= \dfrac{6.13}{1.96 \times 2.135}$

$= 1.47$ say 1.5 ft which is OK

Velocity at low flows $= \dfrac{Q \text{ min}}{\text{Width of channel} \times d \text{ min}}$

$= \dfrac{0.233}{1.96 \times 0.081} = 1.47$ ft/sec

say 1.5 ft.

Thus the velocity will not be disturbed and will be enough for no deposition before the screen.

The bars of 1 cm x 4 cm cross section with clear spacing of 2.5 cm between the bars with long dimensions parallel to flow will therefore be adopted.

Design of Sedimentation Tank

$$\text{Present } Q \text{ (Average)} = \frac{38,000 \times 30}{1000} = 1140 \text{ m}^3/\text{day}$$

$$\text{Ultimate } Q \text{ (Average)} = \frac{75,000 \times 75}{1000} = 5625 \text{ m}^3/\text{day}$$

Present water consumption = 30 l.p.c.p.d.

Present BOD of the sewage = 2000 p.p.m. = 2000 gm/litre

= 60 grams/capita/day.

The present BOD in parts per million is very high. The reason for this is the less consumption of water by the people (which is only 30 litres per capita per day). As the water supply position will improve, the BOD will be reduced appreciably. On the other hand, the present per capita contribution of BOD which is 60 grams per capita per day is low, because of poor diets of the people. It is expected to increase as the standard of the people will improve within the design period of 50 years. Looking to the pace of development and therefore expected rise in the standard of living of people, a contribution figure of 70 grams per capita per day is reasonable and has been adopted in the design.

$$\text{Total BOD of the sewage (in future)} = \frac{70 \times 1,000}{75}$$

$$= 934 \text{ p.p.m.}$$

These high concentrations of BOD will require a greater degree or recirculation for the present, which will be reduced as the water supply position will be improved. Assuming present fixed recirculation rate as 8:1 and future recirculation rate as 3:1

$$\text{Future } Q_{av} = \frac{75,000}{1,000} (75 + 225)$$
$$= 22,500 \ m^3/\text{day}$$

$$\text{Present } Q_{av} = \frac{38,000}{1,000} (30 + 240)$$
$$= 10,250 \ m^3/\text{day}$$

Using a tank 45 ft in dia (about 13.7 meter) with 10 ft (3.05 meter) side depth and bottom slope at 1 inch per foot of slope

$$\text{Volume of tank} = 0.785 \times 13.7^2 \left[3.05 + \frac{1.75}{3.28 \times 3} \right]$$
$$= 475 \ m^3$$

$$\text{Detention Period} = \frac{475 \times 24}{4625} = 2.0 \text{ hours}$$

which is OK.

$$\text{Overflow rate} = \frac{5625 \times 1000 \times 4}{3.8 \times 3.14 \times 45 \times 45} = 935 \text{ gallons per sft/day} \ldots \text{OK}$$

$$\text{Weir Overflow rate} = \frac{5625 \times 1000}{3.8 \times 3.14 \times 45}$$
$$= 10,300 \text{ gallons/linear foot/day.}$$

This is OK.

(b) <u>Design for Present Conditions</u>

Provide two tanks.

Q_{av} (per tank) = 5,125 m^3/day

Using the same size of tanks,

$$\text{Detention period} = \frac{475 \times 24}{5125} = 2.23 \text{ hrs} == \text{OK.}$$

$$\text{Weir overflow rate} = \frac{5125 \times 1000}{3.8 \times 3.14 \times 45}$$

$$= 9{,}600 \text{ gallons/linear ft/day.}$$

Diameter of the Inlet Baffle:

Ratio of Inlet baffle diameter to Tank diameter = 0.15

Therefore diameter of Inlet Baffle = 0.15 x 45 = 6.45

= 6.75 ft (2.06 meters)

Depth of baffle = 4 ft.

C. SECONDARY TREATMENT

The effluent resulting from sedimentation usually contains about 45 to 50 percent of the unstable organic matter that was originally present in the raw sewage. It therefore requires further treatment which can best be made by oxidation and nitrification of the organic matter.

There are a number of methods of oxidation of sewage effluent such as contact beds, intermittant sand filteration, activated sludge, trickling filters and oxidation ponds.

The activated sludge and oxidation ponds have already been discussed while selecting the method of treatment in general, and have not been considered suitable. The quality of effluent from intermittant sand filter is though better than any other method and operation is also simple but the process is not suitable for big towns due to the large area and considerable quantity of sand required. The contact beds besides having large areas of land required has also the disadvantage of indifferent quality of the effluent and requires very careful operation. The rate of treatment is very slow and in fact the attempt to increase the rate has led to the production of trickling filter.

Thus the trickling filter being superior in practically all pertinent considerations will be used.

The secondary treatment will therefore consists of the following units:

(1) Trickling filter

(2) Final sedimentation tank.

The design of each of these is dealt with separately as under:

(1) Design of Trickling Filter

There are numerous variables which affect the performance and, thus the design of trickling filters. The factors which are very important are the following:

(a) Composition and characteristics of the raw sewage received at the treatment plant.

(b) Hydraulic and organic loading to be applied to the filter.

(c) Recirculation system.

(a) Composition of Sewage

The sewage is highly concentrated and has at present a

BOD of 2000 p.p.m. It is expected to reduce to 975 p.p.m. after the water supply position is improved. The industrial wastes are very small and do not compose any substantial part of the sewage to be treated.

The hourly, daily, and seasonal variations in both the volume and the strength of the sewage received at the treatment plant will be taken care of by varying the recirculation. Multiple pumping arrangements will be provided to accomodate the low, average and maximum recirculated flows to the raw sewage so that the flow to the treatment plant is almost constant.

(b) Hydraulic and Organic Loadings

The loading rate of a filter is an important design factor, because the filters are classified according to the applied hydraulic and organic loadings. Although there is no well defined practice for the loadings, the following ranges have been recommended by a FSIWA Subcommittee on Units of Expression, and will be followed in this design.[1]

	Low-rate Filters	High-rate Filters
Hydraulic Loadings		
Gallons per day per sft	25 to 100	200 to 1,000
Millions gallons per acre per day	1.1 to 4.4	8.7 to 44
Organic Loading		
Pounds per 1,000 cubic feet per day	5 to 25	25 to 300
Pounds per acre-foot per day	220 to 1,100	1,100 to 13,000

[1]WPCF Manual on Sewage Treatment Plant Design, op. cit. p. 155.

The above ranges show that the rate of hydraulic loading in high rate filters is 8 to 10 times higher than that of low rate filters. The high rate filters are therefore low in initial cost and require less space and will, therefore, be provided in the treatment plant of Hebron. In addition, it will have the following further advantages.

1. The effect of sewage temperature on high-rate filter is less than that of low-rate filters because of increased hydraulic and organic loading.

2. The head required to operate a high-rate trickling filter is less than that required by a low-rate filter.

Single stage filters will be adopted due to the following reasons:

(1) The BOD of the settled effluent is more than 30 mg/l and the applied load, recirculation included will not exceed 110 lb of BOD per 1000 cft per day.[1]

(2) The desired BOD can be obtained by single stage filteration.

(3) The construction and maintenance cost of the single stage of filteration is much less than the double stage filteration.

(c) Recirculation System

Recirculation though involves extra first cost and also some proportion of operating cost is an accepted method of increasing the BOD removal efficiency and will be provided for

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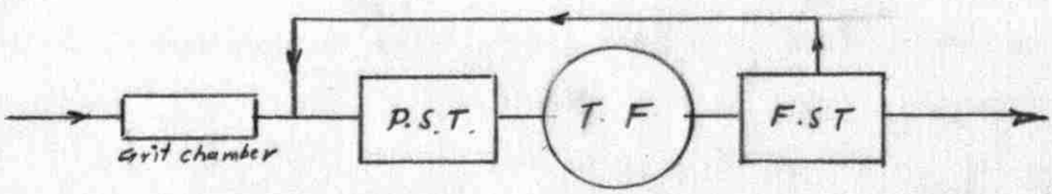
[1] E.W. Steel, op. cit., p. 528.

the following reasons:

1. Organic matters in recycled filter effluent is brought in contact with active biological material on filter more than once. This will increase the contact time and seed the filter throughout its depth with a large variety of organisms.
2. It will allow continuous dosage of the filters regardless of fluctuations in flow and thus keep the bed working more nearly continuously. It will also dilute the strong sewage and will supplement the weak sewage.
3. It will freshen the influent and reduce odours.
4. It will improve distribution over the surface of filter, reduce tendency to clog, and due to being sufficiently high, will also aid in controlling filter flies.
5. It will equalize and reduce loading applied to the filter over a 24 hour period and will thus improve the efficiency.

Flow Diagram

Out of the various systems of recirculation used either individually or in combination, the flow diagram as shown below has been adopted due to the following reasons:



(1) Recirculation through primary tanks will freshen the stale sewage and reduce the scum formation.

(2) The recirculated effluent from the sludge hopper of the secondary settling tank will remove sludge and will thus also reduce depletion of oxygen in plant effluent.

(3) The recirculated flow passing through the settling tank will dampen the variation in loadings applied to the filter over a 24-hour period.

(4) The capacity of the final settling tank will be required only for the balanced flow.

(d) Design Criteria

The design criteria will be as follows:

(1) The shape of the filter will be circular.

(2) The depth of the filter will be 6 feet at the centre and will be increased at the side walls due to the slope in underdrains. The walls will be of reinforced cement concrete and will be 12 inches thick.

(3) The hourly, daily and seasonal variations will be met by varying recirculated flow by means of multiple pumping arrangement. The operation will thus be continuous.

(4) The filter medium will consist of crushed stone which will be sound, hard, and free of dust. The size of the stone will be such that 95 percent or more of the media will pass 4 inches square mesh screen and will be retained on

2 1/2 inch square mesh screen. The pieces will be uniform in size with all three-dimension as nearly equal as possible and will be placed in layers of uniform size with small size stones at the tope of the bed.

(5) The ventilation will be provided with vents or risers on the under drain system so as to maintain the aerobic conditions necessary to secure effective treatment.

(6) The arrangements for flooding the filter will be made for controlling the "filter flies" and "ponding". The walls will be watertight and drainage channel will be provided with gates.

(7) The filter blocks will be of vitrified clay or concrete and will be placed in parallel lines perpendicular to the drainage channel.

(8) The drainage channel will be provided to carry the flow from underdrains and to admit the air to the underdrains for ventilation. The channel will be designed to provide a velocity of 2 to 3 fps and will be rectangular in cross section.

(2) Final Sedimentation Tank

The trickling filter removes only a small percent of the solids that are applied to it in the effluent from the primary settling tank. The principle function of the filter is to change the character of the suspended solids, rather than to remove suspended matter. The suspended solids in the sewage applied to the filter are finely divided and will not settle readily. As a result of oxidation in the filter the solids are changed to a form which being heavier and bulkier, will settle if passed through a tank. The effluent from the trickling filter will therefore be treated by settling in a final sedimentation tank.

The following design criteria will be adopted.

1. The final sedimentation tank will be designed to operate on a continuous flow basin and will be circular in shape.

2. The cleaning will be done manually. The manual cleaning is prefered to reduce the first cost as well as the maintenance cost. The tank will be hopper bottom with steep slopes of 45 degrees to the horizontal so as to facilitate sludge withdrawal.

3. The overflow rate will not exceed 800 gallons per sft. of the area per day.

4. The weir overflow rate will not exceed 15,000 gallons per day per linear foot and would remain preferably upto 10,000 gallons.

5. The uniform distribution of the influent will be accomplished by an concentric influent baffle which distributes the flow uniformly towards the effluent.

6. Two units will be provided for the ultimate flows out of which one will be sufficient for the present.

Design of Trickling Filter (For Combination No. 2)

Present BOD of the sewage = 2000 p.p.m.

Future BOD of the sewage = 934 p.p.m.

Since the initial BOD's of the sewage are high, high recirculation rates will be required for reducing the BOD of the effluent to the allowable rate. The proposed BOD of the effluent is therefore to be so chosen that the effluent is not objectionable and the future flows can be taken care of by increasing the number of units without altering the flow in the units.

The design has therefore been made for two different allowable BODs of the effluent viz. 40 p.p.m and 50 p.p.m. with varying recirculation rates. The design calculation for different combinations have been made as shown in table No. 7. It will be seen that the following three combinations are suitable for adoptation.

1. Combination No. 2 and 6.

Proposed BOD of the effluent = 40 p.p.m.

Present recirculation required = 9:1

Future recirculation required = 4:1

Present flow/tank = 5700 m^3/day

Future flow/tank = 7050 m^3/day.

2. Combination No. 4 and 8

Proposed BOD of the effluent	= 50 p.p.m.
Present recirculation	= 8 : 1
Future recirculation	= 3 : 1
Present flow/tank	= 5125 m^3/day
Future flow/tank	= 5625 m^3/day

3. Combination No. 3 & 5

Present proposed BOD	= 50 p.p.m.
Future Proposed BOD	= 40 p.p.m.
Present recirculation	= 8 : 1
Future recirculation	= 4 : 1
Present flow/tank	= 5125 m^3/day
Future flow/tank	= 7050 m^3/day

Since the difference in the present and future quantity of flow is less in the combination No. 2 and 6 this combination is most suitable and would be adopted. The above three combinations, however, show that the design is flexible enough and the actual operation will be efficiently possible by any of these.

The details of the design of the combination No. 2 and 6 are given below. The design for other combinations have been worked out similarly and the results are tabulated in the table No. 7 for comparison.

TABLE NO: 7

TABLE SHOWING DESIGN OF TRICKLING FILTERS WITH DIFFERENT RECIRCULATIONS

SHEET NO.

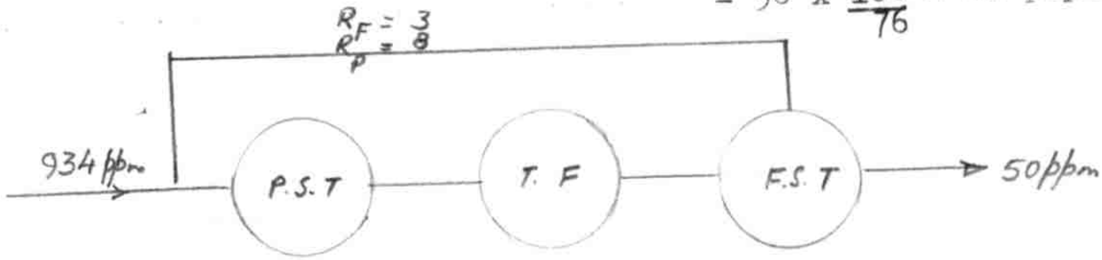
SERIAL NUMBER	B.O.D APPLIED TO THE PLANT in p.p.m	PROPOSED B.O.D OF THE EFFLUENT in p.p.m	PROPOSED RE-CIRCULATION FACTOR	B.O.D OF THE RE-CIRCULATED EFFLUENT in p.p.m	AVERAGE B.O.D APPLIED TO Pr: SED: TANK in ppm:	PRIMARY SEDIMENTATION TANK		B.O.D APPLIED TO TRICKLING FILTER		TRICKLING FILTER (75 ft: DIA:, 6 ft: DEEP) (VOLUME = 26,500 cft)			TRICKLING FILTER (100 ft: DIA, 6 ft: DEEP) (VOLUME 47,000 cft)			AVERAGE FLOW PER TANK in m^3/day	REMARKS
						DETENTION PERIOD in HOURS	B.O.D REMOVAL %	IN p.p.m:	Total BOD in lbs	ORGANIC LOADING lbs/1000cft	EFFICIENCY OF FILTER & F.ST. %	REMAINING B.O.D in p.p.m	ORGANIC LOADING lbs/1000cft	EFFICIENCY OF FILTER & F.S.T %	REMAINING B.O.D p.p.m		
							FOR PRESENT CONDITIONS.										
1.	934	40	3	57	276	2.0	36	177	2190	82.5	66.5	60	46.6	72	48.7	5625	
2.	"	"	4	56	232	2.0	"	148	2310	87	66.0	50	49	72.5	41	7050	
3.	"	"	5	56	202	"	"	129	2400	90	65	45	51	72	36	8450	
4.	"	50	3	71	287	"	"	184	2260	85	66	62.5	48	72	51.5	5625	RECOMMENDED
5.	"	"	4	70	240	"	"	154	2390	90	65	54	53	71.5	44	7050	
							FOR ULTIMATE DESIGN CONDITIONS.										
1.	2,000	40	9	54	249	2.5	40	149	1870	73	67	52	40	73	40	5700	RECOMMENDED
2.	"	"	10	60	236	"	"	146	2020	75	67	48	42	73	39.4	6270	
3.	"	50	8	68	283	"	"	170	1920	72.5	67	56	41	73	46	5125	

Design

Assuming a detention period of 1 1/2 hour in the Final Sedimentation Tank, the percentage removal of suspended matter and 5 day BOD by settling against 50-100 p.p.m. curve = 24 %

BOD applied to final sedimentation tank is, therefore

$= 50 \times \frac{100}{76} = 66$ p.p.m.



Design for Future Conditions

Assume a recirculation of 3 : 1

The effluent for the recirculation will be taken from the bottom of the final ssedimentation tank. Since the BOD applied to the final sedimentation tank is 66 p.p.m. whereas the effluent discharged out of it is 50 p.p.m., the BOD of the recirculated effluent $= 66 + \frac{66 - 50}{3} = 71$ p.p.m.

Average BOD applied to Pr. Sed. Tank $= \frac{934 + (71 \times 3)}{4}$

$= 287$ p.p.m.

Percentage BOD removal by Pr. Sed. Tank for 2 hours detention time ... ... $= 36$ %

Therefore BOD applied to filter $= 287 \times 0.64 = 184$ p.p.m.

Average Q $= \frac{75,000}{1,000} (75 + 225)$

$= 22500\ m^3/day$

Provide 4 tanks.

Flow/tank = 5625 m^3/day.

Total BOD applied to filter = $\frac{184 \times 5625}{454}$

= 2260 lbs.

Adopt diameter of filter as 100 ft and depth as 6 ft

Volume of filter = $\frac{3.14}{4} \times 100 \times 100 \times 6$

= 47,000 cft.

Organic loading = $\frac{2260}{47500} \times 1000$

= 48 lbs/1000 cft.

This is OK

Efficiency of Trickling Filter and Final Sedimentation Tank at this organic loading with R as 0[1] = 72%

Therefore Remaining BOD = 0.28 x 184

= 51.5 p.p.m.

This is OK

<u>Hydraulic Loading:</u>

Applied sewage = 5625 m^3/day

= 1.49 mgd.

Area of filter = 7850 sft.

Hydraulic loading = $\frac{1.49 \times 43560}{7850}$ = 8.26 m.g.a.d.

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[1] Ibid., p. 528 (Fig. 24-3)

Design for Present Conditions

Assume Recirculation as = 8:1

BOD applied to the Final Sed. Tank = 66 p.p.m.

BOD of the recirculated effluent $= 66 + \frac{66-50}{8} = 68$ ppm.

Average BOD applied to Filter $= \frac{2000 + 8 \times 68}{9}$

= 283 p.p.m.

BOD removal by Pr. Sed. Tank (with 2-5 hour detention) = 40 %

Therefore, BOD applied to filter = 0.60 x 283

= 170 p.p.m.

Average Q $= \frac{38,000}{1,000}(30 + 240)$

$= 10,250\ m^3/day$

Provide 2 tanks, Q/tank $= 5125\ m^3/day$

Total BOD $= \frac{170 \times 5125}{454} = 1920$ lbs.

Organic Loading/1000 cft= $\frac{1920}{47000} \times 1000 = 41$ lbs.

Efficiency of Trickling filter and final sedimentation

Tank at 41 lbs per 1000 cubic feet = 73%[1]

Therefore Remaining BOD = 0.27 x 170

= 46 p.p.m.

Since this is less than the allowable, it is OK.

Hydraulic Loading

Applied Sewage $= 5125\ m^3/day$

= 1.36 m.g.d.

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[1] *Ibid.*, p. 528 (Fig. 23-3).

Hydraulic loading $= \frac{1.36 \times 43560}{7850}$

$= 7.52$ m.g.a.d.

This is OK.

Design of Pipes

(a) Inlet Pipe for the Trickling Filter

Future flow/filter $= 7050\ m^3/day = 2.89$ cfs.

Present Flow/filter $= 5700\ m^3/day = 2.34$ cfs.

Adopt 12" size, area = 0.785 sq. ft.

Future velocity $= \frac{2.89}{0.785} = 3.68$ ft/sec.

Present velocity $= \frac{2.34}{0.785} = 2.98$ ft/sec.

Since these are less than permissible maximum velocity of 4 ft/sec.[1] the size of pipe adopted is OK.

Distribution Pipe System Over the Tank

Providing Rotary reaction type distributor with 4 arms at right angles.

Ultimate Flow/filter = 2.89 cfs.

Flow/pipe $= \frac{2.89}{4} = 0.72$ cfs

Provide 9" size pipe, area $= 0.785 \times 0.75 \times 0.75$

$= 0.44$ sq. ft.

Vel $= \frac{0.72}{0.44} = 1.64$ ft/sec.

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[1]WPCF Manual on Sewage Treatment Plan Design, op. cit., p. 172.

Since it is less than 4 ft/sec, it is OK.

For economy and uniform distribution of flow, the 9-inch pipe at the centre of the tank will be reduced towards the other end because the flow is progressively reduced.

Size and Number of Nozzles

The discharge through spray nozzles can be represented by the standard Orifice Formula.[1]

$$Q = Cn.an \sqrt{2\,gh_n}$$

Where Q = Discharge in cubic feet per sec.

C_n = Coefficient of discharge = 0.65

a_n = area of nozzle in sq. ft.

g = acceleration due to gravity

h = head on centre line of office = 6 cft.

$$Q = 0.65 \times 0.785 \times \frac{1}{144}\left[\left(\frac{7}{8}\right)^2 - \left(\frac{3}{8}\right)^2\right] \times \sqrt{64.4 \times 6}$$

$$= 0.0435 \text{ cfs} = 19.6 \text{ g.pm.}$$

Number of Nozzles required per arm

$$= \frac{\text{Flow per pipe}}{\text{Flow taken up by one Nozzle}} = \frac{0.72}{0.0435}$$

$$= 16.6 \text{ say } 17 \text{ nos.}$$

Length of one arm $= \frac{100}{2} = 50$ ft.

Spacing between nozzles $= \frac{50}{17} = 3$ ft.

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[1] L. Metcalf and H.P. Eddy, American Sewerage Practice. Vol. II, (New York; McGraw-Hill Book Co., Inc., 1935), 3rd edn., p. 515.

Provide nozzles at 3 ft C/C.

Depth of Undergrains

Radius of filter = 50 ft.

Provide slope = 1 %

Height in the middle of the Trickling filter = 6 ft.

Height at ends = 6 + 0.5 = 6.5 ft.

Effluent Channel Pipe

Discharge Q = = 2.89 cfs

Minimum permissible velocity = 2 ft/sec.

Recommended velocity = 4 ft/sec.

Area of pipe = $\frac{2.89}{4}$ = 0.72 sft.

Diameter of pipe = $\underline{4 \times 0.72}$ = 0.96 ft.

Provide 12" pipe

Channel Periphery

Area of section required = $\frac{0.72}{2} \times 144$ = 52 inches.

Adopt section = 9" x 6".

Design for Final Sedimentation Tank.

For Future Conditions

Discharge = 22,500 m^3/day

Quantity taken out for recirculation = 22,500 x 3/4

= 16,900 m^3/day

Quantity to be treated by Final Sedimentation Tank = 5600 m^3/day.

Provide two tanks, flow/tank = 2800 m^3/day

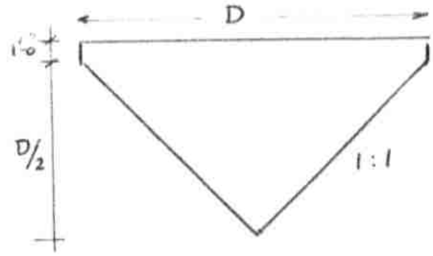
Provide Detention period 1 1/2 hours

Volume of tank = $\frac{2800 \times 1.5}{24}$ = 175 m^3 = 6200 cft.

Volume of tank = $0.785\ D^2 (1+\frac{D}{6})$

62,000 = $0.785D^2 + 0.131D^3$

Therefore D = 34.5 ft or 35 ft



Overflow rate = $\frac{2800 \times 1000 \times 4}{3.8 \times 3.14 \times 35 \times 35}$

= 770 gallons/sft/day ... OK.

For Present Conditions

Q = 10250 m^3/day

Quantity taken out for recirculation = $\frac{10,250 \times 8}{9}$

= 9120 m^3/day

Quantity to be treated by F.S. Tank = 1130 m^3/day

Detention period = $\frac{175 \times 24}{1130}$ = 3.72 hours or

3.75 hours.

Overflow rate = $\frac{1130 \times 100 \times 4}{3.8 \times 3.14 \times 35 \times 35}$

= 315 gallons/sft/day ... **OK**

Weir Overflow rate= $\frac{1130 \times 1000}{3.8 \times 3.14 \times 35}$ = 2700 g/linear foot/day.

= This is OK.

D. Sludge Treatment

The suspended solids and some of the organic dissolved solids which are in the form of sludge or scum create the most difficult portion of the disposal problem in a treatment plant because their polluting potentialities are generally greater than those of liquid effluent and their volume is enormous. So before disposing them finally these require treatment in the treatment plant for (a) reduction of the volume of sludge to be disposed of and (b) control or destruction of its put-rescibility.

Of the unit operations employed for the sludge treatment, the air drying of sludge after digestion will be adopted for this project due to the following reasons:

(1) It is the most economical and simple method.

(2) It breaks down the complex organic matter present in the sludge into simple compounds.

(3) It reduces the quantity of sludge to be disposed of.

(4) It makes the odour of the sludge inoffensive.

(5) The sludge gives up its remaining water more easily, and drying on beds can be accomplished economically.

(6) Digested sludge when dried is more useful for agri-culture purpose than raw sludge, having lost much of its oil contents.

(6) Coliforms are reduced.

Design of Digestion Tank

The major features to be considered in the design of a separate digestion tank are

(a) Volume of sludge that will be digested daily.

(b) Period of digestion.

(c) Method of heating.

(d) Type of tank.

(e) Method of stirring.

(f) Method of adding and removing of sludge and supernatant.

(g) Type of cover.

(h) Method of gas collection.

Each of these are described briefly as under.

(a) Volume of Sludge

The quantity of sludge depends upon the amount and character of the solids in the sewage treatment process. Since these characteristics are not known, the volume of sludge cannot be determined precisely. An estimation of the quantity of sludge has, however, been made by assuming the figures for the normal domestic sewage, and appears with the design.

(b) Period of Digestion

The principal factor affecting the period of digestion or the rate at which digestion of normal domestic sewage sludge takes place is the temperature maintained in the digestion tank. As the temperature increases, the rate of digestion increases, thereby reducing the capacity of the digester. There are two significant temperature zones in which the digestion takes place rapidly. Firstly, a zone of high temperature in which the heat loving organism (thermophlic) are responsible for digestion and secondly the zone of moderate temperature in which the common (misophilic) organism are active. The upper zone is rarely put to use in sewage works because of odour and other operating difficulties and the most practical optimum temperature for sludge digestion is between 80° and 90° F in the mesophilic range.

The temperature of Hebron ranges averagely between 30° C (86° F) in summer to 5° C (or 41° F) in the winter. The temperature in the summer is ideal for digestion to take place without the aid of heating. In the winter, the temperature is though not ideal but the digestion will take place when the temperature in the day time will increase. The heating will therefore not be done due to the following reasons:

(1) The cost of providing extra capacity for digestion at a temperature of 41° F will be less than the extra equipment required for heating and insulating at a higher temperature. It is due to the fact that the total size of sludge digestion and sludge storage tank will not be directly proportional to

the time required for digestion.[1]

(2) Skilled careful operation will not be required for controlling the temperatures.

(3) The maintenance cost will be low.

Type of the Tank

In the process of sludge digestion the digestive action is most rapid in the first few days and then it is slowed down. The digestion of sludge (80 to 90 percent) will therefore be made in the primary tank where mixing of fresh sludge with old material in the tank will be done; and only the ripening of the sludge will be made in the secondary tanks. This would provide the following advantages:

1. The difficulty of drawing supernatant liquor from a single tank due to the gas production and mixing will be overcome, because the separation of supernatant liquor will be best obtained from the secondary tank.

2. The initial cost will bee less as the primary tank will only be covered and equipped with stirring arrangements.

3. The secondary tank would serve to equalize fluctuations that occur with sludge drying.

4. It will reduce the tendency of sludge to short circuit and pass through untreated.

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[1]H.E. Babbit, op. cit., p. 516

5. No measures for preventing the scum formation will be required. A moderate amount of scum if formed will, however, be desirable, because the odours from the sewage surface will be held back and certain amount of heat insulation will be obtained.[1]

(e) Methods of Stirring

The seeding of material to be digested with actively digesting material is essential for rapid sludge digestion. It is necessary, therefore, that each doze of fresh sludge should be mixed well with older material in the tank. To assure this thorough mixing the following three methods are usually adopted.

1. Recirculation of tank content by pumping.
2. Recirculating digester gas by means of compressor and releasing it through a number of vertical pipes below the lower scum level.
3. Mechanical mixing devices.

The recirculation of tank contents require a heat exchanger and a pump. It also requires a constant attendance and being costly is not suitable to be adopted. The recirculation of digester gas is out of question, since the gas will not be stored. The third method, i.e. the mechanical mixing

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[1] Metcalf and Eddy, American Sewage Practice, Vol. III, (New York: McGraw-Hill Book Co., Inc., 1935), p. 365.

device is the simplest and will, therefore, be adopted.

The mixing device will consist of a simplex Screw Pump[1] and a vertical uptake tube within the tank which will extend from just below sludge level to the base of the tank. The screw pump will draw sludge up the vertical tube from the body of the tank and the rotating conical disperser will scatter the sludge over the entire surface of the digesting sludge. Since too much mixing is not good, the pump will not work continuously but will be controlled automatically by time switch, operating once in each hour for a pre-selected period of upto 10 minutes duration. The period of digestion will be so adjusted that complete tank contents are turned over daily and thus thoroughly mixed. The scum which tends to form on the surface will be broken up; the gas bubbles will be freed rapidly and even temperature conditions will be maintained throughout the mass of the sludge.

By means of a reversing switch, the screw pumps will also be used to circulate sludge down the tube, thereby sweeping floating scum into the body of the tank. Thus the rapid flow of sludge down the tube will entrain large volumes of gas which create an intense diffusion of gas bubbles and vigorous disturbance of the sludge surface.

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[1]An equipment for Simplex Sludge Digestion Plants by Ames Crosta Mills and Co., Ltd., Heywood, Lancashire, Publication 70, p. 31.

Method of Adding and Removing Sludge and Supernatant

The crude sludge will be pumped daily into the primary tank through a pipe which passes over the tank wall and discharges below the surface of the sludge. As crude sludge will enter into the primary tank a corresponding volume of digested sludge will be displaced through an overflow pipe into a sight box and will flow through a suitable range of piping to the secondary tank. An auxiliary sludge draw-off will also be provided in the bottom of the primary tank for using under controlled condition, when it is safe to lower the level in the primary tank. The controlling sluice valve will be padlocked to stop unauthorized use.

The quiescent condition in the secondary tank will allow the separation of supernatant liquor. The draw off pipes with controlling valves will be provided at varying levels in the tank to allow the supernatant to be removed. The supernatant liquor will be returned to the inlet of the treatment plant for purification with the sewage. The volume of the supernatant liquor will not be substantial and will therefore not effect the load on the plant. However, if it is more, it will be fed into the sewage at a steady rate over a long period.

The removal of supernatant will thicken the sludge, which will be passed to the drying bed through a pipe as shown in the drawing No. 18.

Type of Cover

The primary tank in which digestion will take place will be covered by the fixed concrete roof. The covering of primary tank is essential because of offensive odours which will arise during mixing and digestion of sludge. The covering will also reduce heat losses from the surface and keep the scum layer more fluid. The fixed type of cover will be provided due to the following reasons:

1. It is low in cost.
2. It gives satisfactory service.
3. The danger of drawing air into the tank will be overcome by drawing the digested sludge equal to the volume of raw sludge received in the primary tank. The supernatant will not be drawn from the primary tank.
4. No maintenance and operation will be required.

Method of Gas Collection

The gas given off by the actively digested sludge will be collected in a gas collecting dome which will be provided on the flat portion of the concrete roof. As the heating of sludge will not be done, the gas is not of much use. The gas holder will therefore not be provided and the gas will be drawn off by a pipe and burned by means of a gas burner.

Design of Sludge Digestion Tank

The volume of the sludge produced is calculated as under:

(a) Volume of Sludge in the Primary Sedimentation Tank

Total settleable solids per capita per day in domestic sewage is given as under.[1]

Volatile solids	...	39 grams.	72.2%
Fixed solids	...	15 grams.	27.8%
Total	=	54 grams..	100%

Total weight of solids = $\frac{75,000 \times 54}{454} = 8930$ lbs.

Specific gravity of solids = $S_s = \frac{250}{100 + 1.5\, p_v}$

Where p_v = Percentage volatile matter in the sludge.

$$S_s = \frac{250}{100 + (1.5 \times 72.2)} = 1.2$$

Volume of Wet sludge[2] = $\frac{W_s}{w}\left[\frac{100 \cdot S_w + p(S_s - S_w)}{(100-p) \cdot S_s \cdot S_w}\right]$

Where W_s = Total weight of dry solids in lbs.

p = Percentage moisture content of the sludge (assumed as 95%).

w = Unit weight of water in lbs.

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[1]G.M. Fair and J.C. Geyer, Water Supply & Waste-Water Disposal, (New York: John Wiley & Son, Inc., 1956), p. 563.

[2]*Ibid.*, p. 760 (Egn: 26.5).

S_s = Specific gravity of solids

S_w = Specific gravity of water (assumed as 1)

$$= \frac{8930}{62.4}\left[\frac{100 + 96\ (1.2 - 1.0)}{6 \times 1.2 \times 1}\right]$$

= 2380 cft daily.

2. Sludge After Trickling Filter From Settling Tank

The daily per capita production of non-settleable suspended solids is 36 grams[1] (26 grams being volatile and 10 grams fixed).

Assuming that 7.5% of the weight of the volatile solids is destroyed during filteration and that 87.5% of the remaining weight of solids is captured in the sludge.[2]

Volume of volatile solids remaining = 0.875 x 26 x 0.925
= 21 grams .. 70%

Fixed solids = 0.875 x 10 = 9 grams .. 30%

Total sludge = 30 grams.

$$\text{Total dry solids} = \frac{30 \times 75{,}000}{454} = 4960 \text{ lbs}$$

$$\text{Specific gravity } S_s = \frac{250}{100 + (1.5 \times 70)} = 1.219$$

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[1]*Ibid.*, p. 563 (Table 20-4).

[2]*Ibid.*, p. 770.

The percentage of solids in the fresh humus sludge of trickling filter is 5 to 10%.[1] Assuming 5% solids,

$$\text{Volume of wet sludge} = \frac{W_s}{w}\left[\frac{100 \cdot S_w + p(S_s - S_w)}{(100-p)\ S_s \cdot S_w}\right]$$

$$= \frac{4960}{62.4}\left[\frac{100 + 95\ (1.219 - 1.0)}{5 \times 1.219 \times 1}\right]$$

$$= 1570 \text{ cft daily.}$$

3. Combined Primary & Filter Sludge - Fresh

By addition, the position of total dry solids in the combined sludge will be as under:

	Primary Sedimentation Tank	Final Sed. Tank	Total	Percentage of Total
Volatile	39 grams	21 gm.	60 gm.	71.4 %
Fixed	15 grams	9 gm.	24 gm.	28.6 %
Total dry solids	54 grams	30 gm.	84 gm.	100 %

Sp. Gravity of Solids, $S_s = \frac{250}{100 + (1.5 \times 71.4)} = 1.206$

The fresh humus from trickling filter mixed with primary sedimentation tank sludge has 3-6% solids.[2] Adopting average figure 4%

Total solids $= \frac{84 \times 75,000}{454} = 13,900$ lbs.

[1]Ibid., p. 770 (Table 26-2).

[2]Ibid., Table

$$\text{Volume of wet sludge} = \frac{13,900}{62.4}\left[\frac{100 + (96 \times 0.206)}{4.0 \times 1.206}\right]$$

$$= 3500 \text{ cft.}$$

4. Primary Sludge Digested

Assuming that 67 percent of the volatile matter is destroyed, 25 percent being converted to fixed solids.[1]

$$\text{Volume of volatile solids} = (1-0.67) \times 39$$

$$= 13 \text{ grams} \ldots 37\%$$

$$\text{Fixed solids} = 15 + 0.25\ (39 - 13) = 22 \text{ grams} \ldots 63\%$$

$$\text{Total} \ldots \qquad = 35 \text{ grams}$$

$$\text{Total weight of solids per day} = \frac{75,000 \times 35}{454} = 5,790 \text{ lbs.}$$

$$S_s = \frac{250}{100 + (1.5 \times 37)} = 1.607$$

The primary digested sludge has 10 to 15 percent sludge solids.[2] Adopting 13%

$$\text{Resulting volume of sludge} = \frac{5790}{62.4}\ \frac{100 + (87 \times 0.607)}{13 \times 1.607}$$

$$= 676 \text{ cft.}$$

5. Combined Primary & Filter Sludge Digested

Assuming 67% of the volatile matter destroyed during digestion, and 25% of the remaining being converted into fixed

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[1] Ibid., p. 769.

[2] Ibid., p. 770 (Table 26-2).

solids[1]

Remaining volatile solids = $(1 - 0.67) \times 60 = 20$ gm. 37%

Total fixed solids $= 24 + 0.25\,(60-20) = 34$ gm. 63%

Total ... ... = 54 gm.

$$\text{Total solids} = \frac{75{,}000 \times 54}{454} = 8930 \text{ lbs.}$$

$$S_s = \frac{250}{100 + (1.5 \times 37)} = 1.61$$

Assuming 92% water content in the digested sludge[2]

$$\text{Volume of wet sludge} = \frac{8930}{62.4}\ \frac{100 + (92 \times 0.61)}{8 \times 1.61}$$

$$= 1735 \text{ cft.}$$

Capacity of Digestion Tank

Capacity of digestion tank is given by the formula[3]

$$C = Vf - 2/3\ (Vf - Vd)\ t$$

Where C = Capacity in cubic feet per capita

Vf = Volume per capita of fresh sludge

Vd = Volume per capita of digested sluge

t = Time required for digestion in days.

Average temperature in summer = 30° C or 86° F

Average temperature in Winter = 5° C or 41° F

Designing for the winter temperature which is critical,

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[1] Ibid., p. 769.

[2] Ibid., p. 770 (Table 26-2).

[3] Ibid., p. 776.

For 41° F, the digestion period = 100 days[1]

$$Vf = \frac{5500}{7500} = 0.0734 \text{ cft}$$

$$Vd = \frac{1735}{75,000} = 0.0231 \text{ cft.}$$

$$C = 0.0734 - 0.67\ (0.0734 - 0.0234) \times 100$$

$$= .04 \times 100 = 4.0 \text{ cft per capita.}$$

Since the sludge is usually not removed as rapidly as it is digested, the capacity of the digestion tank in addition to this should also be sufficient to provide for the following:

(a) Concentration of the digested sludge.

(b) Separation of supernatant liquor.

(c) Storage of the digested sludge during periods when the facilities do not permit ready disposal i.e. in winter.

Out of these the first two are normal variations and are included in the basic capacity valve of 4 cft per capita.[2] The winter storage may also be taken care of by the greatest storage capacity which will be available at the end of summer season when the sludge level will be at its minimum elevation.[3] However, to be on the safe side it is worthwhile to add 25% allowance for these variations.

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[1]E.W. Steel, op. cit., p. 577 (Fig. 26.1).

[2]K. Imhoff and G.M. Fair, op. cit., p. 216.

[3]Ibid., pp. 216-17.

Per capita capacity of digestion tank = 5 cft per capita.

Total capacity required for future = 5 x 75,000

= 375,000 cft or

= 10,600 m^3.

Capacity required for present conditions

= 5 x 38,000 = 190,000 cft

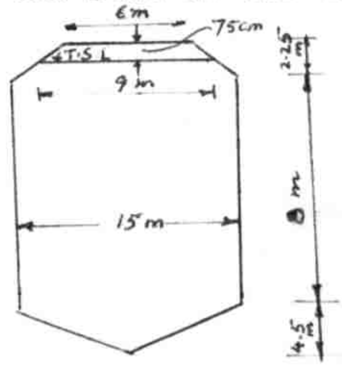
or 5370 m^3

The capacity required for the secondary tank will be less. So adopting a ratio of primary and secondary tank capacities as 2:1 and adopting 6 tanks of equal volume out of which four will act as primary and two will act as secondary.

The capacity of each tank = $\frac{10,600}{6}$

= 1765 m^3

Adopt a circular section with similar dimensions as recommended by Ames Crosta Mills and Co., Ltd., Lancashire, vide publication No. 70, so as to get facility in procurement of mechanical equipment, the size of the digestion tank will be as under:



$$\text{Volume of tank} = \left[\frac{\pi}{4} \times 15^2 \times (8 + 1.5)\right] + \left[\frac{\pi}{8}(15^2 - 9^2) \times 1.5\right]$$
$$= 1680 + 85 = 1765 \ m^3.$$

Design of Sludge Pump

Volume of Combined Primary & Filter Sludge (From page 116)

= 5500 cft.

The sludge will be drawn every 6 hours.

If the pump is operated for 15 minutes

Capacity of pump in g.p.m. $= \frac{5500 \times 7.5}{4 \times 15}$

= 688 g.p.m. or 700 g.p.m.

Static Head required for the pump (from Hydraulic Profile drawing No. 13). = 819.5 - 805

= 16 meters or 52 ft.

From Nomogram, for 8" size pipe

Velocity = 4.5 ft/sec.

hf = 1.3 %

Frictional losses in the 8" diameter rising main

Length of the rising main	= 94 meters
Entrance losses = 12 ft	= 4 meters
2-90° Bend = 2 x 20 = 40 ft	= 12 meters
Losses inside the pumping station	= 2 meters
Total ...	= 112 meters

hf = 112 x 1.3 = 145 cm = 4.7 ft, say 5 ft.

Total head = 52 + 5 = 57 ft.

The performances curve of Peerless pump show as under Curve No. V 4303, size 5 x 6 x 3 - 1/8L

Efficiency 70%, RPM 970, 3 1/8" sphere.

20 HP, Q = 700 gpm at 57 ft head.

This pump or its equivalent will be used.

Recirculation Pumping Station

The design of the sewage treatment plant necessitated the recirculation of treated effluent to the treatment plant. Since the flow through the treatment plant is by gravity a recirculation pumping station is required to pump the effluent to the grit channel sump which is at a higher level. The pumping station will include the following structures:

(a) Wet well

(b) Dry well

(c) Control Room

(d) Pumping units and pressure sewer.

Wet Well. A wet well is provided to act as an equalizing basin for minimizing the fluctuations of load on the pump and for making the automatic operation possible with simple controls. In this particular case, a wet well may even be ignored because the flow through the treatment plant will practically be at a constant rate. However, it will be preferable to provide a wet well to make allowance for the operating levels and to make the operation automatically possible with simple controls.

The wet well will be constructed partly underground and partly above ground as shown in drawing No. 20. The bottom of the well will be sloped towards pump suction steeply (1:1) to prevent the collection of deposit on the floor of the well.

The well will be divided into two portions to facilitate cleaning and repair. An emergency bypass will be provided in the wet well so that under extreme conditions when all the pumps including standby and diesel sets are put out of operation, the sewage is allowed to flow to the low areas or fields nearby.

The size of the wet well will be such that it provides facility for cleaning and inspection. So, the width of the wet well will be 1.20 meter (or 4 ft.), and the length equal to the length of the dry well which will be governed by the size and number of pumps to be installed.

(b) Dry Well. It is proposed to install the pumps in a dry well which will be adjacent to the wet well. This will have the following advantages:

(i) Corrosion of the pump will be reduced.

(ii) Servicing and maintenance will be easy.

(iii) The pumps will be self-primed.

The size of the dry well will depend on the sizes of the pumps required to be installed.

(c) Control Room. The pumps are proposed to be vertical spindle type. The electric motors will be placed inside a control room which will be constructed above the dry well. This will facilitate the chances of damage due to the accidental flooding of dry well and also the maintenance of the electric motor. The size of the control room will be the same as the size of the dry well.

Pumping Units and Pressure Sewer

Vertical spindle, non-clog centrifugal pumps coupled with vertical shaft electric motors will be provided.

In order to keep the flow through the treatment plant constant, the recirculated flow will be pumped at a varying rate in the hours of minimum and maximum flow. This will effect the ratio of recirculation but as the period of such minimum and maximum flow will be very small, there will be no considerable effect on the treatment plant and therefore on the quality of the effluent. The arrangement of pumps will be so made that these in different combinations will provide the required recirculated flow. Adequate capacity of the pump will be provided as standby. A diesel engine set will also be provided to work in emergency during electric failures.

Automatic float control with butterfly valve will be provided for the pump operation so that the pumps are switched on or off according to a pre-determined level in the wet well.

The design will be made for the estimated designed population but initially, only the capacity required for the present population will be installed. The space will be left for the installation of the additional pump units in the pump room as and when needed. In the case of pump house dry well and wet well, however, the reduction in the capacity will not be made, as the cost will not reduce in the same ratio as that of capacity and also because of practical difficulties.

Pressure sewer will be installed for carrying the recirculated flow from the pumping station to grit channel sump. Cast iron pipe will be used because the pressure sewer may be subject to sudden high pressure due to water hammer and also because the protection and maintenance for cast iron pipe sewer will be cheaper and easier. Centrifugal cast iron pipe, with a working pressure of 450 psi will be used.

Design of Pumping Station

To keep the flow in the treatment plant constant, the required recirculated sewage flow to the treatment plant will be as under:

Present Conditions with R = 8

	Raw sewage flow in l.p.m.	Proposed flow to the treatment plant in l.p.m.	Required recirculated flow l.p.m.	g.p.m.
Average	790	7100	6320	1664
Minimum	395	7100	6715	1765
Maximum	1980	7100	5130	1350

Future Conditions with R = 8.

Average	3910	15,640	11,730	3,100
Minimum	1950	15,640	13,685	3,600
Maximum	9760	15,640	5,880	1,550

Capacity of the Pumps

The above requirements show that there is a great variation in the quantity of recirculated flow and it will require installation of several sizes of pumps which will have the following disadvantages:

(i) Greater part of the flow will be dealt with low efficiency pumps in small sizes;

(ii) When one pump is out of commission, there is no standby of equal capacity;

(iii) as there are many sizes of pumps, larger quantities of spare components will have to be kept and these will not be inter-changeable.

Moreover, since the flow will be varying throughout the day, and these recirculated flows which have been determined on the basis of assumption for maximum flow to be 2.5 times and minimum flow to be half of average will remain for a very short time of the day, it will not be very much sound to follow these figures strictly. The sizes of the pumps will therefore be so made that maximum variation is possible with minimum number of pumps.

It is, therefore, proposed to install the following set of pumps for the present:

Three pumps of 700 gpm : P_1

One pump of 1650 gpm : P_2

For the average conditions one pump P_2 will be operated so as to give a discharge of 1650 gpm and the three pumps of 700 gpm will act as standby. For the peak hours of raw sewage flow, the two pumps P_1 (of 700 gpm) will be operated in parallel so as to give a discharge of 1350 gpm. For the minimum flow condition either the pump P_2 will be operated alone or one more pump of 750 gpm will be operated in paralle.

A diesel engine shall also be provided for the capacity of 1650 gpm for emergency during power failure.

For the future when the population reaches to the full designed figure of 75,000 persons, three more pumps of 1650 gpm will be added so as to make the total installation as under:

3 P_1 pumps of 750 gpm.

4 P_2 pumps of 1650 gpm.

During the hours of average raw sewage flow, the two pumps P_2 (1650 gpm) will be operated in parallel so as to give a discharge of 2850 gpm. If this is not sufficient one of the pumps of 750 gpm will be added which at the operating head of 25 ft. (against 3100 gpm) will give 250 gpm so as to make the total of 3100 gpm. For the hours of minimum raw sewage flow, when maximum recirculated flow will be required three pumps P_2

will be operated in parallel so as to give the required flow of 3600 gpm. For the peak raw sewage flow hours, the two pumps P_1 (750 gpm) in parallel or one pump of 1650 gpm will be operated as needed. One of the pumps P_2 will always remain as stand-by.

A diesel engine set shall also be added so as to make the capacity equal to the average flow required.

<u>Size of Pressure Sewer</u>

The static head for the pumping will be as follows:

Water level in the wet well ... 805.0

Maximum water level in the grit channel... 810.68

Therefore Static Head = 810.68 - 805

= 5.68 meters (or 18 ft)

Length of pressure sewer = 184 meters (604 ft).

If the size of pressure sewer is adopted as 18 inches, the velocity at maximum flow (13700 litres) is 4.5 f.p.s. which is O.K. but the velocity at minimum flow is 1.7 f.p.s. which is low. Since the recirculated flow will be drawn from the bottom of the final sedimentation tank it will have sludge mixed with it and therefore the minimum velocity should be at least 3 f.p.s.

A 14" size of pipe gives the velocities as under:

Velocity at max: flow ... 7.5 f.p.s.

Velocity at min: flow ... 3.0 f.p.s.

This size is suitable, as a lesser size will increase the velocity at maximum flow to be too high which will increase the head loss due to water hammer.

$$\text{Water hammer} = \frac{VoC}{g}$$

where Vo = initial velocity

C = speed of pressure wave

$$C = \frac{4720}{\sqrt{1 + \frac{KD}{Et}}}$$

where K = Bulk Modulus = 3×10^5

E = Modulus of Elasticity = 15×10^6

t = thickness of pipe = 0.75 inches

D = Diameter of the sewer = 14 inches

and g = acceleration due to gravity

$$C = \frac{4720}{\sqrt{1 + \frac{3 \times 10^5 \times 14}{15 \times 10^6 \times 0.75}}}$$

$$= 4030$$

$$h = \frac{VoC}{g} = \frac{7.5 \times 4030}{32.2} = 940 \text{ ft} = 410 \text{ psi}$$

<u>Frictional Losses in the Rising Main:</u>

The frictional losses will be calculated for maximum flow, i.e. 13,600 lpm. The losses due to the fittings will be obtained in equivalent length of straight pipe by Nomogram[1]

1. Seelye Elwyn E. "Data Book for Civil Engineers" John Wiley & Sons, New York, Second Printing, 1953, page 6-67.

and added to the straight length of rising main.

(i)	Length of rising main	... = 184 meters
(ii)	Entrance losses = 38 ft. ...	... = 9 meters
(iii)	Losses due to 4 - 45° Bend = 4 x 22 = 88ft.	= 27 meters
(iv)	One gate valve and check valve = 2 x 40 = 80ft.	= 25 meters
(v)	Outlet losses from 14" dia pipe to grit channel sump = 10ft. ...	... = 3 meters
(vi)	Add for loss inside the pumping station	... = 2 meters
	Total	... = 256 meters

Loss of Head for maximum flow from Nomogram = 0.6%

h_f (max) = 256 x 0.60 = 1.54 meter or = 5 ft.

Total head which the pressure pipe line has to sustain

= 940 + 18 + 5 = 963 ft. = 420 psi

It is within allowable pressure of 450 psi and is therefore O.K.

Capacity Head Curve of Pressure Sewer:

The capacity head curve for 14" C.I. pressure sewer in a length of 604 ft. with valve of n = 100 is plotted for various discharge as under:

Discharge in gpm	hf in ft/1000	Total hf
500	0.5	0.32
1000	2.1	1.27

1500	4.0	2.42
2000	6.5	3.92
2500	9.5	5.75
3000	13.0	7.85
3500	17.0	10.25
4000	22.0	13.3

Characteristics of Pumps

For larger pump of 1650 gpm, the performance curves of Peerless pumps have been plotted on page 131 which shows the following details.

Curve No. 4305-4, size 6 x 8 x 4M & 8 x 8 x 4M

Impeller diameter = 14 1/2 inches, efficiency = 70%

RPM = 730, sphere 4 inches,

Discharge 1650 gpm at 21 ft. head, HP = 15

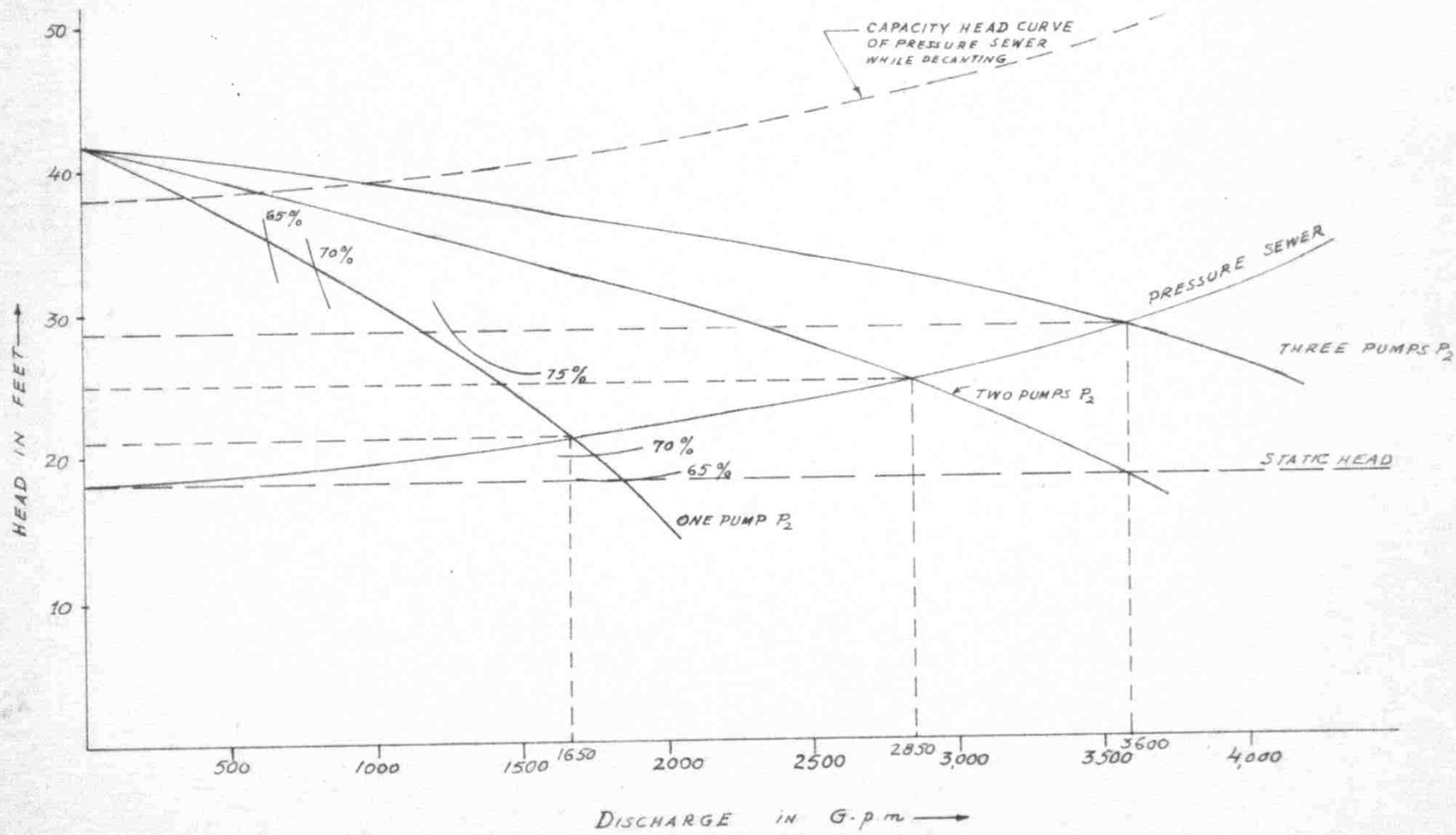
This pump or its equivalent by some other manufacturer will be used

$$N_s = \frac{N\sqrt{Q}}{H^{0.75}} = \frac{730\sqrt{1650}}{21^{0.75}}$$

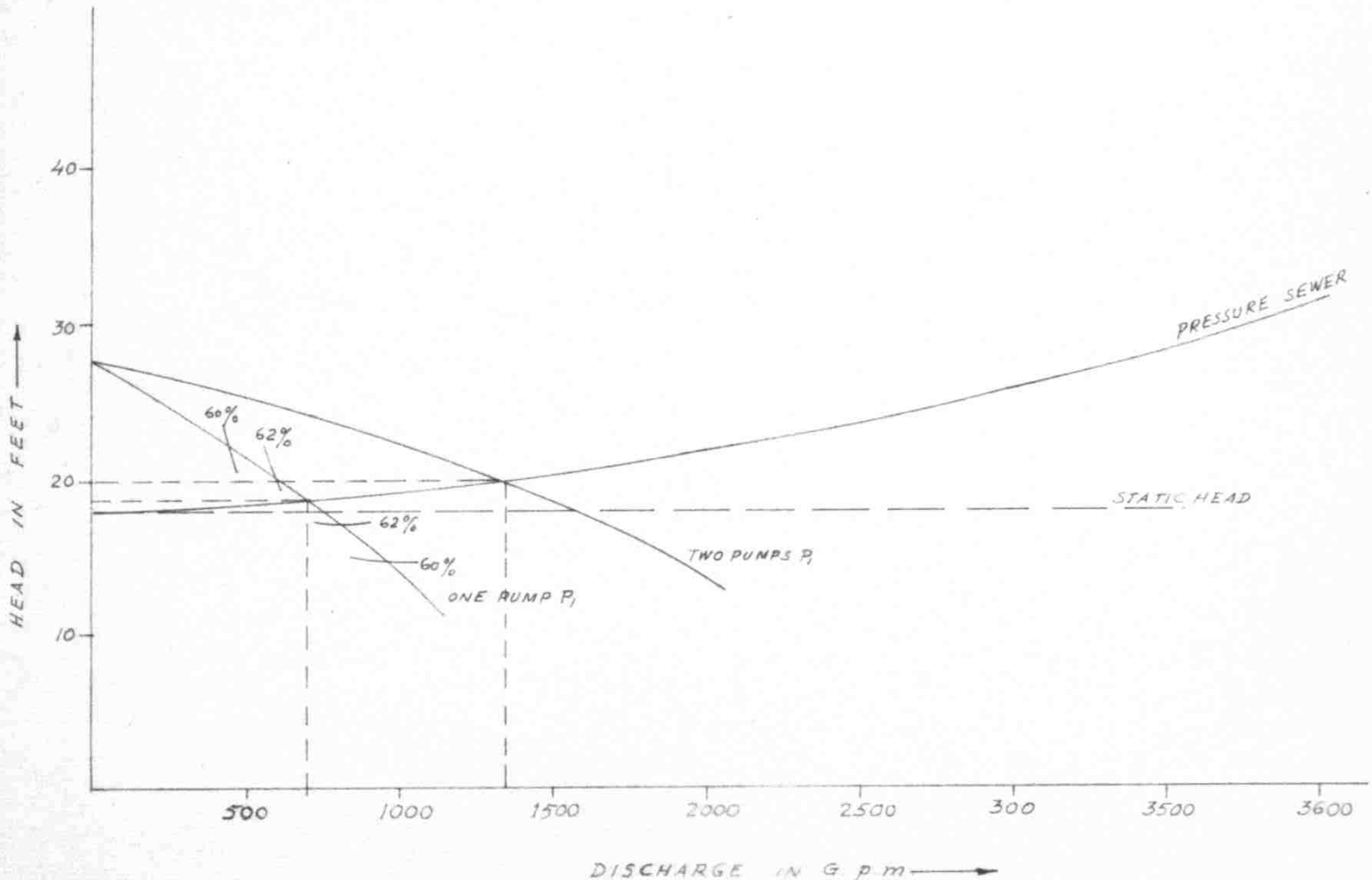
$$= 3,000 \text{ r.p.m.}$$

The pump will be radial flow.

While decanting the final sedimentation tank, the minimum water level in the wet well will be 899.0. The satic head will therefore rise to 11.68 meters (or 38ft). A capacity head curve of pressure sewer against this static head has been shown as dotted in the performance curve of P_2 pump at page 131.



CHARACTERISTIC CURVES OF P_1 PUMPS AND
CAPACITY HEAD CURVE OF PRESSURE SEWER



CHARACTERISTIC CURVES OF P_2 PUMPS AND CAPACITY HEAD CURVES OF PRESSURE SEWER

It shows that the three pumps working in parallel at a head of about 40ft will give a discharge of 1000 gpm which will be alright, as the decantation of final sedimentation tank will not be very often but once in three months or so.

For smaller pump of 700 gpm, the performance curve of Peerless Pumps have been plotted on page 132 which shows the following details:

Curve No. V 4302, size 5 x 6 x 3 - 1/8L

Impeller diameter	= 12", RPM - 730
Efficiency	= 62%
Sphere	= 4 inches
Discharge	= 700 gpm at 19 ft. head
Horse Power	= 5

This pump or its equivalent by some other manufacturer will be used.

$$N_s = \frac{730\sqrt{700}}{(19)^{0.75}} = 2000 \text{ r.p.m.}$$

The pump will be radial flow.

Two Pumps in Parallel

The performance curve for two P_1 pumps (700 gpm) and three P_2 pumps (1650 gpm) have been plotted on pages 131 & 132. This shows that when two P_1 pumps (700 gpm) will be operated in parallel, it will give a discharge of 1350 gpm and when two P_2 pumps (1650 gpm) will be operated, it will give a discharge of 2850 gpm. The three P_2 pumps will give a total discharge of 3600 gpm at a head of 48 ft.

HYDRAULICS OF THE TREATMENT PLANT

The flow to the treatment plant is as follows:

$$\text{Present } Q_{av} = \frac{38,000 \times 30}{24 \times 60} = 790 \text{ lpm}$$

$$\text{Future } Q_{av} = \frac{75000 \times 75}{24 \times 60} = 3910 \text{ lpm}$$

Present Minimum flow = 15% of average = 120 lpm

Present Maximum flow = 2.5 Av = 2.5x790 = 1980 lpm

Future minimum flow = 15% x 3910 = 560 lpm

Future maximum flow = 12,206 lpm

There is a wide variation between minimum and maximum raw flow. Since these flows will be for a short time, these will be taken care of by increasing or decreasing the recirculation by providing several pumps in the recirculation pumping station and thus keeping the flow constant through the units.

The design will thus be based on present and future average rates of flow.

Present recirculation = 8

Future recirculation = 3

Present total discharge = 786 x 9 = 7074 or 7100 l.p.m.

Future total discharge = 3910 x 4 = 15640 l.p.m.

1. From Grit Chamber to Sed: Tank Distribution Tower

Q max = 15,650 l.p.m.

Q min = 7,100 l.p.m.

Using Cast Iron pipes with C - 100

Using Nomogran based on Hazen-Williams Formulae

For max Q, size of pipe	=	16 inches
Velocity	=	6.6 ft./sec.
Loss of Head	=	1.25%
Slope	=	0.0125
For min: flow, size of pipe	=	16 inches
Velocity	=	3 ft./sec. - O.K.
Loss of Head	=	0.3%

(i) Length of Pipe .... = 2 = 25.0 mt.

(ii) Loss of head at entrance in equivalent,

Length of straight pipe from Nomogram[1]

= 22 ft. = 6.7 meters

(iii) Outlet loss in equivalent length of straight pipe

loss due to enlargement from 16" pipe to the dia. of distribution box assuming the ratio of pipe dia. to diameter of distribution box as 1/4)

in equivalent length of straight pipe

= 40 ft. = 12.2 meters

Total length 44.4 meters

Loss of head:

h_f (max) = 44 x 125% = 55cm. or 0.55 meter

1. Seelye E.E. "Data Book for Civil Engineers - Design", Vol. I, John Wiley & Sons, Inc., New York, Second Edition 1953, p.6.-67.

h_f (min) = 44.4 x 0.3% = 13cm. or 0.13 meters

2. <u>From Sed: Tank Tower to Centre of Sed: Tank</u>

Q max in each pipe = $\frac{15650}{4}$ = 3910 l.p.m.

Q min in each pipe = $\frac{7100}{4}$ = 1780 l.p.m.

The entering velocity into the sedimentation tank should be low so as to prevent pronounced currents towards the outlet and to distribute the sewage uniformly over the cross section of the tank. So from Nomogram, for 16 inches dia pipe.

Max:Velocity (for Q max)	=	1.7 ft/sec.
Loss of Head	=	0.1%
Min:Velocity (for Q min)	=	0.8 ft/sec.
Loss of Head	=	0.03%
Slope	=	0.001

(i) Length of Pipe .... = 15 meters.

(ii) Loss of head at entrance (in equivalent length of pipe) for 16" dia pipe = 22ft.=6.7 meters

(iii) Loss of head in 90° Bend (for medium sweep elbow from Nomogram) = 18 ft. = 5.8 meters

Total length 27.5 meters

h_f (max) = 27.5 x 0.1% = 2.75cm. or 0.03 meters

h_f (min) = 24.5 x 0.03% = 0.75cm. or 0.01 meters.

3. <u>From Sedimentation Tank Tower to Trickling Filter Tower</u>:

Q max = 15,650 l.p.m.

Q min = 7,100 l.p.m.

From Nomogram, for 16 inches diameter pipe

Max: Velocity (for Q max) = 6.6 ft./sec.

Loss of Head = 1.2%

Min: Velocity (for Q min) = 3ft/sec. - O.K.

Loss of head = 0.3%

Slope = 0.012

(i) Length of Pipe .... = 56 meters

(ii) Loss of head at entrance for 16" pipe =22ft.= 6.7 meters

(iii) Loss of head at outlet (as in 1 above) =40ft.= 12.2 meters

Total 74.9

SAY <u>75 meters</u>

h_f (max) = 75 x 1.2 = 90cms or 0.9 meter

h_f (min) = 75 x 0.3 = 22.5 cms. or 0.23 meter

4. <u>From Filter Tower to Centre of Filter at Gravel Level</u>:

Q max in each pipe = $\frac{15650}{4}$ = 3910 l.p.m.

Q min in each pipe = $\frac{7100}{4}$ = 1780 l.p.m.

From Nomogram, for 10 inches diameter pipe.

Max: Velocity (for Q max) = 4.2 ft/sec.

Loss of head = 1.0%

Min: Velocity (for Q min)	=	2 ft/sec. - O.K.
Loss of head	=	0.7%
Slope	=	0.25%

(i) Length of pipe (including 6 ft. depth of filter) = 25 meter

(ii) Loss of head at entrance = 15 ft. = 4.6 meter

(iii) Loss of head for one 90° Bend = 18 ft. = 5.8 meter

Total = 35.4 meters

h_f (max) = 35.4 x 1.0% = 35.4 cm.

h_f (min) = 35.4 x 0.25% = 9 cm.

Add to this, loss of head in nozzle by assuming,

(i) height of nozzle above gravel = 15 cm.

(ii) loss in nozzles = 75 cm.

90 cm.

Therefore h_f (max) = 35.4 + 90 = 125.4 cms. or 1.26 meter

h_f (min) = 9.0 + 90 = 99 cms. or 1.00 meter.

From Centre of Trickling Filter to Tower of F.S. Tank:

(i) Length of channel = = 70 meters

Assuming 1/2% slope in the length of channel

loss of head = 35cm. = 35cm.

Add for depth of channel = 20cm.

Add for thickness of slab = 10cm.

Total = 65cm. or = 0.65 meters

5. From Final Sed: Tank Tower to Centre of Final Sed: Tank:

Max. Flow/tank = $\frac{15650}{2}$ = 7825 l.p.m.

Min. Flow/tank $= \frac{7100}{2} =$ 3550 l.p.m.

Adopt size 16 inches, from Nomogram

Max: Velocity (for Q max) = 3.6 ft/sec.

Loss of head = 0.38%

Min: Velocity (for Q min) = 1.5 ft/sec. - O.K.

Loss of head = 0.1%

Slope = 0.0038

(i)	Length of Pipe		= 15 meter
(ii)	Loss of head at Entrance	= 22ft.	= 6.7 meter
(iii)	Loss of head due to 90° Bend	= 25ft.	= 7.5 meter
		Total	= 29.2 meter

h_f (max) = 29.2 x 0.38% = 10.8cm. or 0.11 meter

h_f (min) = 29.2 x 0.1% = 2.92cm. or 0.03 meter

From Centre of Final Sed: Tank to Pumping Station:

Max Flow/tank going to recirculation Pumping Station

= 3/4 x 7825 = 5880 l.p.m.

Min Flow/tank going to recirculation Pumping Station

= 8/9 x 3550 - 3160 l.p.m.

Adopt 12" size pipe, from Nomogram

Max:Velocity (for Q max) = 4 s.ft/sec.

Loss of head = 0.9%

Min:Velocity (for Q min) = 2.4 ft/sec.

Loss of head = 0.3%

Length of pipe .... = 20 meters

Loss of head at entrance = 17ft. = 5.2 meters

One 90^o Bend = 20ft. = 6.1 meters

Total 31.3 meters

h_f (max) = 31.3 x 0.9% = 28.2cm. or 0.3 meter

h_f (min) = 31.3 x 0.3% = 9.39cm. or 0.1 meter

From Sed: Tank to Sludge Digestion Tank Control Room:

Volume of sludge (from page 116)

= 3500 cuft. daily

The sludge will be drawn every 6 hours

Therefore volume $= \frac{3500}{4 x 35.4} = 24.7\ m^3$

= 24700 litres.

If this will be drawn in 15 mts.

Volume per minute $= \frac{24700}{15} = 1646$ l.p.m.

Adopt size of Pipe 8 inches

Velocity = 2.8 ft./sec.

h_f = 0.6%

Loss of head at entrance = 15ft. = 4.5 meter

Loss of head for two 90^o Bend = 2 x 12 = 24ft.= 7.5 meter

Length of Pipe = 309 = 94.0 meter

Total 106.0 meters

h_f (max) = 106 x 0.6% = 63.5 cm. or 0.64 meter

Therefore, the level in the wet well for sludge digestion tank = 810.3 - 0.64 = 809.66

Level in the Treatment Plant Units:

Min: Level in the wet well = 805.2

Max: Level in the wet well = 805.4

W. Level at Final Sed Tank Centre = 805.5

Max: Water level at Final Sed tank tower = 805.5 + 0.11
= 805.61 meter

Min: Water level at Final Sed Tank Tower = 805.5 + 0.03
= 805.53 meter.

2. Max: Water level in the T. Filter Gr. level:

(i) Depth of gravel = 6 ft. = 6 x 30.5 = 183 cm.

(ii) Add losses from centre of T. Filter to distribution tower of F.S. Tank = 65cm.

Total = 248cm.
or 2.48 meter

Max: Water level in the T. Filter gravel 805.61 + 2.48
808.09 or 808.10mt.

3. Max: Water level in the Distribution Tower of Trickling filter = 808.10 + 1.26 = 809.36

4. Min: Water level in the distribution Tower of Trickling Filter
= 808.10 + 1.0 = 809.10 meters

5. Max: Outgoing WL in the Distribution Tower of Pr. S. Tank
= 809.36 + 0.90 = 810.26 meters

Min:Outgoing WL in Distribution Tower of Pr. Sed. Tank

= 809.10 = 0.23 = 809.33 meters

6. Adopt Water Level in the P.S. Tank

= 810.30 meters

7. Max: Incoming WL in the distribution tower of Pr. Sed. Tank

= 810.30 + 0.03 = 810.33 meters

8. Min: Incoming water level in the distribution tower of Pr. Sed Tank = 810.30 + 0.01 = 810.31 meters

9. Max: WL in the grit chamber

= 810.33 + 0.55 = 810.88 meters

Min: water level in the grit chamber

= 810.31 + 0.12 = 810.43 meters.

F. <u>SLUDGE DISPOSAL</u>:

Sludge disposal is one of the main problem involved in sewage treatment, because the sludge which accumulates has to be removed from the site of treatment works. It can be disposed of to fill land or can be used as a fertilizer filler but before it is done so, it must be dried to a suitable degree of moisture. The moisture is removed from sludge of decrease its volume and its characteristics. The sludge containing 75% moisture can thus be moved with a shovel or fork.

Methods for drying sludge include the use of sand beds, presses, vacuum filters, centrifuges and heat dryers. The other less expensive methods such as disposal in permanent lagoons, drying on earth plots, pumping in wet form to farmers, are liable to cause nuisance. The approximate relative cost of the various means of sludge disposal in terms of percentage of the cost of drying on beds is given as under:[1]

Method of Disposal	Cost as a Percentage of Drying on Beds
Disposal in permanent lagoons	25
Drying on earth plots	50
Pumping to farm lands	50
Drying on sludge beds	100
Filter pressing	150
Heat drying of press cake	300
Vacuum filtration and flash drying	670

The method of drying by presses, vacuum filters, centrifuges and heat driers besides being more costly requires extensive mechanical equipment and will thus not be suitable for this treatment plant. The sludge drying beds which is simplest of these practices will be most suitable for this plant and will therefore be adopted.

Drying Bed Area:

The required area of sludge drying beds depends on the nature of the sludge to be dewatered and the climatic conditions. A well digested sludge is dewatered more readily than partly digested sludge.

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1. I.B. Escritt, Sewerage and Sewage Disposal, opcit., p. 320.

According to the upper Mississipi and Great Lakes Boards of State Sanitary engineers, the area required for sludge drying bed for High rate filter type of treatment is 1.5 squire feet per capita.[1] Some of the State Departments of Health in United States for example, Illinois, Iowa, Idaho and Missouri adopt a rate of 1 s.ft per capita.[2] For Hebron, a rate of 1 s.ft. per capita will be quite sufficient, because the drying time will be much shorter due to more sunshine, less rainfall and low relative humidity and will therefore be adopted.

Design Criteria:

The following design criteria will be adopted for the design of sludge drying beds:

1. The embankment will be of concrete walls and will extend 30 cm. above the sand surface.

2. The earth floor of the drying bed will be graded slightly to the underdrain for collecting the efficient.

3. The underdrains will be of 6" dia. drain tile placed in trenches with open joints. The main underdrain will be laid down diagonally over the drying bed, and the laterals which will be of 4" diameter unglazed tile will meet

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1. Steel, E.W., Water Supply & Sewerage, opcit., p. 589 (Table 26-2).

2. Babitt, H.E., Sewerage & Sewage Treatment, opcit. p.538 (Table 91).

at about 30° to main underdrain as shown in the drawing No. 19. The laterals will be placed at 3 meters centre to centre.

4. The bottom will consist of graded gravel in three layers. Around the underdrains, a 5 in layer of 1 1/2 inch to 2 inch aggregate will be placed. It will be topped with 3 inch layer of 1 inch aggregate. The next layer will be of 1/2 inch aggregate 4 inch deep. The total depth from the underdrain tile to the top of the coarse layer will thus be 12 inches.

5. <u>Sand</u>: The sand depth will be 12 inches and will be of good quality to meet the following specifications:

(i) Washed (less than 1% dirt by volume).

(ii) Uniformity co-efficient not over 4.0 and preferably under 3.5, and

(iii) Effective size of grains between 0.3 & 0.75 mm.

6. To provide an economical and satisfactory control of sludge flow to the beds, shear gates will be provided.

7. The sludge will be distributed through pipe lines which will be laid on a grade to give a velocity of at least 2.5 f.p.s.

8. Splash plates of concrete will be provided to distribute the flow of wet sludge uniformly over the bed without causing sand scouring.

9. Multiple beds will be provided to provide operating facility. The size of the beds will be 18 x 8 meters.

10. Facilities for cake removal: The quantity of sludge to be handled is quite appreciable, and it will therefore not be efficiently possible to remove the sludge by wheel borrows. The facility to expedite cake removal will be provided and will consist of dump cars running on narrow gauge (about 24 inches). To avoid power driven equipment, the cars will be moved by hand or by animal for transporting the sludge cake to a point of disposal.

Design of Sludge Drying Beds:

Total ultimate population ... ... 75,000 persons

present population ... ... 38,000 persons

At the rate of 1 s.ft. per capita per day,

total drying bed area required = 75,000 s.ft.

or 7,000 sq. meters

Adopting a size of 18 x 8 meters,

$$\text{number of drying beds required} = \frac{7,000}{18 \times 8}$$

$$= 49 \text{ beds}$$

50 Beds will therefore be provided.

Drying Bed area required for the present = 38,000 s.ft

or 3550 sq. meters

$$\text{Number of drying Beds required} = \frac{3550}{18 \times 8} = 25 \text{ Nos.}$$

G. SEWAGE FLOW MEASUREMENT DEVICE:

In the maintenance of sewer systems and efficient operation of the sewage treatment plant, it is necessary to secure information on the rate of flow of sewage. Since a partial flume has already been provided for velocity control, it will be used for the measurement of rate of flow. The measurement will be made in open channel flow by translating accurately the difference of elevation of sewage flow into the rate of flow. A small well will be constructed at a distance 2/3rd length 'A' from the flume throat and will be provided with a float. The float will be connected to a device to convert the depth of sewage in the well directly into the rates of flow.

H. CHLORINATION

The effluent from the treatment plant will be chlorinated for disinfection before disposing off to the fields for crop irrigation. To be effective, the residual chlorine will ordinarily be not less than 0.2 to 1.0 mg/litre and will require a contact time of not less than 14 minutes.[1] Under these conditions, chlorination of effluent from secondary treatment will result in 99.9% reduction in the coliform content of the effluent. According to the Ten-State Standard, the dose for normal domestic sewage treated in a trickling filter plant is 15 milligrams per litre.[2]

1. WPCF Manual on "Sewage Treatment Plant Design" opcit. p. 283.
2. Ibid. p. 283

Using this rate the total chlorine required per day will be as follows:

$$= \frac{75{,}000 \times 75 \times 15}{1000 \times 454} = 186 \text{ lbs.}$$

The chlorine will be obtained in manufacturers 150 lbs. cylinders, or other suitable size governed by the availability of chlorine locally and the policy of the supplier. The liquid chlorine containers will be handled with utmost care as the free chlorine combines with any moisture and becomes toxic. The chlorine feeding equipment will consist of vacuum feed type of chlorinator which incorporates all safety features required to prevent the accidental discharge of chlorine gas. The equipment will be installed in a room for maintenance and handling of chlorine containers. Only one unit will be installed for the present and the other unit will be added when the population reaches the designed figure. A room having an area of 15 sq. meters will be provided.

I. LABORATORY, OFFICE, STORE & WORKSHOP

The performance and efficiency of a sewage treatment plant are dependent to a large degree upon supervision and efficient maintenance. The following service building will therefore be constructed at the treatment plant. These will be arranged and designed with primary consideration to the activities of operating personnel.

Laboratory Cum Office Building

It will consist of two portions:

(a) the laboratory portion.

(b) the office portion.

The laboratory will be concerned primarily for carrying out the following tests on a routine basis:

(i) B.O.D.

(ii) Dissolved Oxygen

(iii) Suspended Solids

(iv) Residual Chlorine

(v) PH, Temperature, and Odour.

(vi) Coliform Test.

It will comprise of a chemical laboratory room of about 5 x 8 meters for conducting all laboratory operations for the above test and for desks, files, and a chemist office of about 3 x 4 meters size. The space for office will be adequate to provide the plant superintendent the required privacy needed for proper supervision of personnel and for filing of plant records and correspondence. It will comprise of one room for general office and record of about 4 x 5 meters, and a superintendents room of about 3 x 4 meters. A small reception room will also be provided in view of increasing public interest.

Store

A store room is important for keeping stock of spare parts, materials and operating supplies. In addition it will also take care of plant portable equipment which is bulky and vulnerable to damage if not properly stored. The room will be designed to allow trucks to back up to a loading dock and will be equipped with an overhead door. It will be provided with steel shelves and bins.

Workshop:

A workshop is very essential for the maintenance of mechanical equipment particularly in a location remote from commercial machine shop like Hebron. It will consist of two portions:

(a) General repair shop and

(b) Machine shop

and will be situated in the same building in adjoining rooms. Two separate rooms will be provided in view of the fact that the precision tools used in the machine shop would be vulnerable to damage in a general repair shop. The repair shop will be equipped with facilities to store small tools and a desk and chair for daily log and report work.

A suitable size of store and workshop would best be decided after consultation with the plant superintendent and the mechanical foreman. Sufficient space to house those has been provided as shown in the layout plant drawing (Drawing No. 12).

CHAPTER V

COST ESTIMATE

To have an idea of the cost of the project, a cost estimate is needed. The preparation of detailed specifications and cost estimate is beyond the scope of the present study. Also the actual rates for various items of work can only be determined accurately by competitive tenders or by inviting rates from different dealers. It is, therefore proposed here to prepare an approximate quantities of work done and cost involved to have an approximate idea of the cost of the project for allocation of funds and inviting of tenders. The estimates of quantities and cost will be as under:

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
1	(a)Supplying and consturcting 18" dia. pipe sewer at invert depth upto 1.5 meter complete job.	1220	Meters	4.20	5,120
	(b)Supplying & constructing 18" dia. pipe sewer at an invert depth upto 3 meter complete job.	370	Meters	5.00	1,850
2	Supplying & constructing 16" dia. pipe sewer at invert depth upto 1.5 inch complete job.	1425	Meters	3.80	5,400

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
3	Supplying & constructing 14" dia pipe sewer at an invert depth upto 1.5 meter, complete job.	785	Meter	3.50	2,750
4	Supplying & constructing 12" dia pipe sewer at an invert depth upto 1.5 meter complete job.	142	Meter	3.0	430
5	(a) Supplying & constructing 10" dia pipe sewer at an invert depth upto 1.5 meter, complete job.	621	Meter	2.50	1,550
	(b) Supplying & constructing 10" dia pipe sewer at an invert depth upto 3 meters, complete job.	77	Meter	2.70	240
	(c) Supplying & constructing 10" dia pipe sewer at an invert depth above 3 meters, complete job.	375	Meter	3.00	1,130
6	(a) Supplying & Constructing 8" dia pipe sewer at an invert depth of 1.5 meters complete job (for main & trunk sewers	7845	Meter	2.20	17,250
	(b) Supplying & constructing 8" dia pipe sewer upto an invert depth of 3 meters complete job.	973	Meter	2.50	2,430
	(c) Supplying & constructing 8" dia pipe sewer upto an invert depth of 1.5 meters for laterals (qty taken as approximate) complete job.	14000	Meter	2.20	30,800

Item No. No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
7	Constructing manholes as per drawing complete job:				
	(a) Over 18" pipe sewer upto 1.5 meter depth.	5	Nos.	50.0	250
	(b) Over 18" pipe sewer upto 3.00 meter depth	17	Nos.	70.0	1190
	(c) Over 16" pipe sewer upto 1.5 meter depth	20	Nos.	45.0	900
	(d) Over 14" pipe sewer upto 1.5 meter depth	11	Nos.	40.0	495
	(e) Over 12" pipe sewer upto 1.5 meter depth	3	Nos.	35.0	105
	(f) Over 10" pipe sewer upto 1.5 meter depth	11	Nos.	40.0	440
	(g) Over 10" pipe sewer upto 3.0 meter depth	1	Nos.	50.0	50
	(h) Over 10" pipe sewer above 3.0 meter depth	4	Nos.	60.0	240
	(i) Over 8" pipe sewer above 1.5 meter depth	122	Nos.	30.0	3660
	(j) Over 8" pipe sewer upto 3 meter depth	16	Nos.	40.0	460
	(k) Over 8" dia lateral sewers	300	Nos.	30.0	9000
					86740
				say	87000 J.D.

STORM WATER SEWER

Item No.	Description	Estimated qquantities	Unit	RRate	Cost in J.D.
1	Supplying and constructing 36" dia pipe sewer at invert depth upto 3 meters depth complete job.	550	Meter	7.50	4,125
2	Supplying and constructing 42" dia pipe sewer at invert depth upto 3 meter, complete job.	550	Meter	8.50	4,675
3	(a) Supplying and constructing 48" dia pipe sewer at invert depth upto 3 meter complete job.	500	Meter	9.25	4,625
	(b) Supplying and constructing 48" dia pipe sewer at invert depth above 3 meter depth complete job.	350	Meter	10.0	3,500
4	Supplying and constructing 54" dia pipe sewer at invert depth upto 3.5 meter depth complete job.	470	Meter	12.0	5,640
5	Supplying and constructing 60" dia pipe sewer at invert depth upto 3.5 meter depth complete job.	480	Meter	13.50	6,480
6	Constructing manholes as per drawing complete job:				
	(a) Over 36" pipe sewer upto 3 meter depth	8	Each	80.0	640

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
	(b) Over 42" pipe sewer upto 3 meter depth.	8	Each	80.0	640
	(c) Over 48" pipe sewer upto 3.5 meter depth.	8	Each	85.0	680
	(d) Over 48" pipe sewer above 3.5 meter depth.	6	Each	90.0	540
	(e) Over 54" pipe sewer upto 3.5 meter depth.	6	Each	90.0	540
	(f) Over 60" pipe sewer upto 3.5 meter depth.	16	Each	90.0	1440
					30,525
				say	31,000 J.D.

TREATMENT PLANT (1st Phase)

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
1	Constructing Grit channel as per drawing including supplying and fixing bar racks constructing partial flame supplying measuring device, all accessories, complete job.	1	Each	750	750
2	Construction of Primary Sedimentation Tanks as per drawing including the mechanical scraping equipment required, all piping and accessories complete job.	2	Each	6000	12,000
3	Construction of Trickling Filter including supplying the rotary distributor, filter media, and other items as per drawing, complete job.	3	Each	10000	20,000

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
4	Construction of Final Sedimentation Tank as per drawing complete job.	1	Each	4,000	4,000
5	Construction of Sludge Digestion Tank:				
	(a) with fixed concrete top including supplying the screw pump, and other items as per drawing, complete job.	2	Each	8,000	16,000
	(b) with open top, complete job.	1	Each	4,000	4,000
6	Construction of Pumping station including cost of pumps and motors, dry well, wet well and control room, rising main, complete job.	1	Each	20,000	20,000
7	Construction of Sludge drying beds including all piping work etc., complete job.	25	Each	800	20,000
8	Construction of distribution tower for Primary Sedimentation Tanks, complete job.	1	Each	2,000	2,000
9	Construction of Control Building for Sludge Digestion tank, complete job.	1	Each	5,000	5,000
10	Construction of Distributor tower for trickling filter, complete job.	1	Each	15,00	1,500
11	Construction of Laboratory and Office	1	Each	2,500	2,500

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
12	Construction of Store	1	Each	2,500	2,500
13	Constructionof Workshop	1	Each	3,000	3,000
14	Construction of Chlorination Plant	1 1	Each	2,000	2,000
15	Construction of compound wall and gate	-	Job	5,000	5,000
16	Cost of land	30,000	sq.mtr	0/50	15,000
17	General expenses, development of land and all preparings etc.		LS		40,000
					1,75,250
				say	1,76,000 J.D.

II. TREATMENT PLANT (Second Phase)

1	Construction of Primary Sedimentation Tanks as per drawing including mechanical scraping equipment required, all piping and accessories complete job.	2	Each	6,000	12,000
2	Construction of Trickling Filter including supplying the rotary distributor filter media and other items as per drawing, complete job.	2	Each	10,000	20,000
3	Construction of Final Sedimentation Tank as per drawing, complete job.	1	Each	4,000	4,000
4	Construction of Sludge Digestion Tank (a) with fixed concrete top i/c supplying the screw pumps, and other items				

Item No.	Description	Estimated quantities	Unit	Rate	Cost in J.D.
	and accessories required, complete job.	2	Each	8000	16,000
	(b) Digestion Tank with open top complete job.	1			
5	Construction of Sludge Drying Beds including all piping work etc., complete job.	25	Each	800	20,000
6	General Expenses	L.S.	Each	800	10,000
	Total	...	...	...	86,000

ABSTRACT OF COST

I	Sanitary Sewer ...	JD.	87,000
II	Storm water Sewer ...	JD.	31,000
III	Treatment Plant, 1st Phase ...	JD.	176,000
IV	Treatment Plant, 2nd Phase ...	JD.	86,000
	Total ,...	JD.	380,000

equivalent to \$ 1,013,000 for a population of 75,000 persons.

Cost per capita ... $\frac{1,013,000}{75,000}$

= \$ 13.50

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FLOW IN MAIN SEWERS.

SHEET NO. 1

SEWER	AREA IN DUNUMS	POPULATION DENSITY PER DUNUM	POPULATION		DISCHARGE @ 75 l.p.c.p.d in l.p.m	
			INCREMENT	TOTAL	AVERAGE	MAXIMUM
A	81 13	35 20	2835 260	3095	161	644
B	15 62	65 20	975 1240	2,215	116	464
C	94 18.15	35 65	3290 1180	4,470	232	928
D	70 63	35 65	2250 4095	6,345	330	1320
E	47	35	1645	1645	85	340
F	33.8 55.0	65 35	2197 1925	4,122	215	860
G	55 49	35 12	1925 588	2,513	131	524
H	30.6 100.0	35 20	1071 2,000	3,071	160	640
I	476	12	5712	5,712	298	1192
J	52	12	624	624	33	132
K	109	12	1308	1308	68	272

ACCUMULATIVE FLOW IN TRUNK SEWER

SHEET NO: 2

MANHOLES		AREA DIRECTLY SERVED IN DUNUMS	POPULATION DENSITY PER DUNUM	POPULATION		DISCHARGE @ 75 l.p.c.p.d in l.p.m		Q_{MAX}: FROM MAIN SEWER	ACCUMULATIVE Q in l.p.m
FROM	TO			INCREMENT	TOTAL	AVERAGE	MAXIMUM		
T.99	T.81	500	12	6,000	6,000	313	780	–	780
T.81	T.80	–	–	–	–	–	–	272	1052
T.80	T.72	–	–	–	–	–	–	132	1184
T.72	T.64	56.2	20	1124					
		325.5	12	3906	5030	262	655	–	1839
T.64	T.63	11.6	35	406	406	22	55	1192	3,086
T.63	T.60	–	–	–	–	–	–	1164	4,250
T.60	T.56	28	35	980					
		32.5	65	2112					
		122	35	4270	7362	384	960	–	5,210
T.56	T.54	–	–	–	–	–	–	860	6,070
T.54	T.51	48	65	3120	3120	162	405	–	6,475
T.51	T.48	–	–	–	–	–	–	340	6,815
T.48	T.47	201.5	35	7052					
		48	65	3120	10,172	530	1330	–	8,145
T.47	T.46	–	–	–	–	–	–	1320	9,465
T.46	T.42	–	–	–	–	–	–	928	10,393
T.42	T.39	125	35	4375					
		83	20	1660	6035	314	785	–	11,178
T.39	T.1	–	–	–	–	–	–	1108	12,286

DESIGN OF MAIN SEWERS.

SHEET NO. 3

LINE	FROM MANHOLE		Q_{av} in (liters/mt)	Q_{max} in (litres/mt)	LENGTH in (METERS)	GROUND ELEVATIONS in METERS		DIA. of PIPE in INCHES	GRADE of SEWER	FALL of SEWER (METERS)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l/m	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
A	A.12	A.11	161	644	75	899.4	894.6	8	0.068	5.1	10.5	6,000	898.4	893.3
	A.11	A.10	"	"	75	894.6	889.5	"	"	"	"	"	893.3	888.2
	A.10	A.9	"	"	50	889.5	886.2	"	"	3.4	"	"	888.2	884.8
	A.9	A.8	"	"	75	886.2	882.1	"	0.052	3.9	8.7	5,000	884.8	880.9
	A.8	A.7	"	"	75	882.1	879.1	"	0.038	2.9	7.5	4,400	880.9	878.0
	A.7	A.6	"	"	75	879.1	877.0	"	0.028	2.1	6.4	3,700	878.0	875.9
	A.6	A.5	"	"	75	877.0	873.3	"	0.052	3.9	8.7	5,000	875.9	872.0
	A.5	A.4	"	"	75	873.3	867.3	"	0.080	6.0	11.5	6,700	872.0	866.0
	A.4	A.3	"	"	75	867.3	860.4	"	0.096	7.1	13.0	7,500	866.0	858.9
	A.3	A.2	"	"	65	860.4	854.3	"	"	6.2	"	7,500	858.9	852.7
	A.2	A.1	"	"	45	854.3	851.6	"	0.06	2.1	10.0	5,800	852.7	850.6
	A.1	T.39	"	"	25	851.6	851.3	"	0.012	0.3	4.0	2,400	850.6	850.3
B	B.6	B.5	116	464	62	863.3	861.5	8	0.028	1.74	6.4	3,800	862.2	860.46
	B.5	B.4	"	"	63	861.5	860.5	"	0.018	1.13	5.0	3,000	860.46	859.33
	B.4	B.3	"	"	62	860.5	858.7	"	0.03	1.86	6.6	3,800	859.33	857.57
	B.3	B.2	"	"	63	858.7	856.2	"	0.04	2.52	7.8	4,500	857.57	855.05
	B.2	B.1	"	"	67	856.2	853.0	"	0.052	3.48	9.0	5,200	855.05	851.57
	B.1	T.39	"	"	70	853.0	851.25	"	0.022	1.54	6.4	3,800	851.57	850.03
C	C.3	C.2	232	928	41	865.4	865.4	8	0.01	0.4	3.7	2,200	864.4	864.0
	C.2	C.1	"	"	75	865.4	861.5	"	0.048	3.7	8.7	5,000	864.0	860.3
	C.1	T.46	"	"	50	861.5	860.4	"	0.032	1.6	7.3	4,200	860.3	858.7

DESIGN OF MAIN SEWERS

SHEET NO: 4

LINE	FROM MANHOLE		Q_{av} in l.p.m	Q_{max} in l.p.m	LENGTH in METERS	GROUND ELEVATIONS IN METERS		DIA: OF PIPE IN INCHES	GRADE of SEWER	FALL of SEWER	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l.p.m	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
D	D.13	D.12	330	1320	50	927.1	924.7	8	0.064	3.2	10.3	6,000	926.1	922.9
	D.12	D.11	"		75	924.7	919.3	"	"	4.8	"	6,000	922.9	918.1
	D.11	D.10	"		75	919.3	914.2	"	0.074	5.55	11.0	6,500	918.1	912.55
	D.10	D.9	"		75	914.2	906.1	"	0.106	7.95	14.0	8,000	912.55	904.45
	D.9	D.8	"		75	906.1	899.0	"	0.108	8.1	14.0	8,000	904.45	896.35
	D.8	D.7	"		63	899.0	896.2	"	0.034	2.14	7.5	4,200	896.35	894.21
	D.7	D.6	"		75	896.2	886.7	"	0.138	10.35	15.5	9,000	894.21	883.86
	D.6	D.5	"		50	886.7	875.8	"	0.208	10.4	16.6	9,200	883.86	873.46
	D.5	D.4	"		50	875.8	870.4	"	0.088	4.4	13.0	7,400	873.46	869.66
	D.4	D.3	"		50	870.4	866.7	"	0.066	3.3	11.0	6,200	869.06	865.76
	D.3	D.2	"		100	866.7	864.2	"	0.025	2.5	6.4	3,800	865.76	863.26
	D.2	D.1	"		62	864.2	863.8	"	0.012	0.74	4.0	2,400	863.26	862.52
	D.1	T.47	"		50	863.8	861.9	"	0.03	1.5	6.9	4,000	862.52	861.02
E	E.7	E.6	85	340	75	887.8	886.2	8"	0.038	2.75	7.8	4,500	886.8	884.05
	E.6	E.5	"	"	50	886.2	883.3	"	0.038	1.9	7.8	4,500	884.05	882.15
	E.5	E.4	"	"	100	883.3	878.2	"	0.055	5.5	9.0	5,200	882.15	876.65
	E.4	E.3	"	"	70	878.2	876.1	"	0.025	1.75	6.0	3,500	876.65	874.9
	E.3	E.2	"	"	62	876.1	872.7	"	0.054	3.35	9.0	5,200	874.9	871.55
	E.2	E.2	"	"	68	872.7	869.5	"	"	3.66	"	"	871.55	867.90
	E.1	T.51	"	"	75	869.5	865.3	"	"	4.05	"	"	867.90	863.85

DESIGN OF MAIN SEWERS

SHEET NO. 5

LINE	FROM MANHOLE UPPER	FROM MANHOLE LOWER	Q_{av} in l.p.m	Q_{max} in l.p.m	LENGTH in METERS	GROUND ELEVATIONS in METERS UPPER MANHOLE	GROUND ELEVATIONS in METERS LOWER MANHOLE	DIA: OF PIPE INCHES	GRADE of SEWER	FALL of SEWER (meters)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL ft/sec	INVERT ELEVATIONS IN METERS UPPER MANHOLE	INVERT ELEVATIONS IN METERS LOWER MANHOLE
F	F.11	F.10	215	860	37	922.3	917.5	8	0.124	4.6	14.0	8,000	920.5	915.90
	F.10	F.9	"	"	63	917.5	909.3	"	0.138	8.7	15.0	8,500	915.90	907.2
	F.9	F.8	"	"	62	909.3	900.8	"	"	8.6	"	"	907.2	898.6
	F.8	F.7	"	"	61	900.8	894.8	"	0.092	5.6	12.5	7,200	898.6	893.0
	F.7	F.6	"	"	62	894.8	888.5	"	"	5.7	"	"	893.0	887.3
	F.6	F.5	"	"	78	888.5	888.8	"	0.008	0.63	3.2	1,900	887.3	886.67
	F.5	F.4	"	"	77	888.8	886.9	"	0.008	0.62	"	"	886.7	886.0
	F.4	F.3	"	"	78	886.9	880.2	"	0.096	7.5	12.5	7200	886.05	878.5
	F.3	F.2	"	"	45	880.2	877.0	"	0.081	3.65	11.0	6,500	878.5	874.85
	F.2	F.1	"	"	37	877.0	872.8	"	"	3.0	"	"	874.85	871.85
	F.1	T.56	"	"	50	872.8	869.0	"	0.056	2.8	9.5	5,500	871.85	869.0
G	G.16	G.15	131	524	85	929.8	926.4	8	0.0386	3.28	7.5	4,500	928.8	925.52
	G.15	G.14	"	"	65	926.4	921.6	"	0.084	5.46	11.0	6,500	925.52	920.04
	G.14	G.13	"	"	60	921.6	916.2	"	"	5.04	"	"	920.04	915.0
	G.13	G.12	"	"	55	916.2	912.6	"	0.062	3.41	10.3	6,000	915.0	911.59
	G.12	G.11	"	"	60	912.6	909.1	"	"	3.72	"	"	911.59	907.87
	G.11	G.10	"	"	60	909.1	905.3	"	"	"	"	"	907.87	904.15
	G.10	G.9	"	"	50	905.3	903.6	"	0.033	1.65	7.0	4,000	904.15	902.50
	G.9	G.8	"	"	50	903.6	901.9	"	"	"	"	"	902.50	900.85
	G.8	G.7	"	"	50	901.9	900.0	"	0.044	2.2	7.5	4,400	900.85	898.65

— Continued on next sheet

DESIGN OF MAIN SEWERS

SHEET NO 6

LINE	FROM MANHOLE		Q_{av} in l.p.m	Q_{max} in l.p.m	LENGTH in METERS	GROUND ELEVATIONS IN METERS		DIA. OF PIPE INCHES	GRADE of SEWER	FALL of SEWER	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL ft/sec	INVERT ELEVATIONS in METERS	
						UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
G (Continued)	G.7	G.6	131	524	50	900.0	897.6	8	0.044	2.2	7.5	4,400	898.65	896.45
	G.6	G.5	"	"	75	897.6	893.2	"	0.057	4.27	9.2	5,500	896.45	892.18
	G.5	G.4	"	"	75	893.2	689.5	"	"	"	"	"	892.18	887.91
	G.4	G.3	"	"	75	889.5	886.5	"	0.030	2.25	7.0	4,000	887.91	885.66
	G.3	G.2	"	"	60	886.5	883.3	"	0.052	3.12	9.2	5,500	885.66	882.54
	G.2	G.1	"	"	33	883.3	881.8	"	"	1.67	"	"	882-54	880.87
	G.1	T.63	"	"	63	881.8	881.9	"	0.008	0.54	3.2	1,900	880.87	880.33
H	H.12	H.11	160	640	75	937.1	930.6	8	0.093	7.0	12.0	7,000	936.0	929.0
	H.11	H.10	"	"	75	930.6	920.8	"	0.132	9.9	14.5	8,500	929.0	919.0
	H.10	H.9	"	"	50	920.8	913.6	"	0.140	7.0	15.0	9,000	919.0	912.1
	H.9	H.8	"	"	58	913.6	906.2	"	"	8.10	"	"	912.1	904.0
	H.8	H.7	"	"	42	906.2	904.1	"	0.016	0.67	4.0	2,400	904.0	903.33
	H.7	H.6	"	"	50	904.1	903.4	"	0.016	0.80	"	"	903.33	902.53
	H.6	H.5	"	"	75	803.4	899.9	"	0.06	4.5	9.7	5,800	902.53	898.03
	H.5	H.4	"	"	50	899.9	892.0	"	0.152	7.6	16.0	9,500	898.03	890.43
	H.4	H.3	"	"	58	892.0	891.2	"	0.008	0.46	3.2	1,900	890.43	889.97
	H.3	H.2	"	"	57	891.2	891.5	"	0.008	"	"	"	889.97	889.51
	H.2	H.1	"	"	52	891.5	886.8	"	0.007	4.0	11.0	6,400	889.51	885.51
	H.1	T.63	"	"	60	886.8	881.9	"	0.08	4.80	"	6,500	885.51	880.71

DESIGN OF MAIN SEWERS

SHEET NO: 7

LINE	FROM MANHOLE		Q_{av} in l.p.m	Q_{max} in l.p.m	LENGTH in METERS	GROUND ELEVATIONS IN METERS		DIA: of PIPE INCHES	GRADE of SEWER	FALL of SEWER meters	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l.p.m	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
I	I.19	I.18	298	1192	75	946.0	938.3	8	0.10	7.5	13.0	7,500	944.5	937.0
	I.18	I.17	"	"	75	938.3	930.8	"	0.10	7.5	"	"	937.0	929.5
	I.17	I.16	"	"	50	930.8	925.2	"	0.116	5.8	13.5	7,800	929.5	923.7
	I.16	I.15	"	"	50	925.2	919.0	"	"	"	"	"	923.7	917.9
	I.15	I.14	"	"	70	919.0	911.0	"	"	8.1	"	"	917.9	909.7
	I.14	I.13	"	"	68	911.0	904.2	"	0.10	6.8	13.0	7,500	909.7	902.9
	I.13	I.12	"	"	62	904.2	898.0	"	"	6.2	13.0	7,500	902.9	896.7
	I.12	I.11	"	"	50	898.0	893.5	"	0.094	4.7	13.5	7,400	896.7	892.0
	I.11	I.10	"	"	67	893.5	889.4	"	0.054	3.48	9.5	5,500	892.0	888.52
	I.10	I.9	"	"	63	889.4	888.8	"	0.005	0.32	2.55	15,00	888.52	888.20
	I.9	I.8	"	"	60	888.8	888.5	"	"	0.3	"	"	880.20	887.9
	I.8	I.7	"	"	60	888.5	888.4	"	"	"	"	"	887.9	887.6
	I.7	I.6	"	"	63	880.4	889.1	"	"	"	"	"	887.6	887.28
	I.6	I.5	"	"	75	889.1	890.5	"	"	0.32	"	"	887.28	886.9
	I.5	I.4	"	"	75	890.5	889.0	"	0.005	0.37	2.55	1500	886.9	886.53
	I.4	I.3	"	"	75	889.0	888.6	"	"	"	"	"	886.53	886.16
	I.3	I.2	"	"	75	888.6	888.2	"	"	"	"	"	885.16	885.79
	I.2	I.1	"	"	75	888.2	886.8	"	"	"	"	"	885.79	885.52
	I.1	T.64	"	"	62	886.8	884.8	"	0.028	1.74	6.6	3,800	885.52	883.78

DESIGN OF MAIN SEWERS

SHEET NO: 8

LINE	FROM MANHOLES		Q_{av} in l.p.m	Q_{max} in l.p.m	LENGTH IN METERS	GROUND ELEVATIONS IN METERS		DIA: of PIPE INCHES	GRADE of SEWER	FALL of SEWER (meters)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l.p.m	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
J	J.6	J.5	33	132	50	937.2	934.9	8	0.042	2.1	7.7	4,600	936.1	934.0
	J.5	J.4	"	"	50	934.9	932.8	"	"	"	"	"	934.0	931.9
	J.4	J.3	"	"	75	932.8	929.8	"	3.15	"	"	"	931.9	928.75
	J.3	J.2	"	"	38	929.8	926.7	"	0.096	3.65	12.5	7,200	928.75	925.10
	J.2	J.1	"	"	62	926.7	915.8	"	0.170	10.5	17.0	10,000	925.10	914.60
	J.1	T.80	"	"	25	915.8	912.3	"	0.144	3.6	15.0	9,000	914.60	911.0
K	K.7	K.6	68	272	75	946.1	943.1	8	0.04	3.0	8.0	4,600	945.0	942.0
	K.6	K.5	"	"	50	943.1	937.4	"	0.124	6.2	14.0	8,000	942.0	935.4
	K.5	K.4	"	"	50	937.4	931.2	"	"	"	"	"	935.4	929.2
	K.4	K.3	"	"	50	931.2	926.0	"	0.10	5.0	13.0	7,500	929.2	924.2
	K.3	K.2	"	"	75	926.0	920.5	"	0.072	5.4	11.0	6,400	924.2	918.8
	K.2	K.1	"	"	50	920.5	913.35	"	0.13	6.5	14.0	8,000	918.8	912.3
	K.1	T.81	"	"	75	913.35	913.5	"	0.008	0.6	3.2	1,900	912.3	911.7

DESIGN OF TRUNK SEWER

SHEET NO: 9

LINE	FROM MANHOLE UPPER	FROM MANHOLE LOWER	Q_{max} FROM MAIN in l.p.m	ACCUMULATIVE Q_{max} in l.p.m	LENGTH in METERS	GROUND ELEVATIONS IN METERS UPPER MANHOLE	GROUND ELEVATIONS IN METERS LOWER MANHOLE	DIA: of PIPE INCHES	GRADE of SEWER	FALL of SEWER (METERS)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l/m	INVERT ELEVATIONS IN METERS UPPER MANHOLE	INVERT ELEVATIONS IN METERS LOWER MANHOLE
T	T.99	T.98	-	780	65	959.90	956.5	8	0.053	3.45	8.8	5,000	958.2	954.75
	T.98	T.97	-	"	60	956.5	953.15	"	"	3.18	"	"	954.75	951.57
	T.97	T.96	-	"	60	953.15	949.25	"	0.037	4.37	11.0	6,000	951.57	947.20
	T.96	T.95	-	"	70	949.25	944.0	"	"	5.10	"	"	947.20	942.10
	T.95	T.94	-	"	70	944.0	938.7	"	"	5.10	"	"	942.10	937.00
	T.94	T.93	-	"	70	938.7	934.8	"	0.052	3.64	8.8	5,000	937.00	933.36
	T.93	T.92	-	"	55	934.8	932.0	"	"	2.86	"	"	933.36	930.52
	T.92	T.91	-	"	65	932.0	929.45	"	0.036	2.34	7.5	4,200	930.52	928.16
	T.91	T.90	-	"	60	929.45	927.45	"	"	2.16	"	"	928.16	926.0
	T.90	T.89	-	"	85	927.45	925.20	"	0.025	2.13	6.0	3,500	926.0	923.87
	T.89	T.88	-	"	80	925.20	923.45	"	"	2.0	"	"	923.87	921.87
	T.88	T.87	-	"	80	923.45	921.55	"	"	2.0	"	"	921.87	919.87
	T.87	T.86	-	"	75	921.55	919.40	"	"	1.87	"	"	919.87	918.0
	T.86	T.85	-	"	75	919.4	917.6	"	"	"	"	"	918.0	916.13
	T.85	T.84	-	"	80	917.6	916.25	"	0.016	1.28	4.8	2800	916.13	914.85
	T.84	T.83	-	"	80	916.25	915.0	"	"	"	"	"	914.85	913.57
	T.83	T.82	-	"	75	915.0	914.0	"	"	1.2	"	"	913.57	912.37
	T.82	T.81	-	"	55	914.0	913.45	"	"	0.88	"	"	912.37	911.49
	T.81	T.80	272	1052	41	913.45	912.45	"	0.027	1.1	6.4	3,700	911.49	910.39
	T.80	T.79	132	1184	64	912.45	910.0	"	0.032	2.05	6.5	3,800	910.39	908.34

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DESIGN OF TRUNK SEWER (Continued)

SHEET NO: 10

LINE	FROM MANHOLE		Q_{max}: FROM MAIN SEWER l.p.m	ACCUMULATIVE Q_{max} in l.p.m	LENGTH IN METERS	GROUND ELEVATIONS IN METERS		DIA: of PIPE inches	GRADE of SEWER	FALL of SEWER (meters)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL m	INVERT ELEVATIONS (IN METERS)	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
T (Contd)	T.79	T.78	-	1184	60	910.0	908.2	8	0.032	1.92	6.5	3800	908.34	906.42
	T.78	T.77	-	"	75	908.2	905.8	"	"	2.42	"	"	906.42	904.0
	T.77	T.76	-	"	60	905.8	904.1	"	0.026	1.56	6.1	3,500	904.0	902.46
	T.76	T.75	-	"	70	904.1	902.1	"	"	1.82	"	"	902.46	900.64
	T.75	T.74	-	"	70	902.1	900.45	"	"	"	"	"	900.64	898.82
	T.74	T.73	-	"	60	900.45	899.45	"	0.013	0.78	4.3	2,500	898.82	898.04
	T.73	T.72	-	"	65	899.45	898.7	"	"	0.85	"	"	898.04	897.19
	T.72	T.71	-	1839	75	898.7	899.0	"	0.008	0.6	3.25	1,900	897.19	896.59
	T.71	T.70	-	"	"	899.0	899.3	"	"	"	"	"	896.59	895.99
	T.70	T.69	-	"	"	899.3	899.8	"	"	"	"	"	895.99	895.39
	T.69	T.68	-	"	"	898.8	898.5	"	"	"	"	"	895.39	894.79
	T.68	T.67	-	"	"	898.5	896.7	"	"	"	"	"	894.79	894.19
	T.67	T.66	-	"	77	896.7	892.6	"	0.0376	2.9	7.5	4200	894.19	891.29
	T.66	T.65	-	"	63	892.6	888.9	"	0.065	"	10.0	5,800	891.29	887.19
	T.65	T.64	-	"	60	888.9	884.7	"	"	3.9	"	"	887.19	883.29
	T.64	T.63	1192	3086	60	884.7	881.9	"	0.052	3.12	9.0	5,000	883.29	880.17
	T.63	T.62	1164	4250	65	881.9	879.45	"	0.039	2.54	8.0	4,500	880.17	877.63
	T.62	T.61	-	"	65	879.45	877.8	10	0.023	1.5	6.6	6,000	877.63	876.13
	T.61	T.60	-	"	62	877.8	876.3	"	"	1.43	6.6	"	876.13	874.70
	T.60	T.59	"	5210	63	876.3	874.3	"	0.032	2.02	8.0	7,400	874.70	872.68

Continued on next sheet.

DESIGN OF TRUNK SEWER (Continued)

SHEET NO: 11

LINE	FROM MANHOLES		Q_{max} FROM MAIN SEWER	ACCUMULATIVE Q in	LENGTH IN METERS	GROUND ELEVATIONS IN METERS		DIA: OF PIPE	GRADE of SEWER	FALL of SEWER	VELOCITY FLOWING FULL	CAPACITY FLOWING FULL	INVERT ELEVATIONS in METERS	
	UPPER	LOWER	l.p.m	l.p.m	-	UPPER MANHOLE	LOWER MANHOLE	inches		(meters)	ft/sec.	l.p.m	UPPER MANHOLE	LOWER MANHOLE
T (Contd)	T.59	T.58	-	5,210	60	874.3	872.5	10	0.032	1.92	8.0	7,400	872.68	870.76
	T.58	T.57	-	"	65	872.5	871.15	"	0.02	1.3	6.1	5,600	870.76	869.46
	T.57	T.56	-	"	58	871.15	870.35	"	"	1.16	"	"	869.46	868.30
	T.56	T.55	860	6,070	72	870.35	869.20	12	0.011	0.79	5.0	6,500	868.30	867.51
	T.55	T.54	-	"	70	869.20	868.45	"	0.011	0.77	"	6,500	867.51	866.74
	T.54	T.53	-	6,475	60	868.45	868.0	14	0.011	0.66	5.5	9,600	866.74	866.08
	T.53	T.52	-	"	63	868.0	865.9	"	0.028	1.77	9.0	16,000	866.08	864.31
	T.52	T.51	-	"	58	865.9	865.4	"	0.011	0.64	5.5	9,500	864.31	863.57
	T.51	T.50	340	6,815	54	865.4	863.8	"	0.024	1.3	8.5	15,000	863.57	862.27
	T.50	T.49	-	"	72	863.8	862.4	"	0.009	0.65	4.8	9,000	862.27	861.62
	T.49	T.48	-	"	75	862.4	862.7	"	"	0.68	"	"	861.62	860.94
	T.48	T.47	-	8,145	58	862.7	861.85	"	"	0.52	"	"	860.94	860.42
	T.47	T.46	1320	9,465	81	861.85	860.4	"	0.024	1.94	8.5	15,000	860.42	858.48
	T.46	T.45	928	10,393	64	860.4	858.25	"	"	1.54	"	"	850.48	856.94
	T.45	T.44	-	"	60	858.25	856.7	"	"	1.44	"	"	856.94	855.50
	T.44	T.43	-	"	70	856.7	855.75	"	"	1.68	"	"	855.50	853.82
	T.43	T.42	-	"	70	855.75	853.80	"	"	1.68	"	"	853.82	852.14
	T.42	T.41	-	11,178	60	853.8	853.2	16	0.012	0.72	6.2	14,000	852.14	851.42
	T.41	T.40	-	"	60	853.2	852.25	"	"	"	"	"	851.42	850.70
	T.40	T.39	-	"	75	852.25	851.4	"	"	0.90	"	"	850.70	849.80

Continued on next sheet.

DESIGN OF TRUNK SEWER (Continued)

SHEET NO: 12

LINE	FROM MAN HOLES		Q_{max} FROM MAIN SEWER l.p.m	ACCUMULATIVE Q. in l.p.m	LENGTH IN METERS	GROUND ELEVATIONS IN METERS		DIA of PIPE INCHES	GRADE of SEWER	FALL of SEWER (meters)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL l.p.m	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
T (Contd)	T. 39	T. 38	1108	12,286	67	851.4	849.2	16	0.034	2.28	11.0	25,000	849.80	847.52
	T. 38	T. 37	"	"	78	849.2	848.55	"	0.016	1.25	7.4	17,000	847.52	846.27
	T. 37	T. 36	–	"	75	848.55	846.8	"	"	1.2	"	"	846.27	845.07
	T. 36	T. 35	–	"	85	846.8	845.5	"	"	1.36	"	"	845.07	843.71
	T. 35	T. 34	–	"	75	845.5	844.25	"	"	1.2	"	"	843.71	842.51
	T. 34	T. 33	–	"	75	844.25	843.5	"	0.011	0.82	6.0	14,000	842.51	841.69
	T. 33	T. 32	–	"	75	843.5	842.4	"	"	"	"	"	841.69	840.87
	T. 32	T. 31	–	"	85	842.4	841.25	"	0.015	1.28	7.0	16,500	840.87	839.59
	T. 31	T. 30	–	"	80	841.25	840.15	"	"	1.20	"	"	839.59	838.39
	T. 30	T. 29	–	"	80	840.15	838.6	"	"	"	"	"	838.39	837.19
	T. 29	T. 28	–	"	80	838.6	837.6	"	"	"	"	"	837.19	836.00
	T. 28	T. 27	–	"	75	837.6	837.4	"	0.009	0.67	5.2	14,000	836.0	835.33
	T. 27	T. 26	–	"	75	837.4	836.45	"	"	"	"	"	835.33	834.66
	T. 26	T. 25	–	"	75	836.45	835.55	"	"	"	"	"	834.66	834.00
	T. 25	T. 24	–	"	75	835.55	834.25	"	0.018	1.35	8.0	17000	834.00	832.65
	T. 24	T. 23	–	"	75	834.25	832.7	"	"	"	"	"	832.65	831.30
	T. 23	T. 22	–	"	80	832.7	832.45	"	0.005	0.40	4.2	14,500	831.30	829.90
	T. 22	T. 21	–	"	75	832.45	832.80	"	"	0.38	"	"	829.90	829.52
	T. 21	T. 20	–	"	75	832.80	832.80	"	"	"	"	"	829.52	829.12
	T. 20	T. 19	–	"	75	832.80	831.80	"	"	"	"	"	829.12	828.74

DESIGN OF TRUNK SEWER (Continued)

SHEET NO: 13

LINE	FROM MANHOLES		Q_{max} FROM MAIN SEWER l.p.m	ACCUMULATIVE Q in l.p.m	LENGTH IN METERS	GROUND ELEVATIONS IN METERS		DIA OF PIPE inches	GRADE OF SEWER	FALL OF SEWER (meters)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL ft	INVERT ELEVATIONS IN METERS	
	UPPER	LOWER				UPPER MANHOLE	LOWER MANHOLE						UPPER MANHOLE	LOWER MANHOLE
T (Contd)	T.19	T.18	–	12,286	75	831.8	831.5	18	0.005	0.38	4.2	14,500	828.74	828.36
	T.18	T.17	–	"	70	831.5	831.15	"	"	0.35	"	"	828.36	828.0
	T.17	T.16	–	"	75	831.15	830.0	"	"	0.38	"	"	828.0	828.62
	T.16	T.15	–	"	75	830.0	828.6	"	0.023	1.73	10.0	28,000	828.62	826.89
	T.15	T.14	–	"	75	828.6	826.70	"	"	"	"	"	826.89	825.16
	T.14	T.13	–	"	75	826.70	824.85	"	"	"	"	"	825.16	823.43
	T.13	T.12	–	"	75	824.85	823.15	"	"	"	"	"	823.43	821.70
	T.12	T.11	–	"	75	823.15	821.3	"	"	"	"	"	821.70	819.97
	T.11	T.10	–	"	75	821.3	819.8	"	"	"	"	"	819.97	818.24
	T.10	T.9	–	"	65	819.8	819.4	"	0.008	0.52	5.4	16,000	818.24	817.72
	T.9	T.8	–	"	70	819.4	818.75	"	"	0.56	"	"	817.72	817.16
	T.8	T.7	–	"	70	818.75	817.75	"	0.014	0.98	7.3	21,000	817.16	816.18
	T.7	T.6	–	"	70	817.75	816.7	"	"	"	"	"	816.18	815.20
	T.6	T.5	–	"	70	816.7	815.90	"	"	"	"	"	815.20	814.22
	T.5	T.4	–	"	70	815.90	814.7	"	"	"	"	"	814.22	813.24
	T.4	T.3	–	"	65	814.7	814.1	"	"	0.91	"	"	813.24	812.33
	T.3	T.2	–	"	70	814.1	813.1	"	0.005	0.35	4.3	12,500	812.33	811.98
	T.2	T.1 (TREATMENT PLANT MANHOLE)	–	"	65	813.1	812.4	"	"	0.33	"	"	811.98	811.65

CALCULATION OF CONTRIBUTING AREAS & INLET CONCENTRATION TIME

NAME OF THE DRAINAGE AREA	CONTRIBUTING AREA IN ACRES	AVERAGE SLOPE OF WATER SHED	DISTANCE FROM REMOTEST PT: in feet	TIME OF INLET: CONCENTRATION T in mts	
a	81	50	1870	15	
b	81	50	1280	13	
c	26	70	1510	12	
d	36	90	1580	11.3	
e	25.5	125	2000	12.0	
f	50.	70	2170	13.5	
g	45	220	990	8.0	
h	75	35	4750	23.0	
j	55	75	1480	11.8	
k	70	175	1300	9.5	
l	99	85	3120	15.5	
m	49	90	1480	11.0	
n	32.5	210	1150	8.8	
p	48.0	100	1950	12.0	

SHEET NO: 15

DESIGN OF STORM WATER SEWER SYSTEM (By Lloyd Davies & Tangent Method with Imp: factor 0.5)

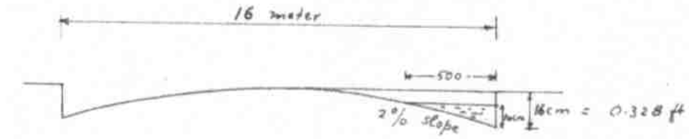
MANHOLES		NAME OF CONTRIBUTING DRAINAGE ARES (in Acres)	CONTRIBUTING DRAINAGE AREAS (in Acres)	IMPERVIOUS AREAS IN (acres)	TIME OF CONCENTRATION IN MINUTES	RAINFALL INTENSITY IN inches/hr	RUN-OFF in c.f.s.	SLOPE	DIAMETER OF SEWER (in inches)	VELOCITY FLOWING FULL ft/sec	CAPACITY FLOWING FULL c.f.s.	LENGTH OF SEWER		TIME IN SEWER in Mts:	GROUND ELEVATIONS (in Meters)		INVERT ELEVATIONS (in Meters)	
FROM	TO											IN METERS	IN FEET		UPPER MANHOLE	LOWER MANHOLE	UPPER MANHOLE	LOWER MANHOLE
S. 52	S. 47	a & b	162	81	15	1.09	88	0.025	36"	14.75	105	400	1320	1.48	928.60	918.40	926.0	916.0
S.47	S.45	"	"	"	"	"	"	0.018	36"	12.8	90	150	490	0.64	918.40	915.55	916.0	913.3
S. 45	S. 43	a, b, c, & d	224	112	17.12	1.00	112	0.014	42	12.5	120	175	570	0.76	915.55	913.35	913.3	910.85
S.43	S.38	a, b .... to e	249.5	124.75	17.88	0.969	121	0.030	42	19.0	180	375	1230	1.08	913.35	901.8	910.85	898.85
S.38	S. 31	a, b..... to g	344.5	172.25	18.96	0.937	162	0.0145	48	13.5	165	600	1978	2.44	901.8	895.0	898.85	890.15
S. 31	S. 28	"	"	"	"	"	"	0.048	48	24.0	300	250	820	0.57	895.0	880.5	890.15	878.15
S. 28	S. 24	a, b, ... to j	474.5	237.25	21.97	0.855	203	0.029	48	19.5	245	280	918	0.78	880.5	872.7	878.15	870.0
S. 24	S. 20	"	"	"	"	"	"	0.014	56	12.5	205	320	1050	1.40	872.7	868.1	870.0	865.5
S. 20	S. 18	"	"	"	"	"	"	0.026	56	19.5	300	150	490	0.42	868.1	864.8	865.5	861.6
S. 18	S. 15	a, b, c ... to k	544.5	272.25	24.0	0.832	248	0.010	60	13.2	260	250	820	1.07	864.8	861.8	861.6	859.1
S. 15	S. 12	a, b, c ... to l	643.5	321.75	24.0	"	292	0.028	60	22.0	420	230	755	0.63	861.8	856.2	859.1	852.66
S. 12	S. 8	a, b, c, ... to m	692.5	346.25	24.0	"	315	0.0165	60	16.5	320	295	970	1.03	856.2	851.4	852.66	847.96
S. 8	S. 1	a b, c .... to p	772.7	350.0	24.0	"	316	0.016	60	16.5	320	225	740	0.75	851.4	842.6	847.96	839.4

DESIGN OF STORM WETER SEWER SYSTEM & GUTTERS (BY Lloyds Davies Method)

SHEET NO: 16

MANHOLES		NAME OF CONTRIBUTING DRAINAGE AREAS	Run off by Davies Method as per sheet No 15	Runoff taken up by Gutters cfs	Balance Runoff cfs	Required Dia of SEWER inches	Designed Q cfs	Designed V ft/sec	Designed Slope
FROM	TO								
S.52	S.47	a & b	88	37	51	30"	65	13.2	0.025
S.47	S.45	"	"	"	"	"	55	11.3	0.018
S.45	S.43	a, b, c & d	112	31	81	36"	82	11.4	0.014
S.43	S.38	a, b, .. to e	121	41	80	36"	120	16.4	0.03
S.38	S.31	a, b .. to g	162	5.9	156	48"	170	14.0	0.015
S.31	S.28	"	"	"	"	48"	300	23.0	0.048
S.28	S.24	a, b .. to j	203	5.4	198	48"	250	19.5	0.029
S.24	S.20	"	"	"	"	56"	205	12.5	0.014
S.20	S.18	"	"	"	"	56"	300	19.5	0.026
S.18	S.15	a, b, c to k	248	4.4	244	60"	260	13.2	0.01
S.15	S.12	a, b .. to l	292	5.8	286	60"	420	22.0	0.028
S.12	S.8	a b .... to m	315	4.7	310	60"	320	16.5	0.0165
S.8	S.1	a, b ... to p	316	5.7	310	60"	320	16.5	0.016

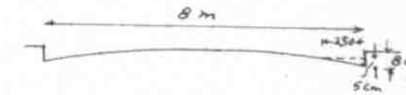
SECTION OF 16 Meter Wide Road



Cross Sectional Area $= \frac{1}{2} \times 500 \times 10 = 2500\ cm^2 = 2.69$ sft

For two sides of streets, area = 5.38 sft

SECTION OF 8 Meter Wide Road



Cross Sectional Area $= \frac{1}{2} \times 250 \times 5$

$= 625\ cm^2 = 0.67$ sft

For two sides of streets, area = 1.34 sft

$$Q = \frac{0.557}{n} \cdot z \cdot s^{1/2} \cdot d^{8/3}$$

$$z = \frac{1}{0.02} = 50$$

$n = 0.013$ (assumed)

s = slope of watershed

d = depth of water in ft.

Q for 16 Meter Wide Section

$$= \frac{0.557}{0.013} \times 50 \times 0.328^{8/3}\ s^{1/2}$$

$$= 118.5 \times s^{1/2}$$

Q for 8 Meter Wide Section

$$= \frac{0.557}{0.013} \times 50 \times 0.164^{8/3}\ s^{1/2}.$$

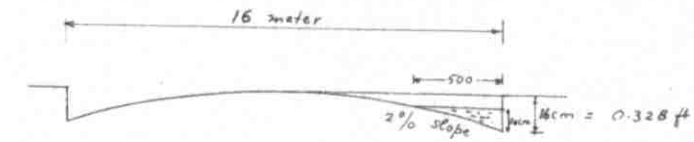
$$= 18.6 \times s^{1/2}$$

DESIGN OF STORM WETER SEWER SYSTEM & GUTTERS (BY Lloyds Davies Method)

SHEET NO: 16

MANHOLES FROM	MANHOLES TO	NAME OF CONTRIBUTING DRAINAGE AREAS	Runoff by Davies Method as per sheet No. 15	Runoff taken up by Gutters cfs	Balance Runoff cfs	Required Dia of SEWER inches	Designed Q cfs	Designed V ft/sec	Designed Slope
S.52	S.47	a & b	88	37	51	30"	65	13.2	0.025
S.47	S.45	"	"	"	"	"	55	11.3	0.018
S.45	S.43	a, b, c & d	112	31	81	36"	82	11.4	0.014
S.43	S.38	a, b, ... to e	121	41	80	36"	120	16.4	0.03
S.38	S.31	a, b .. to g	162	5.9	156	48"	170	14.0	0.015
S.31	S.28	"	"	"	"	48"	300	23.0	0.048
S.28	S.24	a, b .. to j	203	5.4	198	48"	250	19.5	0.029
S.24	S.20	"	"	"	"	56"	205	12.5	0.014
S.20	S.18	"	"	"	"	56"	300	19.5	0.026
S.18	S.15	a, b, c to k	248	4.4	244	60"	260	13.2	0.01
S.15	S.12	a, b ..... to l	292	5.8	286	60"	420	22.0	0.028
S.12	S.8	a, b .... to m	315	4.7	310	60"	320	16.5	0.0165
S.8	S.1	a, b ... to p	316	5.7	310	60"	320	16.5	0.016

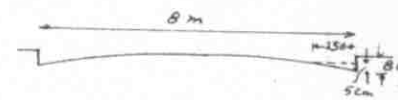
SECTION OF 16 Meter Wide Road



Cross Sectional Area $= \frac{1}{2} \times 500 \times 10 = 2500 \text{ cm}^2 = 2.69$ sft

For two sides of streets, area = 5.38 sft

SECTION OF 8 Meter Wide Road



Cross Sectional Area $= \frac{1}{2} \times 250 \times 5$

$= 625 \text{ cm}^2 = 0.67$ sft

For two sides of streets, area = 1.34 sft

$$Q = \frac{0.557}{n} \cdot z \cdot s^{1/2} \, d^{8/3}$$

$$z = \frac{1}{0.02} = 50$$

$n = 0.013$ (assumed)

s = slope of watershed

d = depth of water in ft.

Q for 16 Meter Wide Section

$$= \frac{0.557}{0.013} \times 50 \times 0.328^{8/3} \; s^{1/2}$$

$$= 118.5 \times s^{1/2}$$

Q for 8 Meter Wide Section

$$= \frac{0.557}{0.013} \times 50 \times 0.164^{8/3} \cdot s^{1/2}$$

$$= 18.6 \times s^{1/2}$$

NUMBER OF MANHOLES

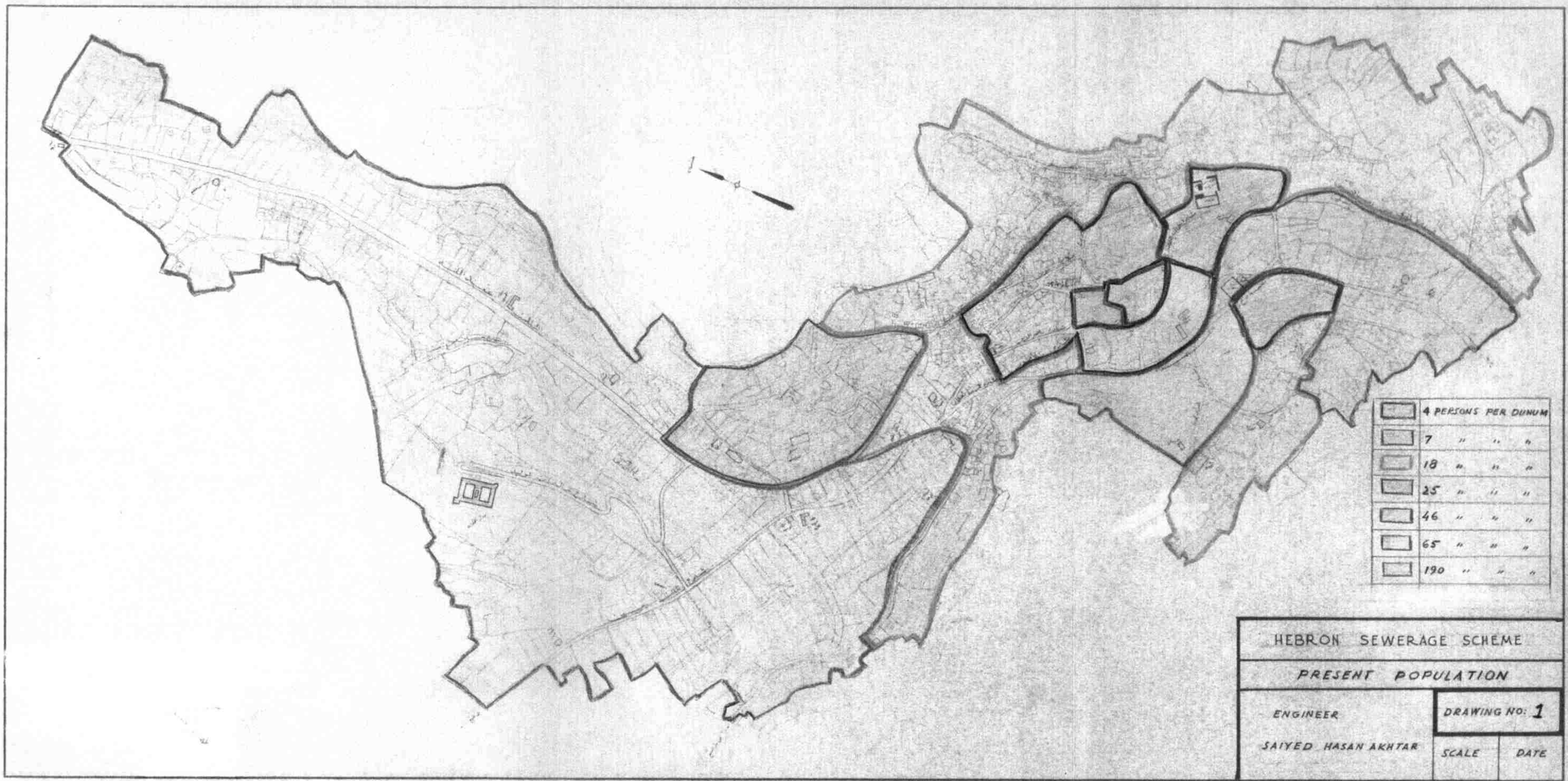
SHEET NO: 17

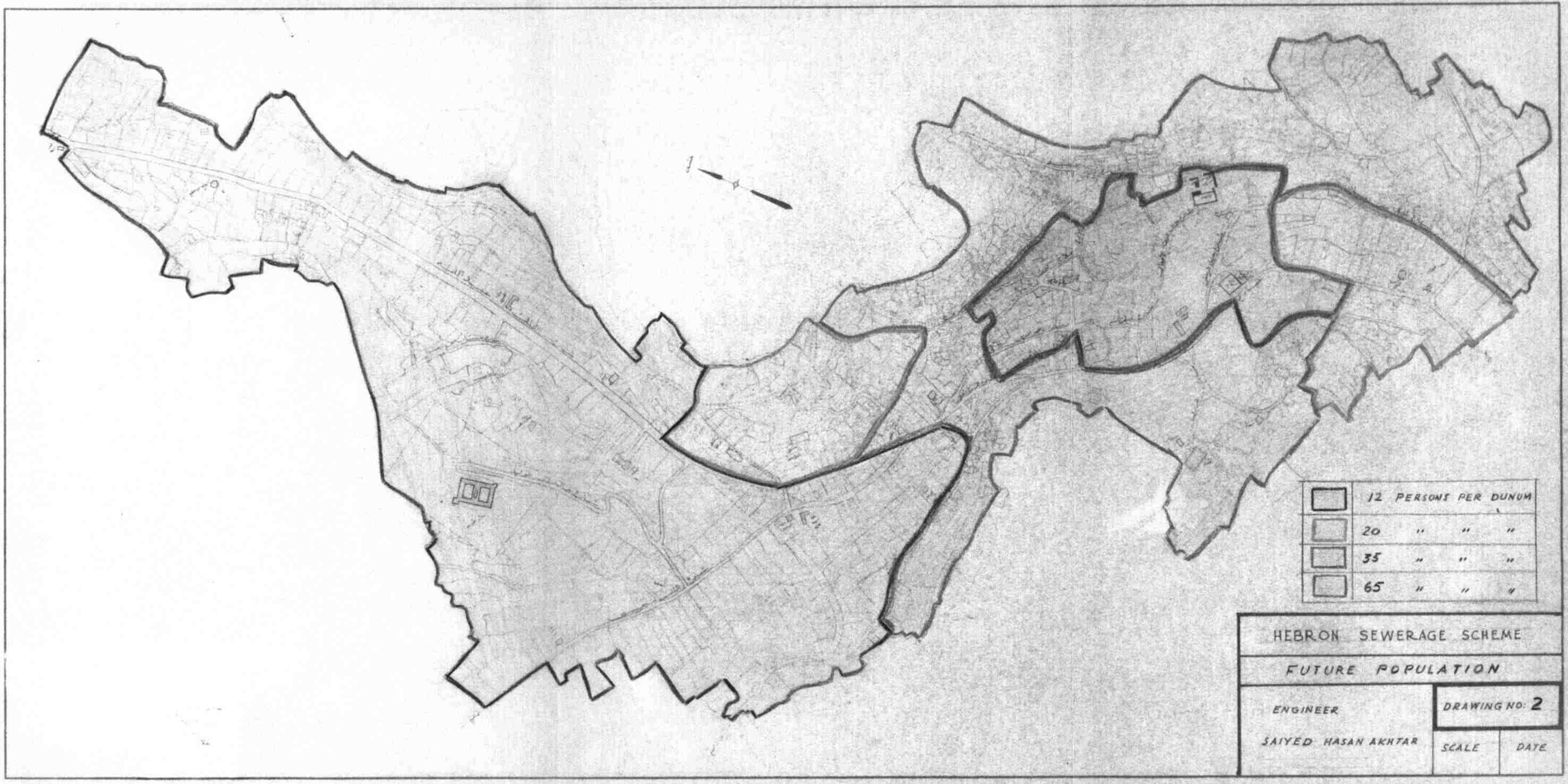
NAME of SEWER	NUMBER OF MANHOLES									
	8" φ		10" φ			12" φ	14" φ	16" φ	18" φ	
	Below $1\frac{1}{2}$ m depth	$1\frac{1}{2}$ m - 3 m depth	Below $1\frac{1}{2}$ m depth	$1\frac{1}{2}$ - 3 m. depth	Above 3 m depth	Below $1\frac{1}{2}$ m depth	Below $1\frac{1}{2}$ m depth	Below $1\frac{1}{2}$ m depth	Below $1\frac{1}{2}$ m depth	$1\frac{1}{2}$ m - 3 m depth
MAIN SEWER 'A'	12	-	-	-	-	-	-	-	-	-
" 'B'	6	-	-	-	-	-	-	-	-	-
" 'C'	3	-	-	-	-	-	-	-	-	-
" 'D'	9	4	-	-	-	-	-	-	-	-
" 'E'	6	-	-	-	-	-	-	-	-	-
" 'F'	8	3	-	-	-	-	-	-	-	-
" 'G'	16	-	-	-	-	-	-	-	-	-
" 'H'	9	3	-	-	-	-	-	-	-	-
" 'I'	13	6	-	-	-	-	-	-	-	-
" 'J'	6	-	-	-	-	-	-	-	-	-
" 'K'	7	-	-	-	-	-	-	-	-	-
TRUNK SEWER	27	-	11	1	4	3	11	20	17	5
TOTAL	122	16	11	1	4	3	11	20	17	5

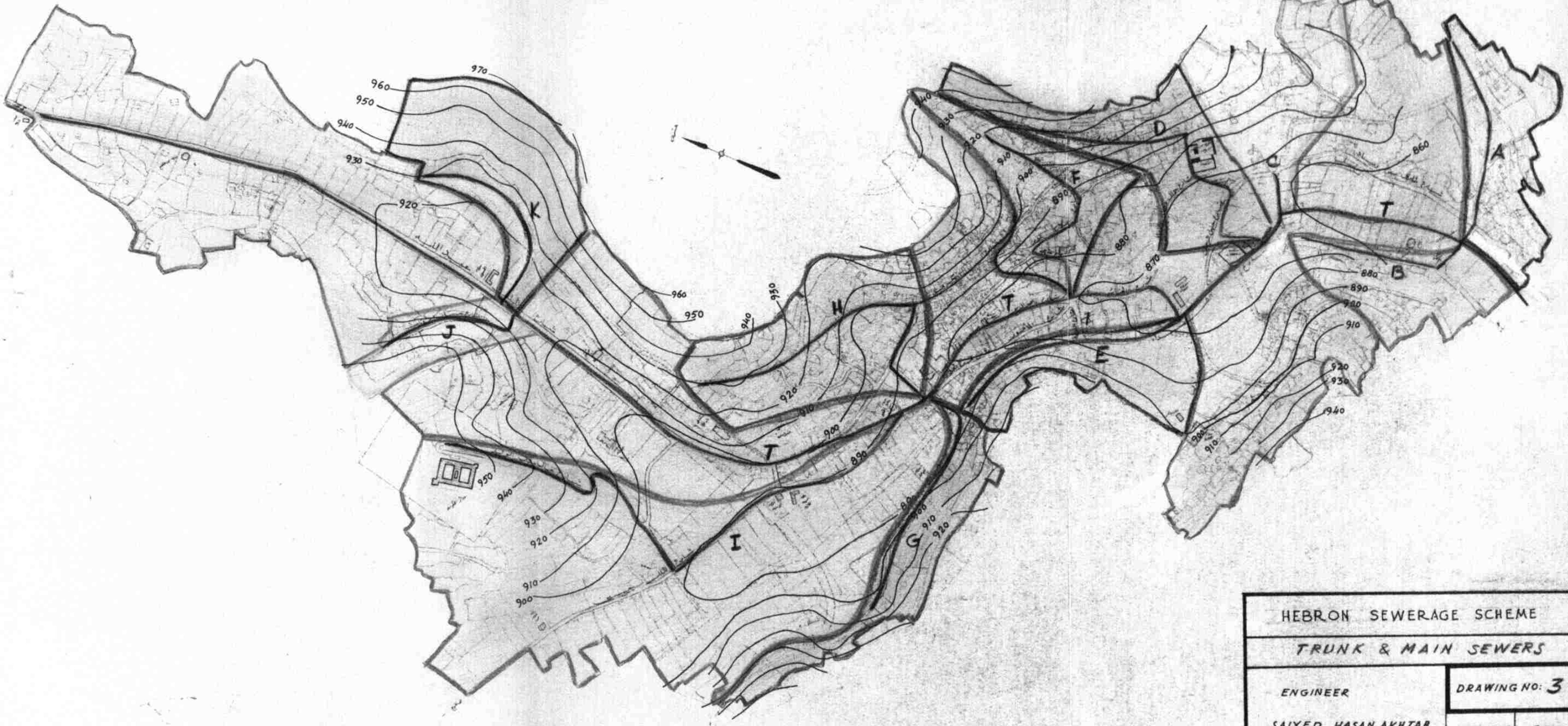
QUANTITY OF SEWER PIPE

SHEET NO: 18

NAME of SEWER	LENGTH OF SEWERS IN METERS.									
	8" φ		10" φ			12" φ	14" φ	16" φ	18" φ	
	BELOW 1½ m. depth	1½ m-3m depth	BELOW 1½ m. depth	1½ m-3m depth	ABOVE 3 m depth	BELOW 1½ m depth	BELOW 1½ m depth	BELOW 1½ m depth	BELOW 1½ m depth	1½ m-3m depth
MAIN SEWER 'A'	775	-	-	-	-	-	-	-	-	-
MAIN SEWER 'B'	387	-	-	-	-	-	-	-	-	-
MAIN SEWER 'C'	166	-	-	-	-	-	-	-	-	-
MAIN SEWER 'D'	572	278	-	-	-	-	-	-	-	-
MAIN SEWER 'E'	500	-	-	-	-	-	-	-	-	-
MAIN SEWER 'F'	497	183	-	-	-	-	-	-	-	-
MAIN SEWER 'G'	960	-	-	-	-	-	-	-	-	-
MAIN SEWER 'H'	625	75	-	-	-	-	-	-	-	-
MAIN SEWER 'I'	813	437	-	-	-	-	-	-	-	-
MAIN SEWER 'J'	300	-	-	-	-	-	-	-	-	-
MAIN SEWER 'K'	425	-	-	-	-	-	-	-	-	-
TRUNK SEWER	1825	-	621	77	375	142	785	1425	1220	370
TOTAL	7845	973	621	77	375	142	785	1425	1220	370


4 PERSONS PER DUNUM
7 " " "
18 " " "
25 " " "
46 " " "
65 " " "
190 " " "
HEBRON SEWERAGE SCHEME
PRESENT POPULATION
ENGINEER
SAIYED HASAN AKHTAR
DRAWING NO: 1
SCALE
DATE


12 PERSONS PER DUNUM
20 " " "
35 " " "
65 " " "
HEBRON SEWERAGE SCHEME
FUTURE POPULATION
ENGINEER
SAIYED HASAN AKHTAR
DRAWING NO: 2
SCALE
DATE



HEBRON SEWERAGE SCHEME

TRUNK & MAIN SEWERS

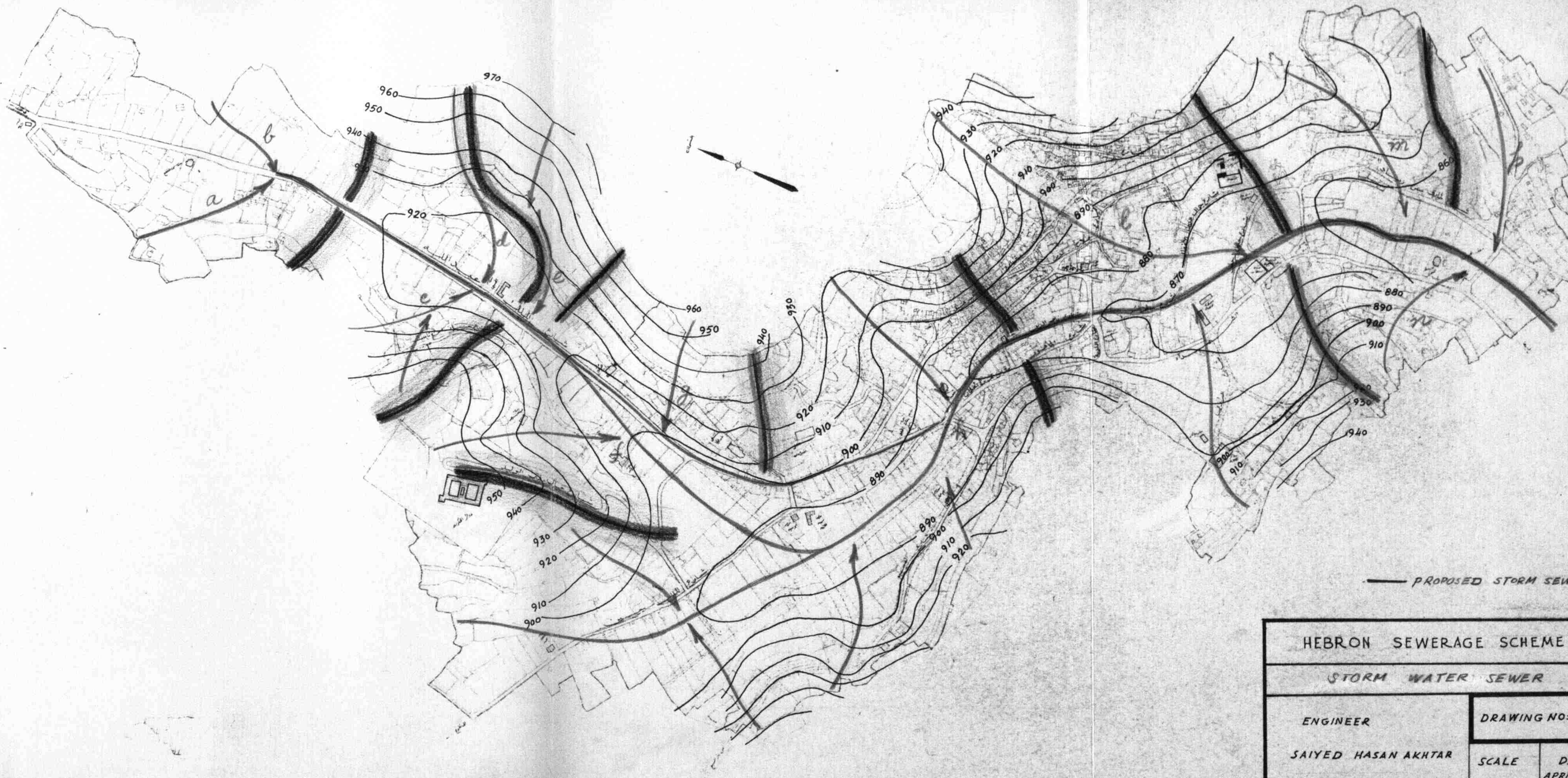
ENGINEER

SAIYED HASAN AKHTAR

DRAWING NO: 3

SCALE

DATE APRIL 1965



— PROPOSED STORM SEWER

HEBRON SEWERAGE SCHEME

STORM WATER SEWER

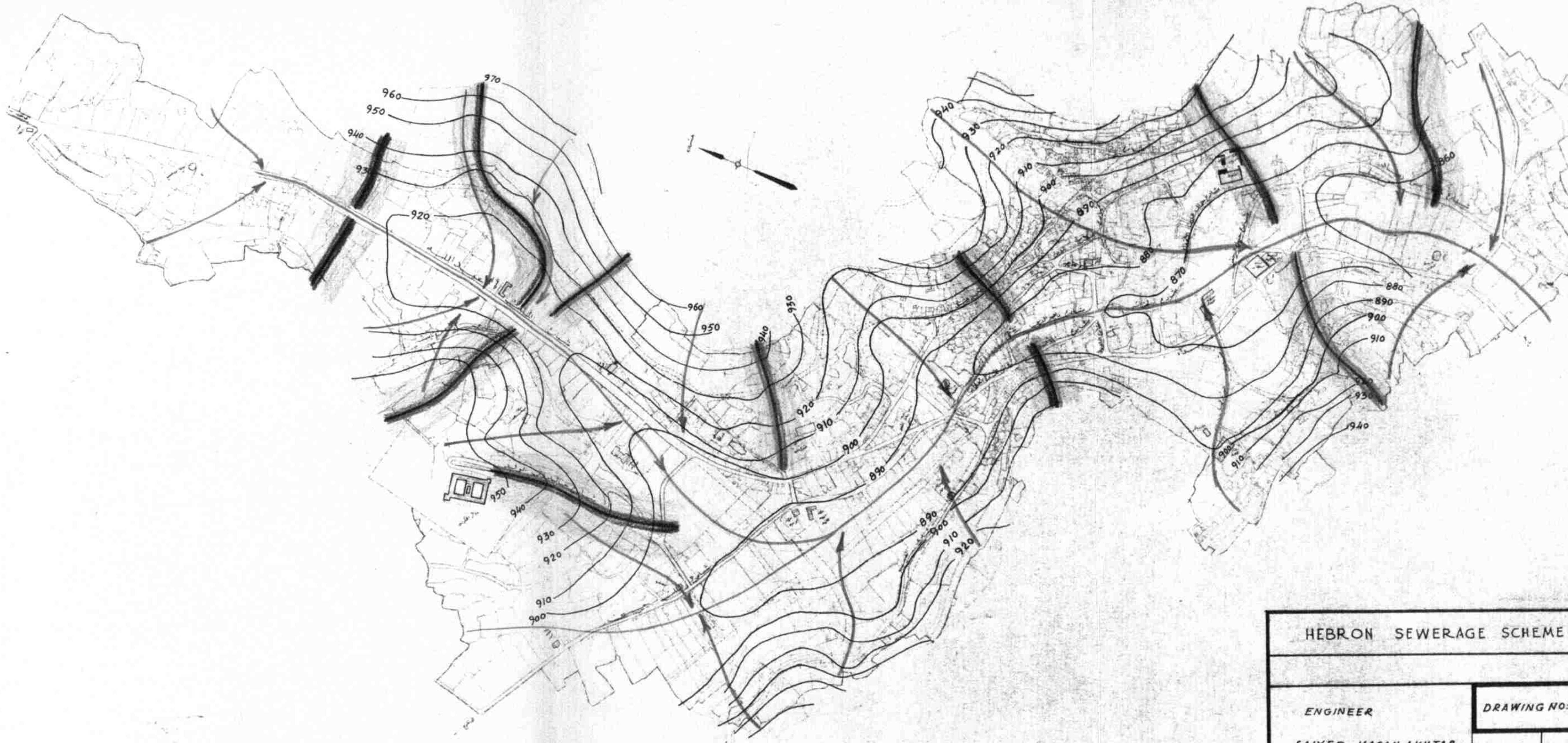
ENGINEER

SAIYED HASAN AKHTAR

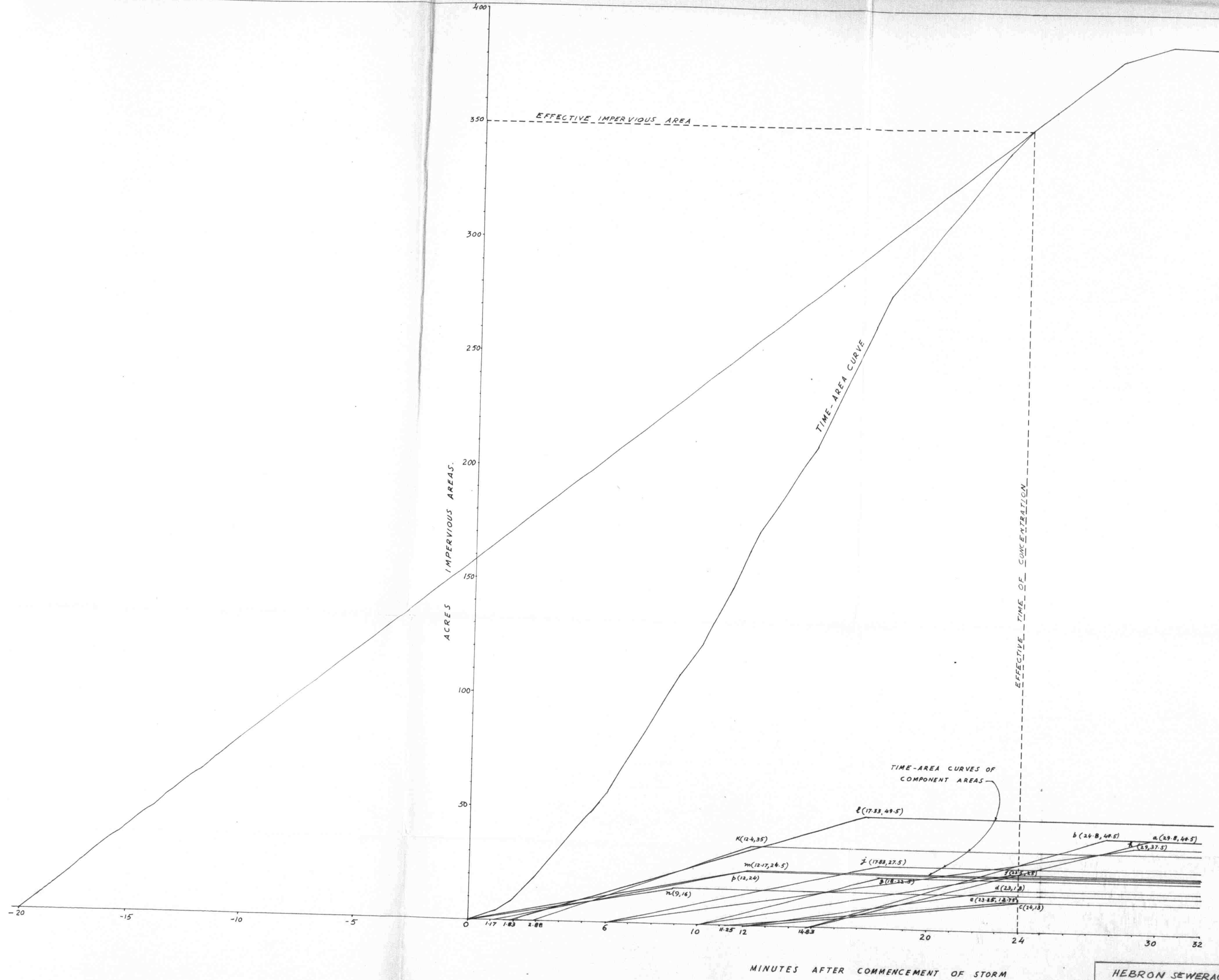
DRAWING NO: 4

SCALE

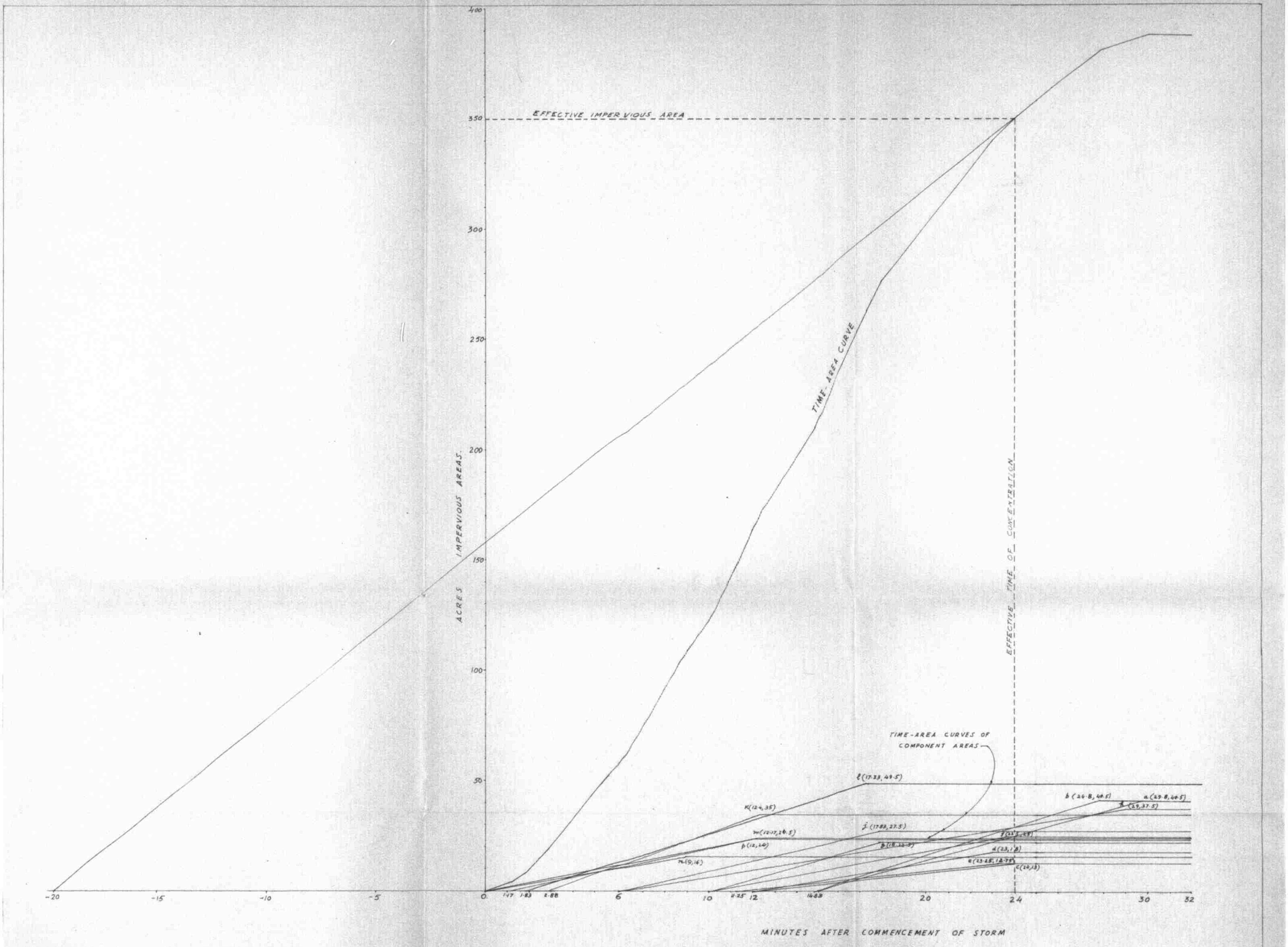
DATE
APRIL 1965



HEBRON SEWERAGE SCHEME		
ENGINEER	DRAWING NO:	
SAIYED HASAN AKHTAR	SCALE	DATE APRIL 1965



HEBRON SEWERAGE SCHEME	
TIME AREA CURVE	
ENGINEER SAIYED HASAN AKHTAR ADVISOR PROFESSOR SAMIR EL-KHORI	5 SCALE As shown / DATE APR. 1965

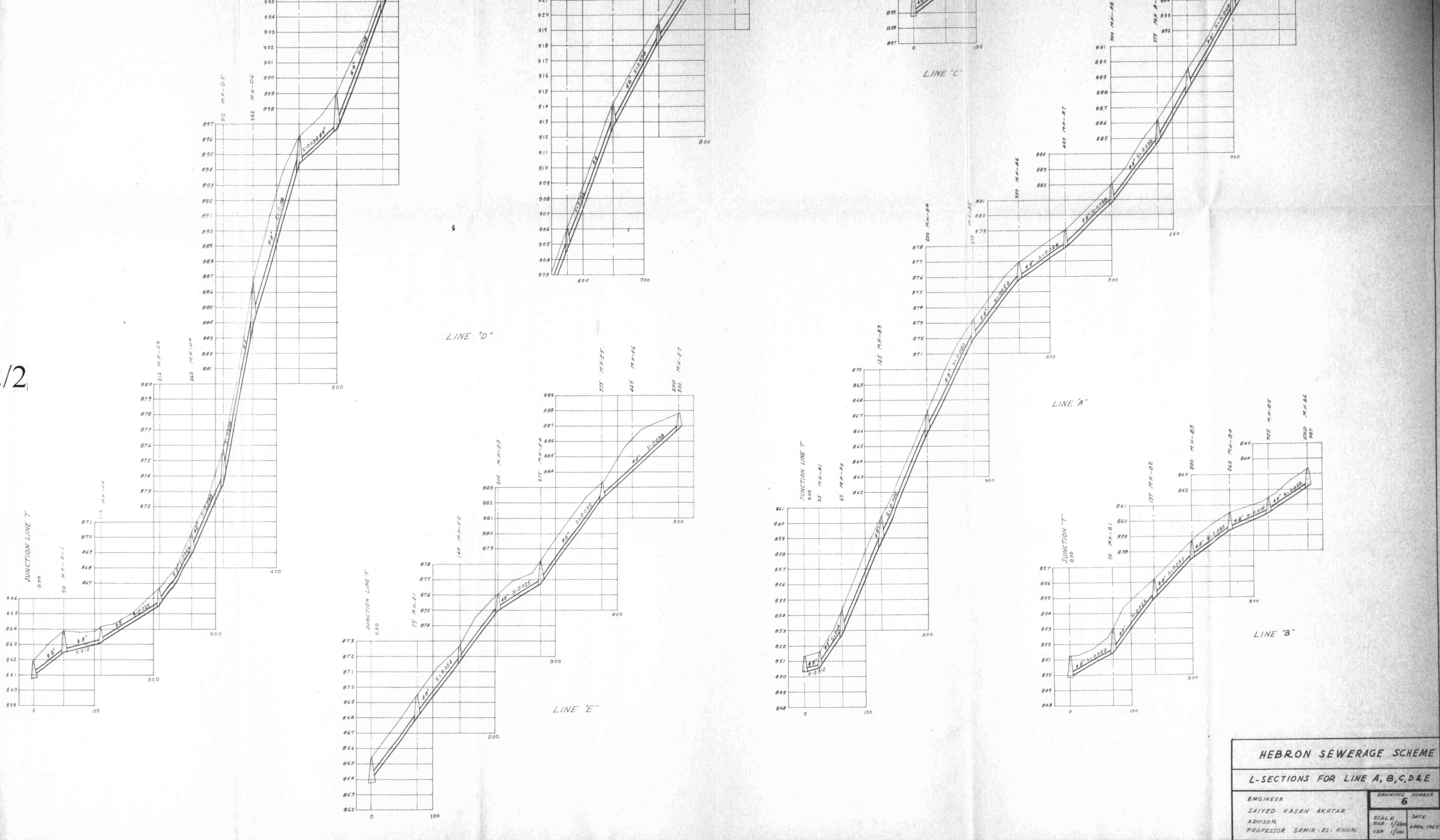


EFFECTIVE IMPERVIOUS AREA
TIME-AREA CURVE
EFFECTIVE TIME OF CONCENTRATION
TIME-AREA CURVES OF COMPONENT AREAS
ℓ (17.33, 49.5)
K(12.4, 35)
j (17.83, 27.5)
m(12.17, 26.5)
p (12, 24)
n(9, 16)
g (18, 22.5)
b (24.8, 40.5)
a (29.8, 40.5)
h (29, 37.5)
d (23, 18)
e (23.25, 12.75)
c (24, 13)
ACRES IMPERVIOUS AREAS.
MINUTES AFTER COMMENCEMENT OF STORM
400
350
300
250
200
150
100
50
-20
-15
-10
-5
0.
1.17
1.83
2.88
6
10
11.25
12
14.83
20
24
30
32

1/2

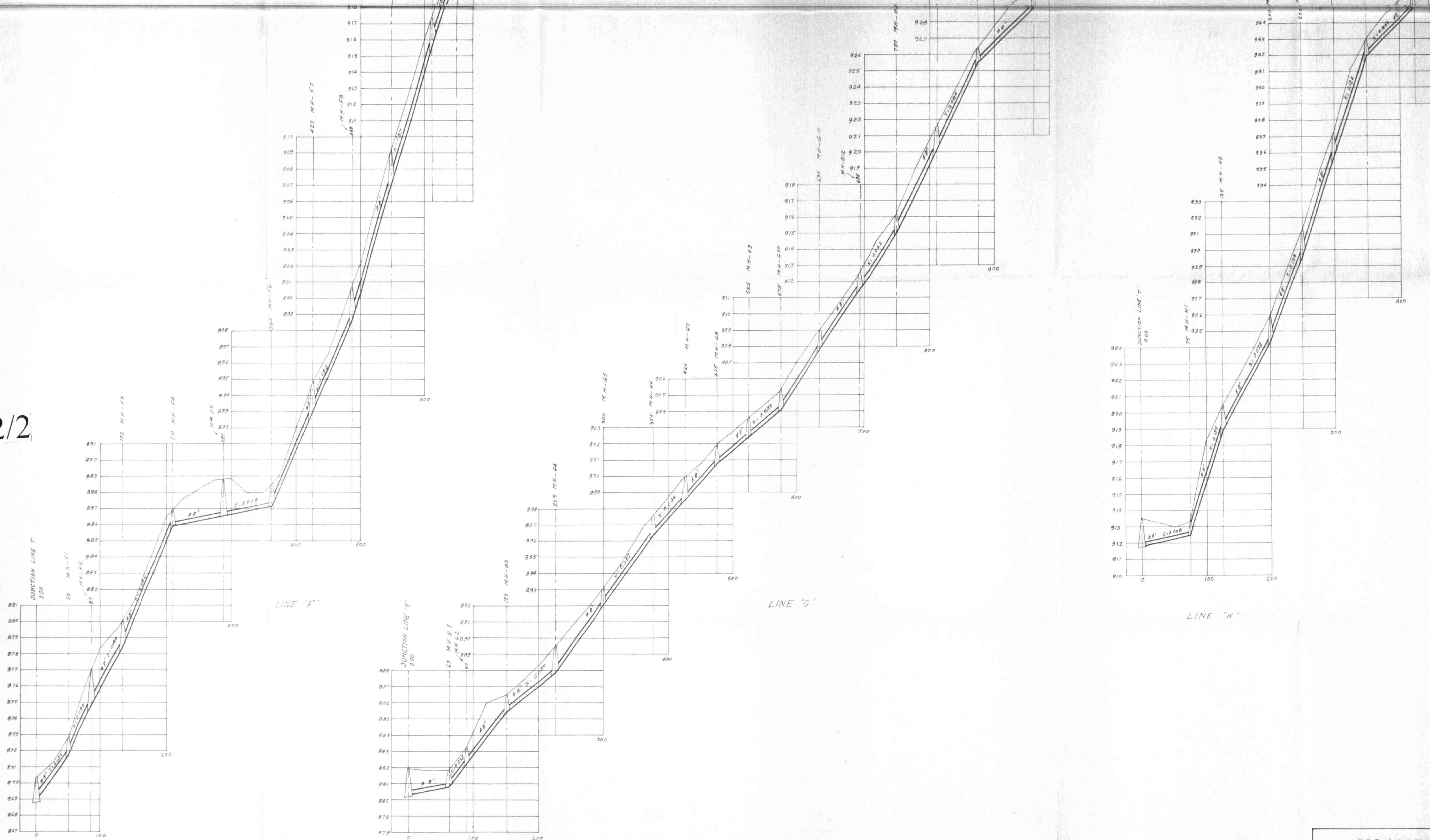

LINE "D"
LINE "C"
LINE "A"
LINE "B"
JUNCTION LINE T
M.H.-D-1
M.H.-D2
M.H.-D3
M.H.-D4
M.H.-D5
M.H.-D6
M.H.-D7
M.H.-D8
M.H.-D9
M.H.-D10
M.H.-D11
M.H.-D12
M.H.-D13
M.H.-C1
M.H.-C2
M.H.-C3
M.H.-A1
M.H.-A2
M.H.-A3
M.H.-A4
M.H.-A5
M.H.-A6
M.H.-A7
M.H.-A8
M.H.-A9
M.H.-A10
M.H.-A11
M.H.-A12
M.H.-B1
M.H.-B2
M.H.-B3
M.H.-B4
M.H.-B5
M.H.-B6

2/2


LINE "C"
LINE "D"
LINE "A"
LINE "E"
LINE "B"
HEBRON SEWERAGE SCHEME
L-SECTIONS FOR LINE A, B, C, D & E
ENGINEER
SAIYED HASAN AKHTAR
ADVISOR
PROFESSOR SAMIR-EL-KHURI
DRAWING NUMBER
6
SCALE
HOR: 1/2500
VER 1/100
DATE
APRIL 1965

1/2

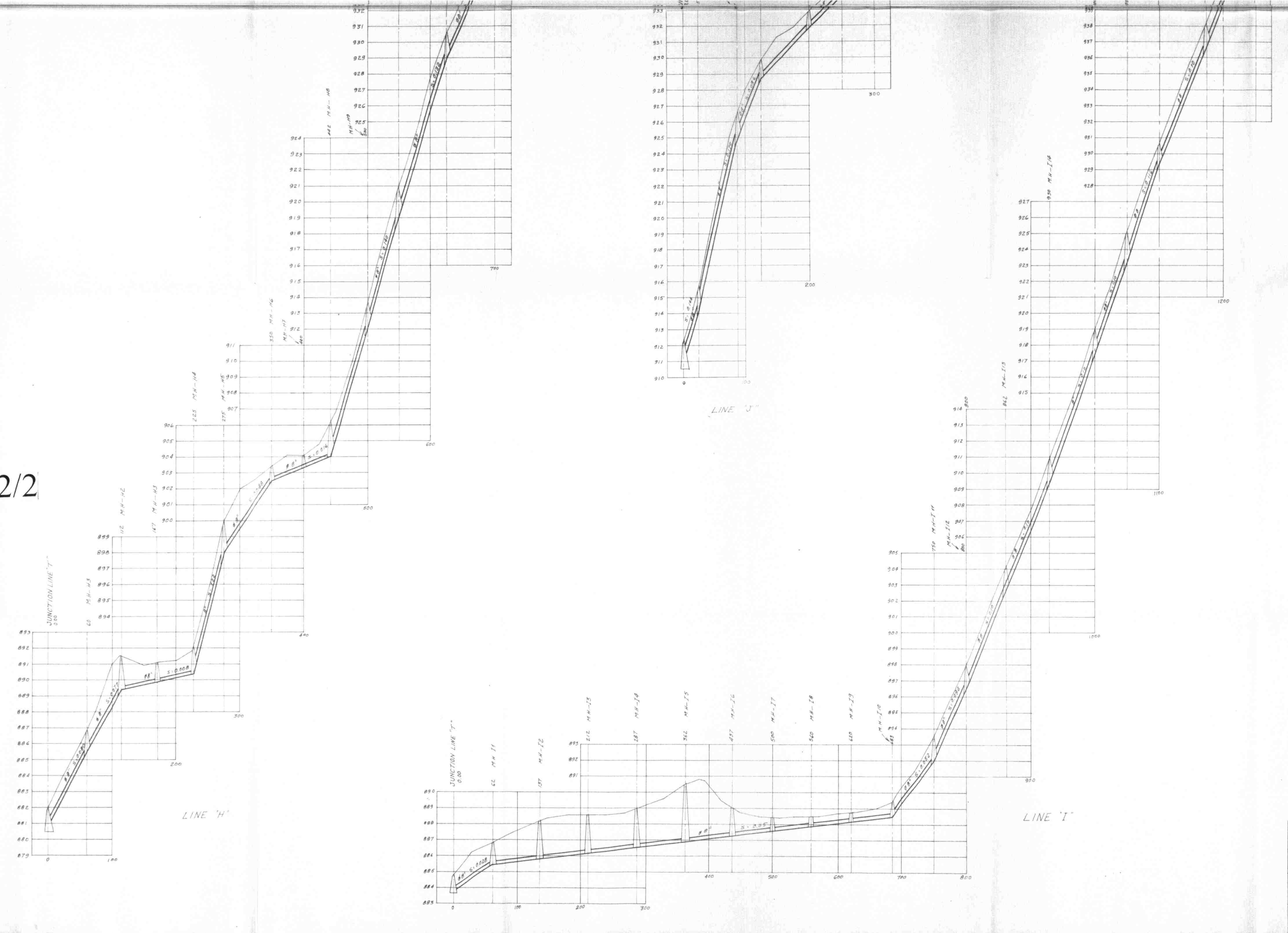
2/2



HEBRON SEWERAGE SCHEME		
L-SECTIONS FOR LINE F, G & K		
ENGINEER SAIYED HASAN AKHTAR ADVISOR PROFESSOR SAMIR-EL-KHURI	DRAWING NUMBER 7	
	SCALE HOR: 1/2500 VER: 1/100	DATE APRIL 1965

1/2

2/2



HEBRON SEWERAGE SCHEME

L-SECTIONS FOR LINE H, I & J

ENGINEER :
SAIYED HASAN AKHTAR

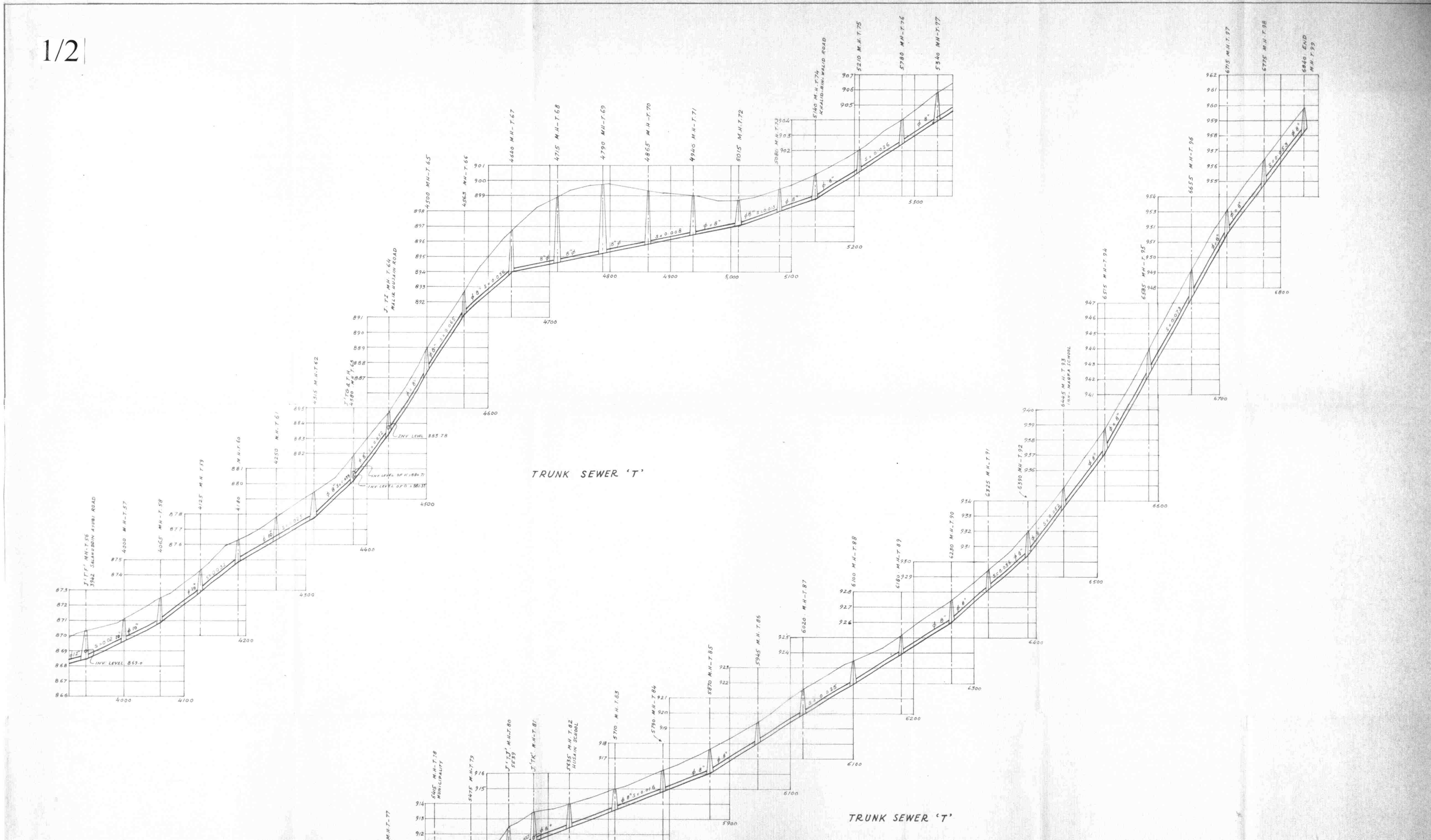
ADVISOR :
PROFESSOR SAMIR-EL-KHURI

DRAWING NUMBER
8

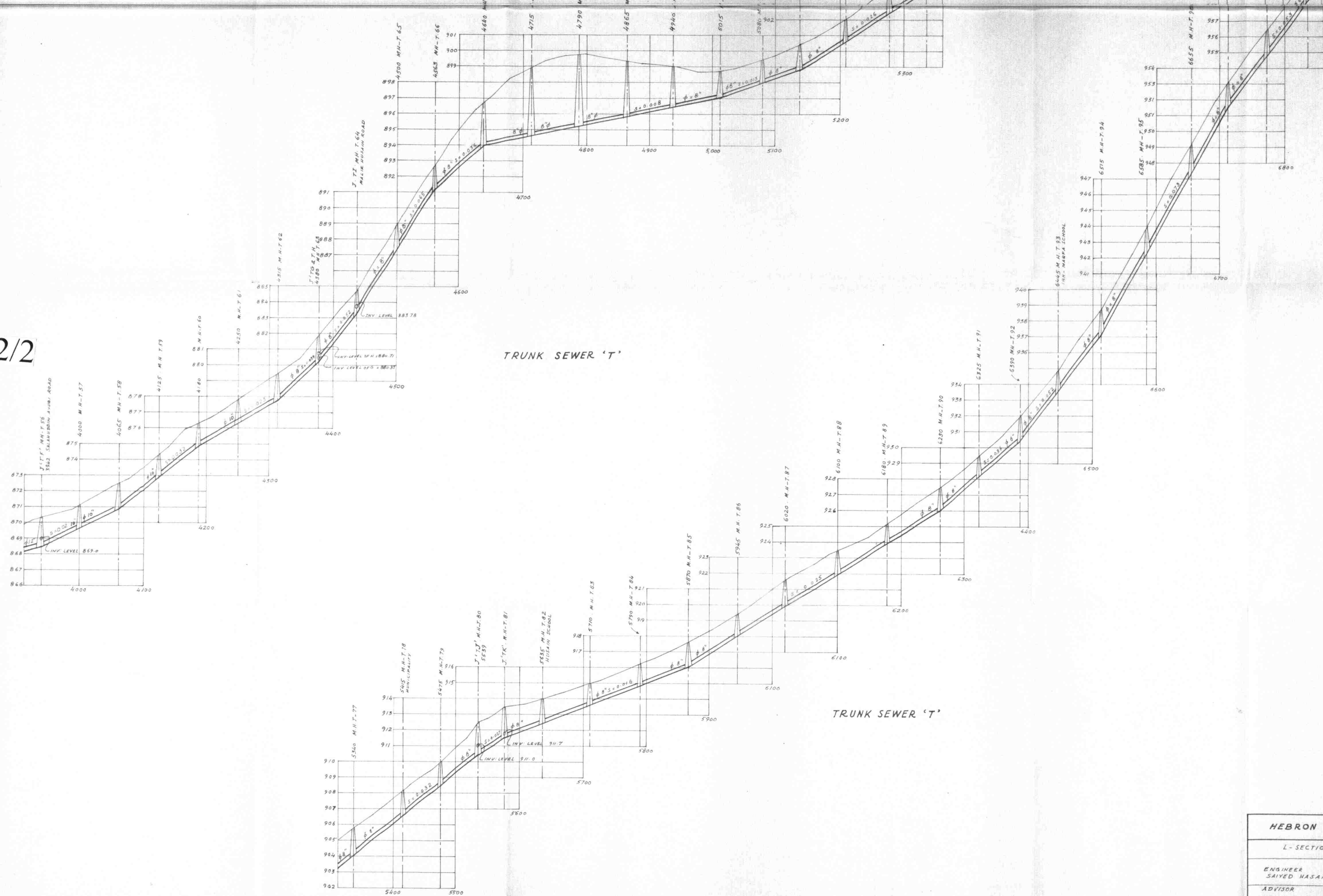
SCALE
HOR : 1/2500
VER : 1/100

DATE
APRIL 1965

1/2

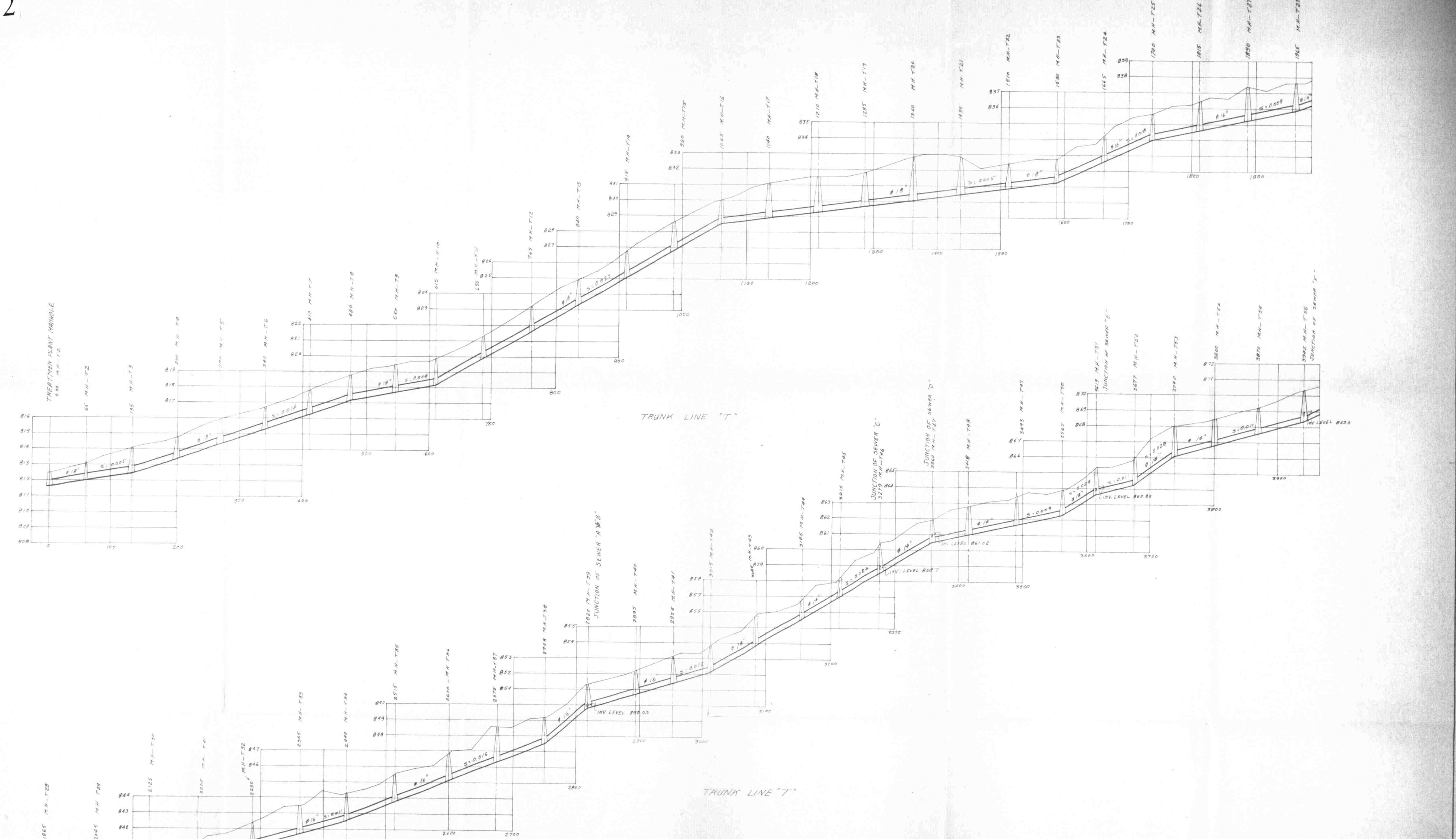


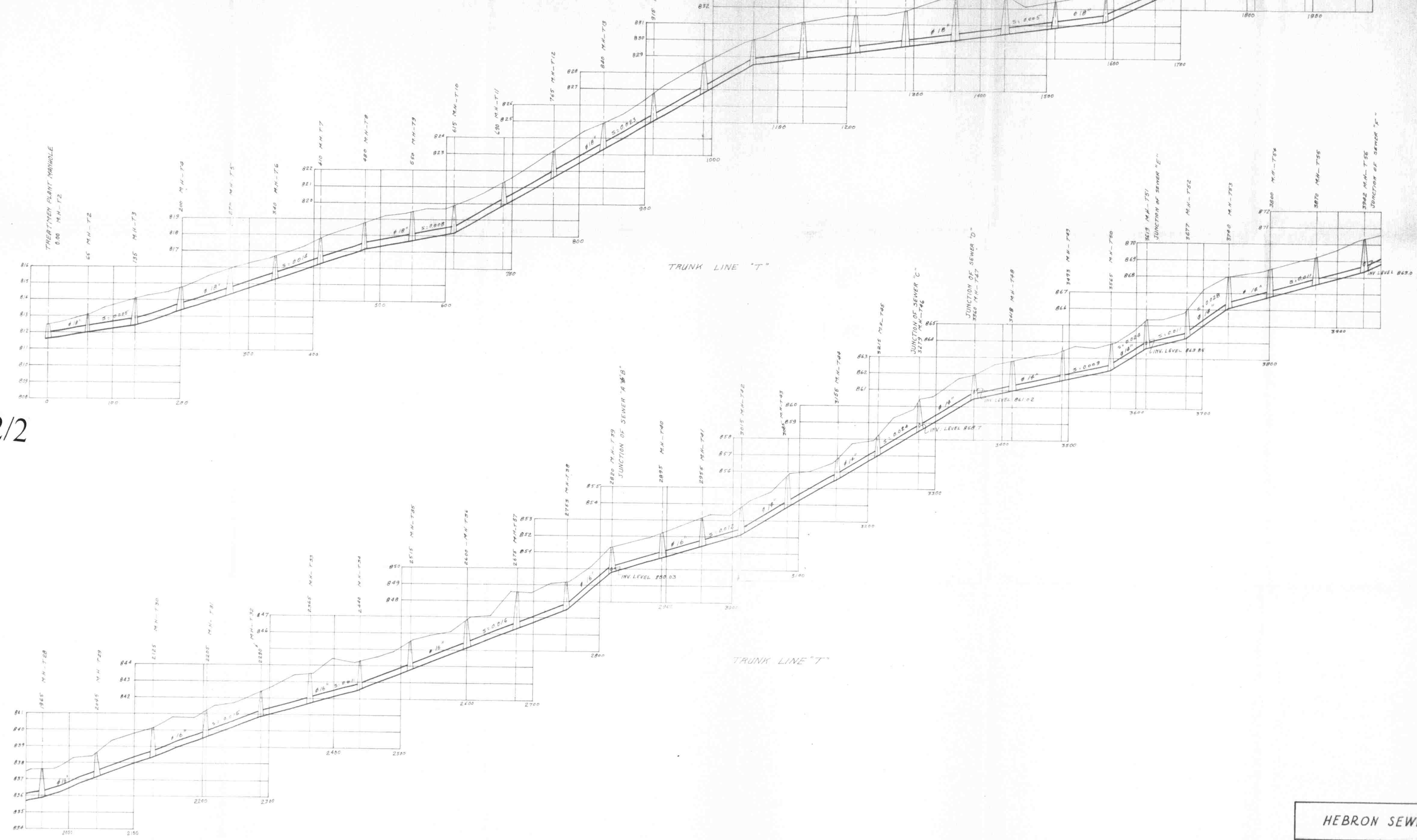
2/2



HEBRON SEWERAGE SCHEME	
L-SECTION OF TRUNK SEWER	
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 9
ADVISOR PROFESSOR SAMIR-EL-KHORI	SCALE HOR: 1/2500 VER: 1/100 · DATE APRIL 1964

1/2


TRUNK LINE "T"
TREATMENT PLANT MANHOLE
JUNCTION OF SEWER "A" & "B"
JUNCTION OF SEWER "C"
JUNCTION OF SEWER "D"
JUNCTION OF SEWER "E"
TRUNK LINE "T"



2/2

HEBRON SEWERAGE SCHEME

L-SECTION FOR TRUNK SEWER (CONTINUED)

ENGINEER:
SAIYED HASAN AKHTAR

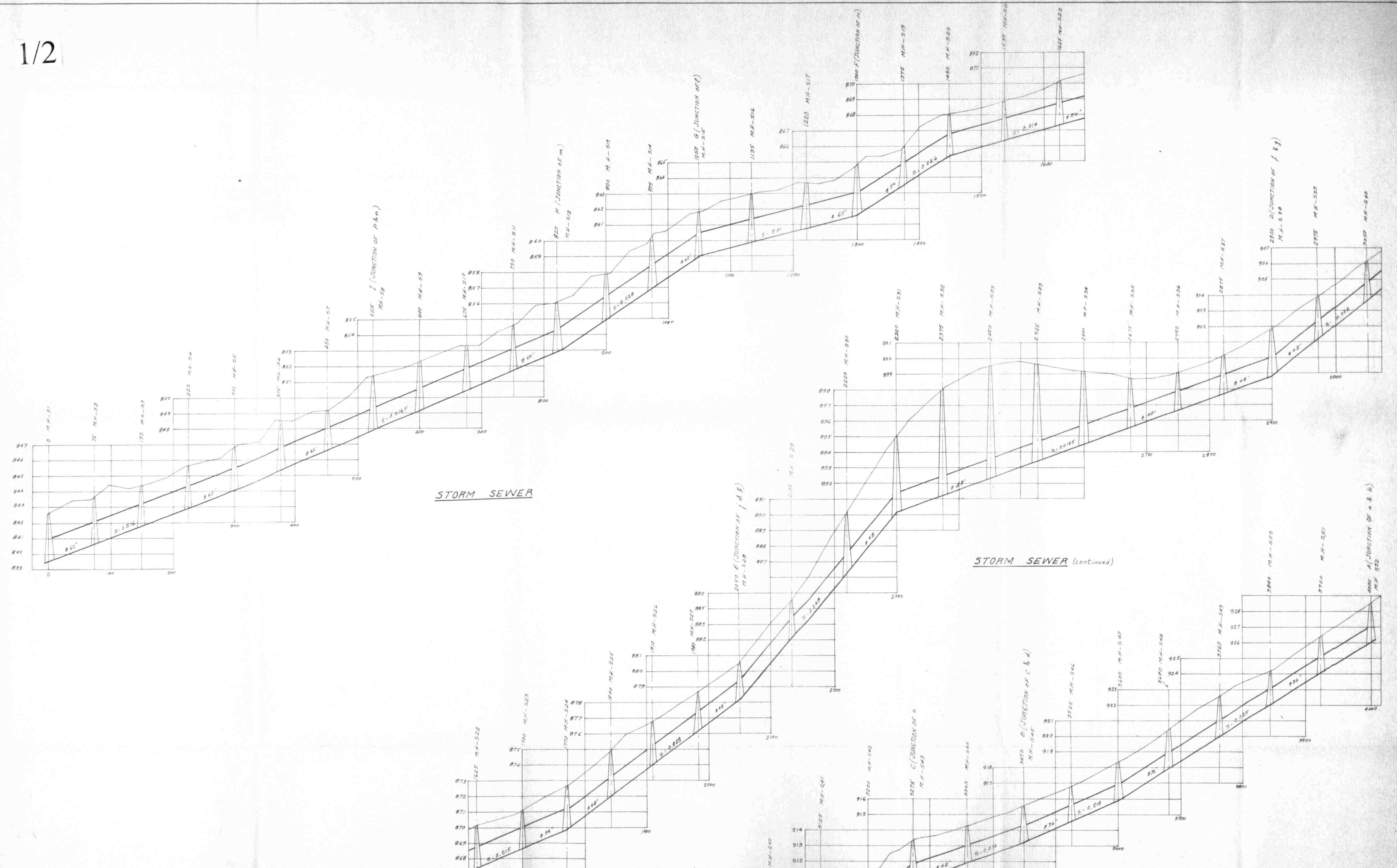
ADVISOR:
PROFESSOR SAMIR-EL-KHURI

DRAWING NUMBER: 10

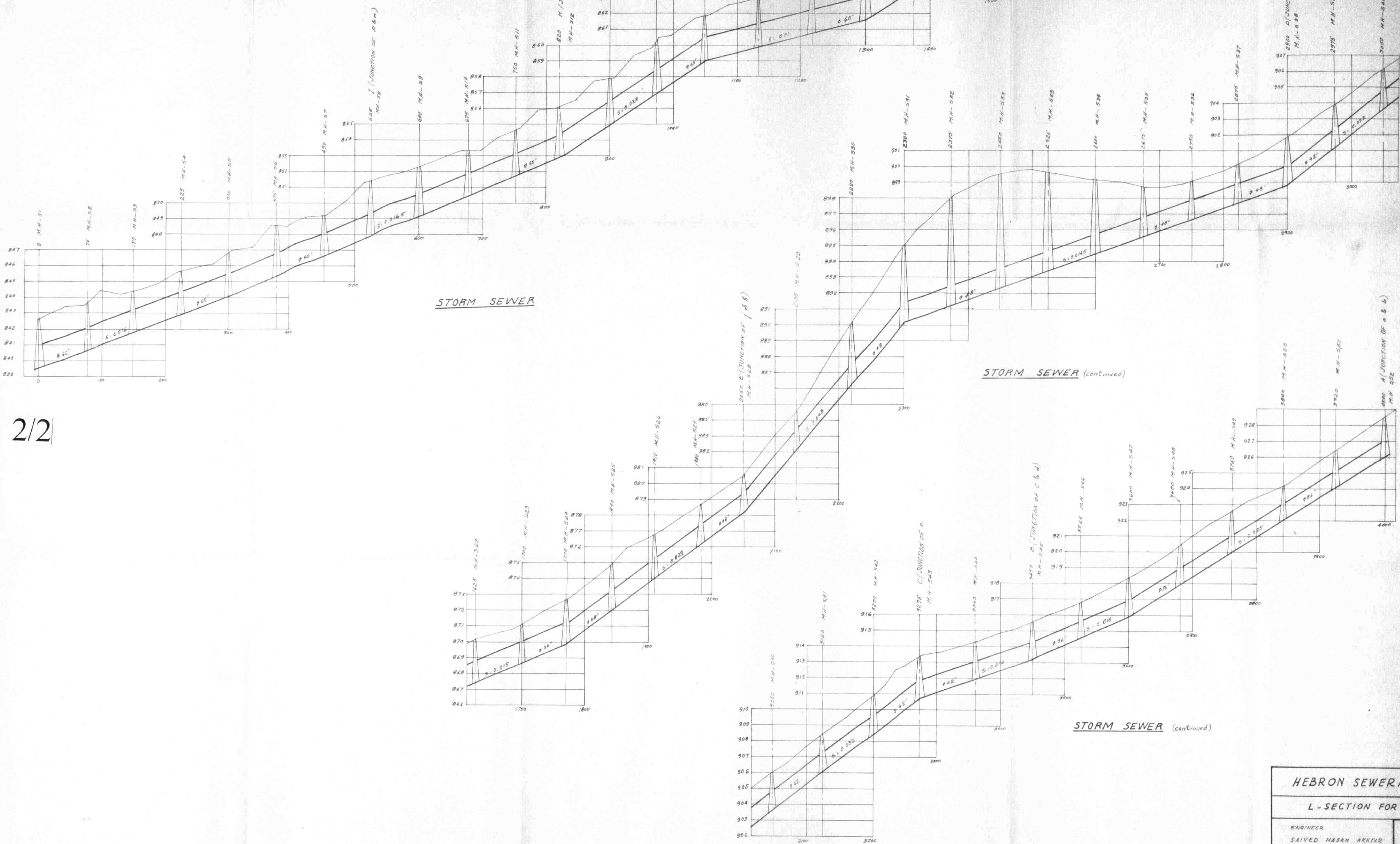
SCALE: HOR 1/2500, VER 1/100

DATE: APRIL 1965

1/2



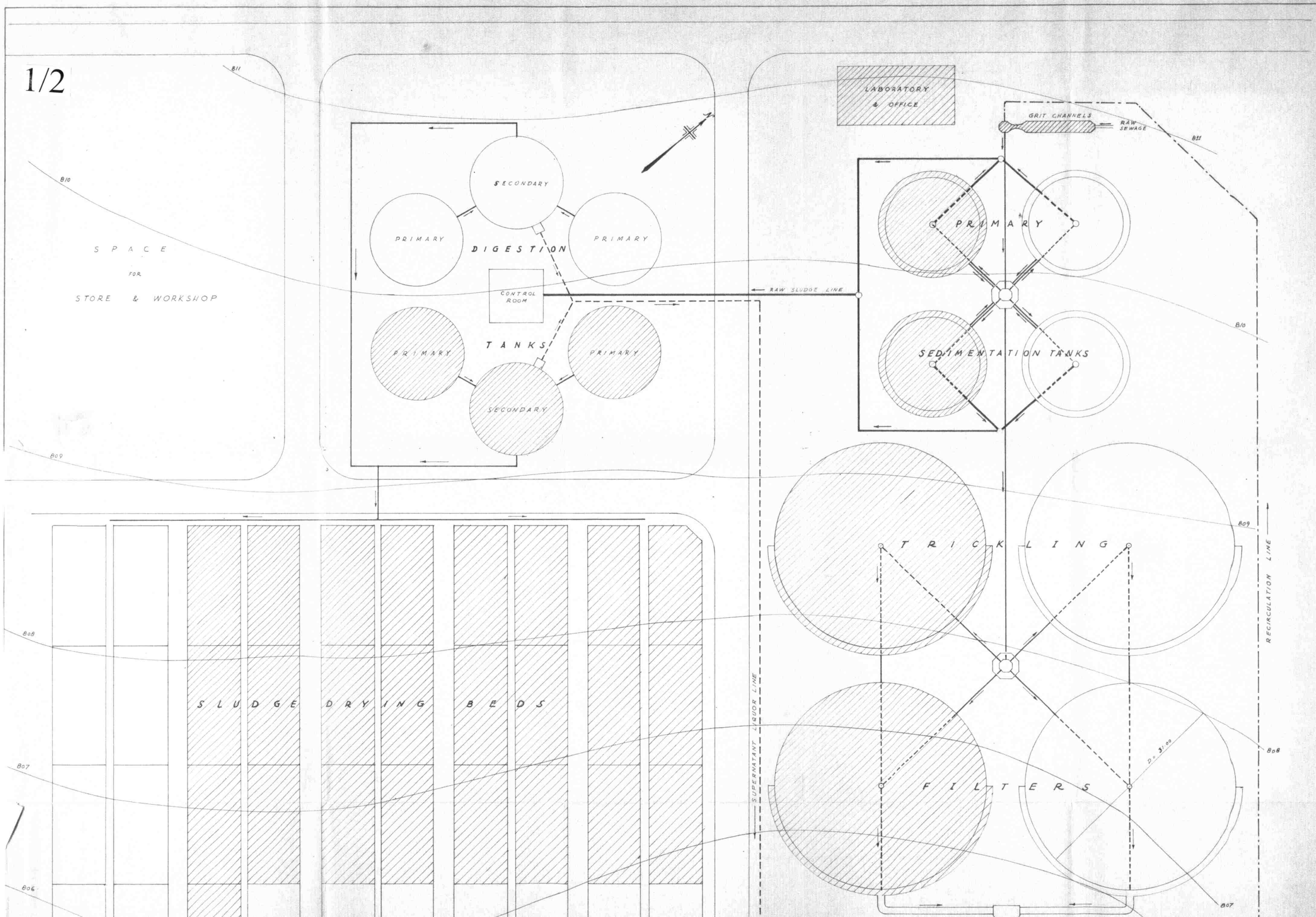
2/2

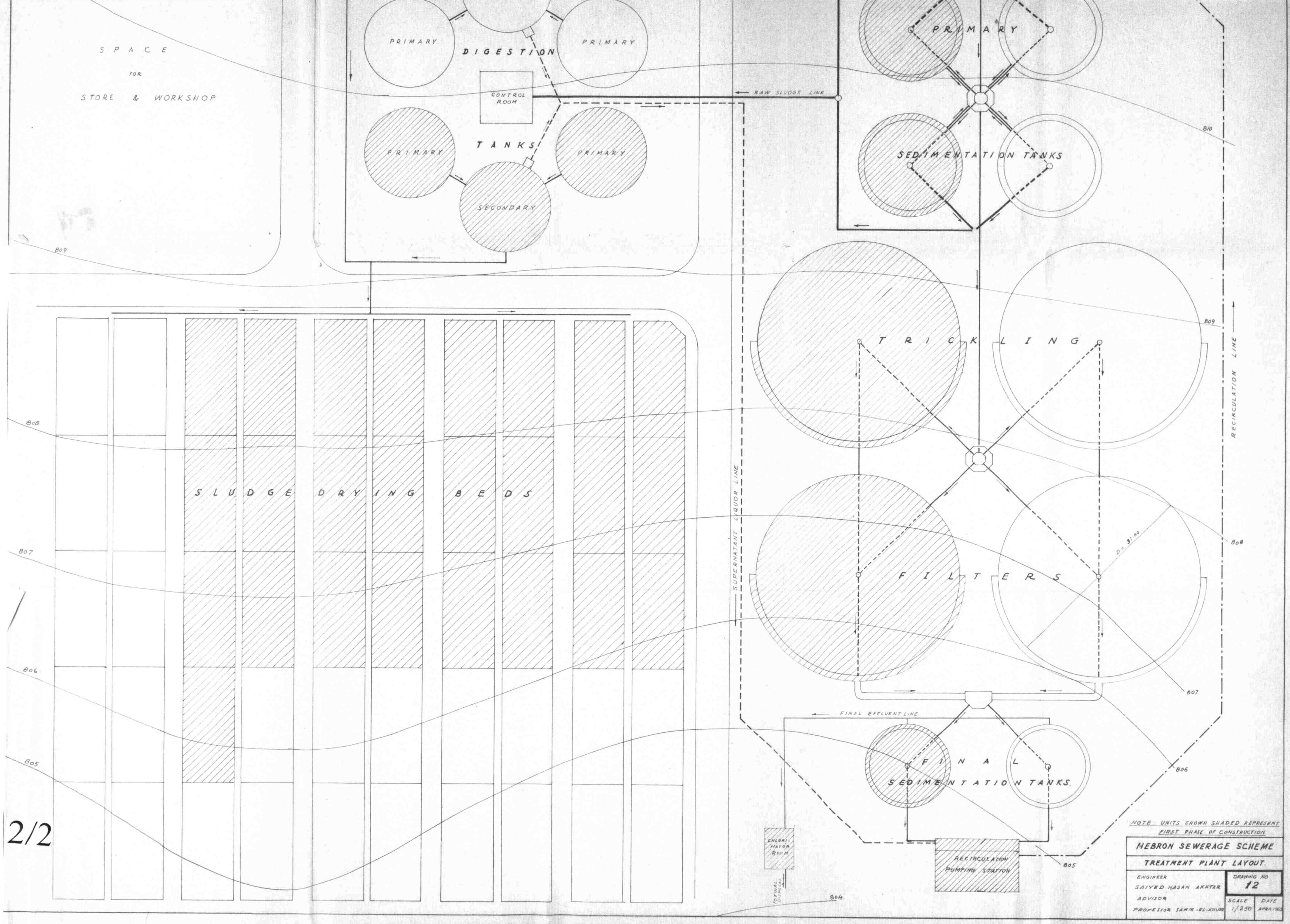


HEBRON SEWERAGE SCHEME

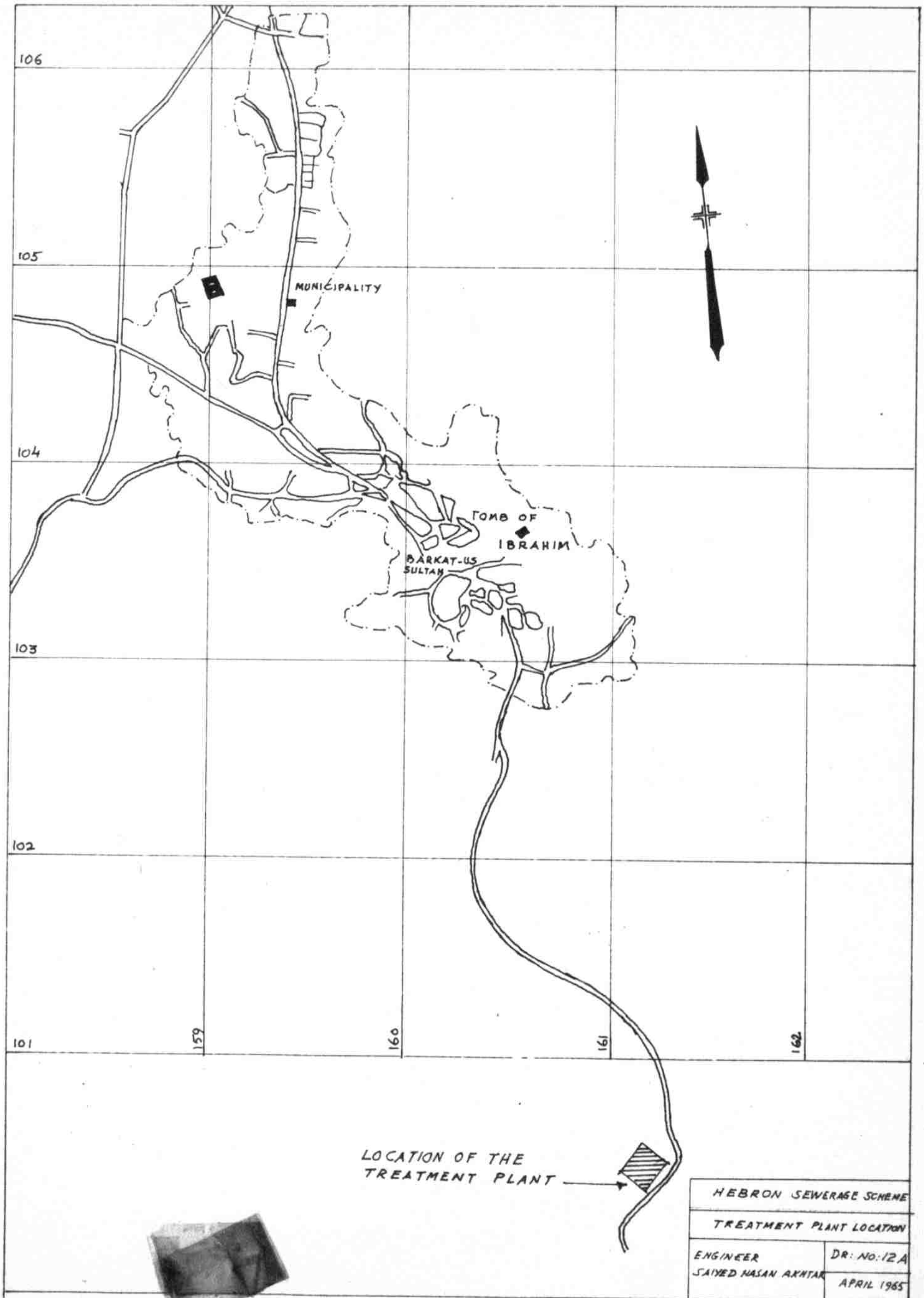
L-SECTION FOR STORM SEWER

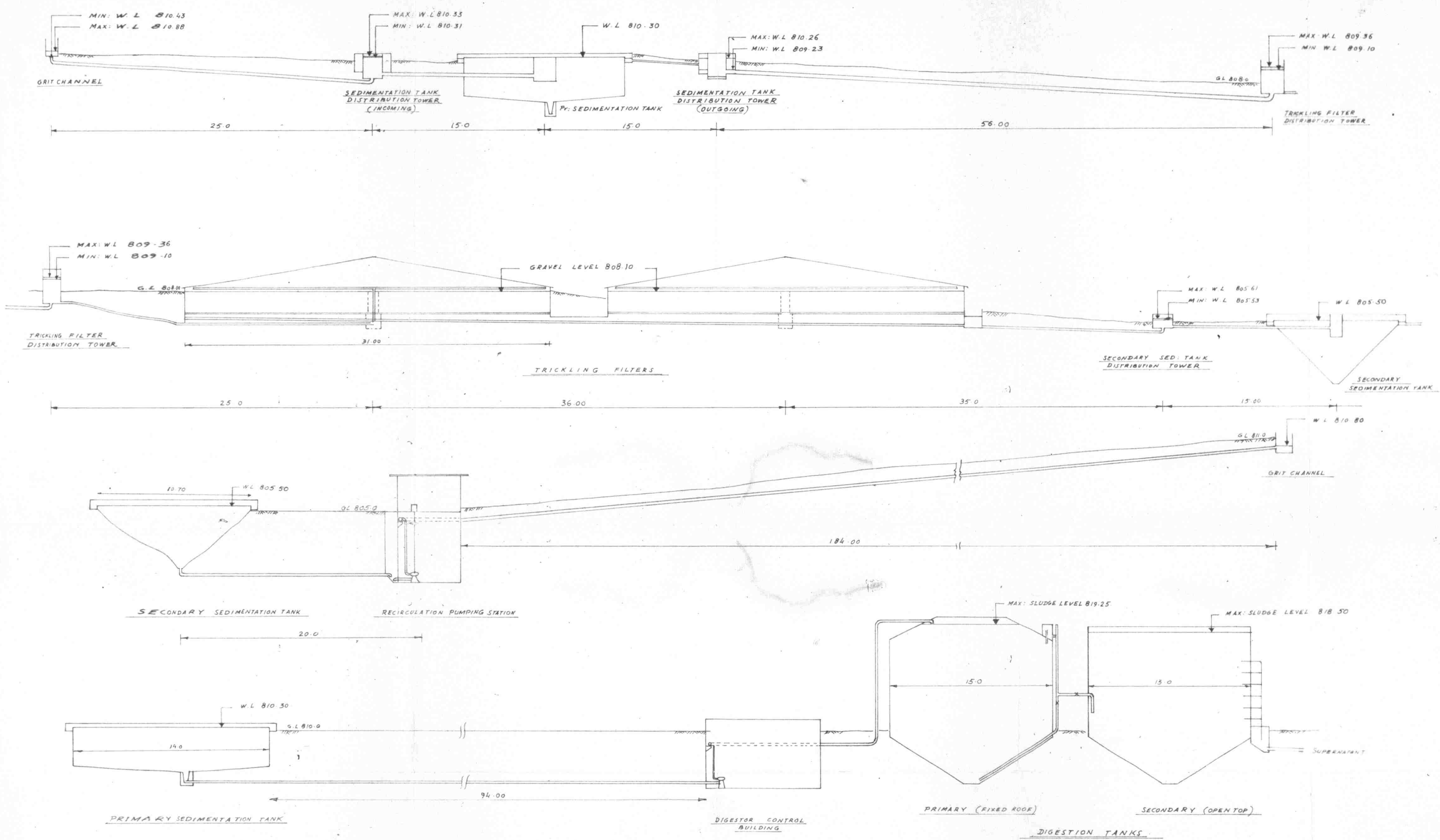
ENGINEER SAIYED HASAN AKHTAR ADVISOR PROFESSOR SAMIR-EL-KHURI	DRAWING NUMBER 11	
	SCALE HOR. 1/2500 VER 1/100	DATE APRIL 1965


1/2
SPACE
FOR
STORE & WORKSHOP
LABORATORY
& OFFICE
GRIT CHANNELS
RAW SEWAGE
SECONDARY
PRIMARY
DIGESTION
PRIMARY
CONTROL ROOM
PRIMARY
TANKS
PRIMARY
SECONDARY
RAW SLUDGE LINE
PRIMARY
SEDIMENTATION TANKS
TRICKLING
FILTERS
SLUDGE DRYING BEDS
SUPERNATANT LIQUOR LINE
RECIRCULATION LINE
811
810
809
808
807
806

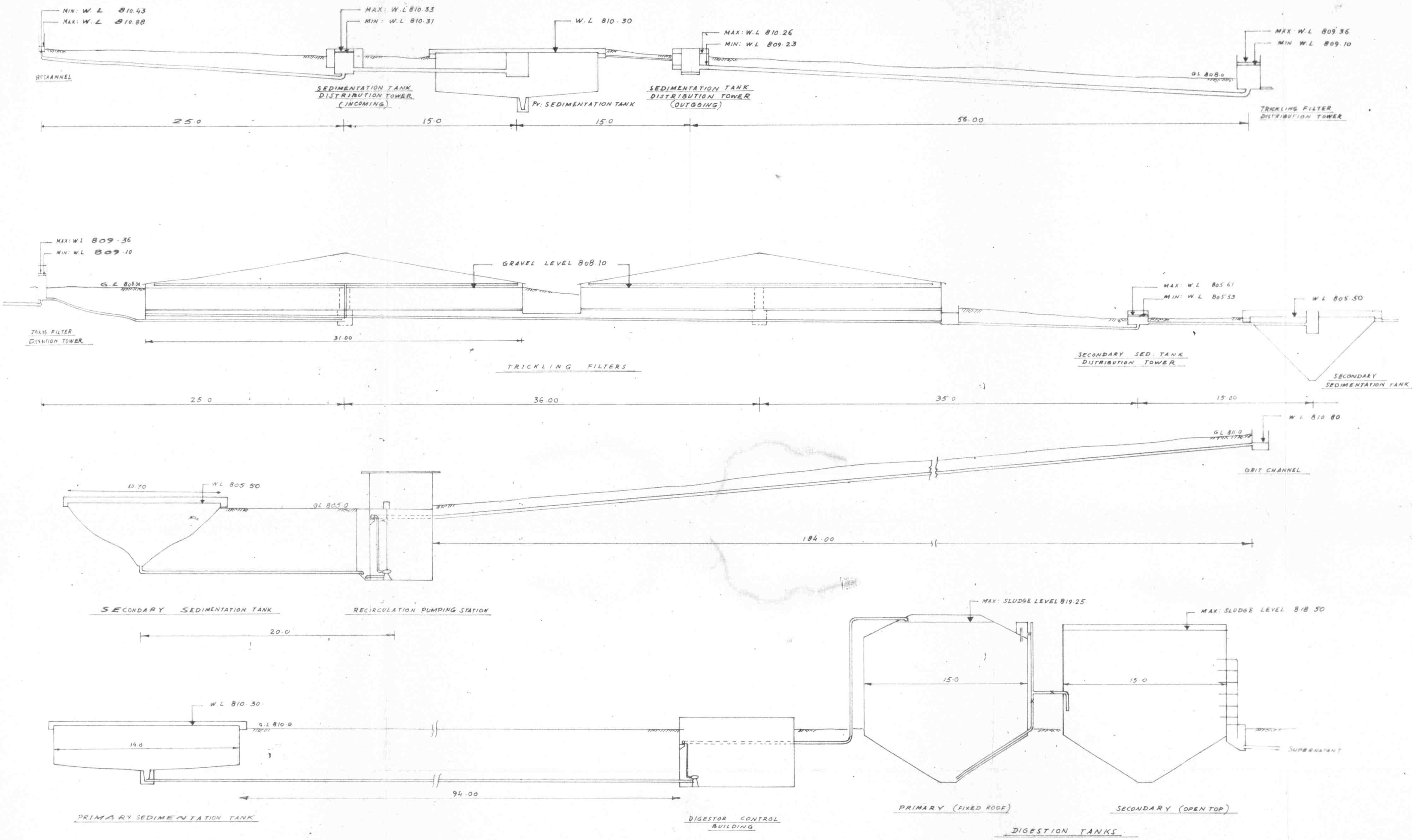

SPACE FOR STORE & WORKSHOP
PRIMARY
DIGESTION
PRIMARY
CONTROL ROOM
PRIMARY
TANKS
PRIMARY
SECONDARY
RAW SLUDGE LINE
PRIMARY
SEDIMENTATION TANKS
810
809
SLUDGE DRYING BEDS
808
807
806
805
804
SUPERNATANT LIQUOR LINE
TRICKLING
FILTERS
RECIRCULATION LINE
FINAL EFFLUENT LINE
FINAL SEDIMENTATION TANKS
CHLORINATOR ROOM
RECIRCULATION PUMPING STATION
NOTE: UNITS SHOWN SHADED REPRESENT FIRST PHASE OF CONSTRUCTION
HEBRON SEWERAGE SCHEME
TREATMENT PLANT LAYOUT.
ENGINEER
SAIYED HASAN AKHTAR
ADVISOR
PROFESSOR SAMIR-EL-KHURI
DRAWING NO
12
SCALE
1/250
DATE
APRIL 1965

2/2


106
105
104
103
102
101
159
160
161
162
MUNICIPALITY
TOMB OF
IBRAHIM
BARKAT-US
SULTAN
LOCATION OF THE
TREATMENT PLANT
HEBRON SEWERAGE SCHEME
TREATMENT PLANT LOCATION
ENGINEER
SAIYED HASAN AKHTAR
DR: NO: 12 A
APRIL 1965

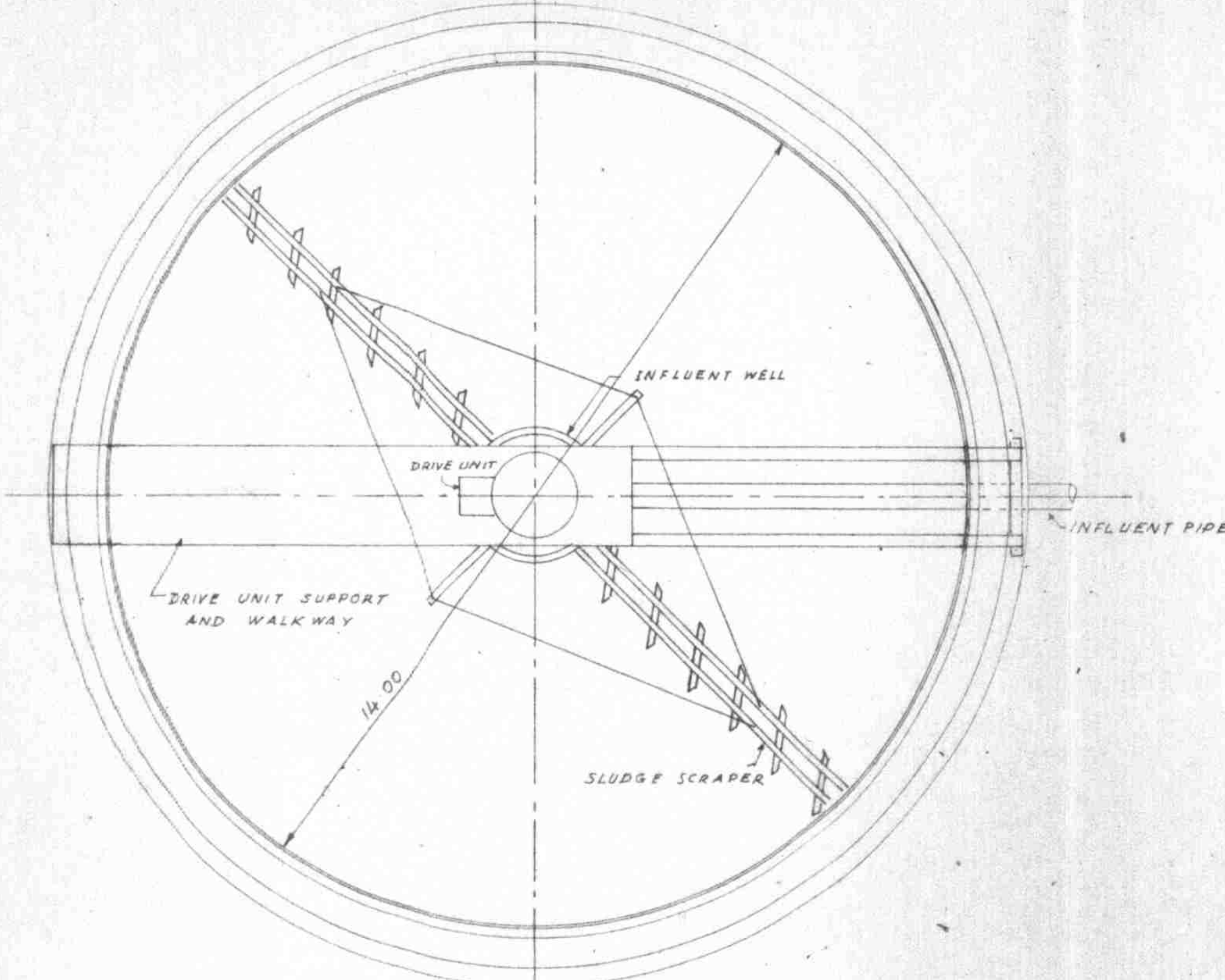


HEBRON SEWERAGE SCHEME		
HYDRAULIC PROFILE		
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 13	
ADVISOR PROFESSOR - SAMIR - EL - KHURI	SCALE 1/200	DATE APRIL 1965

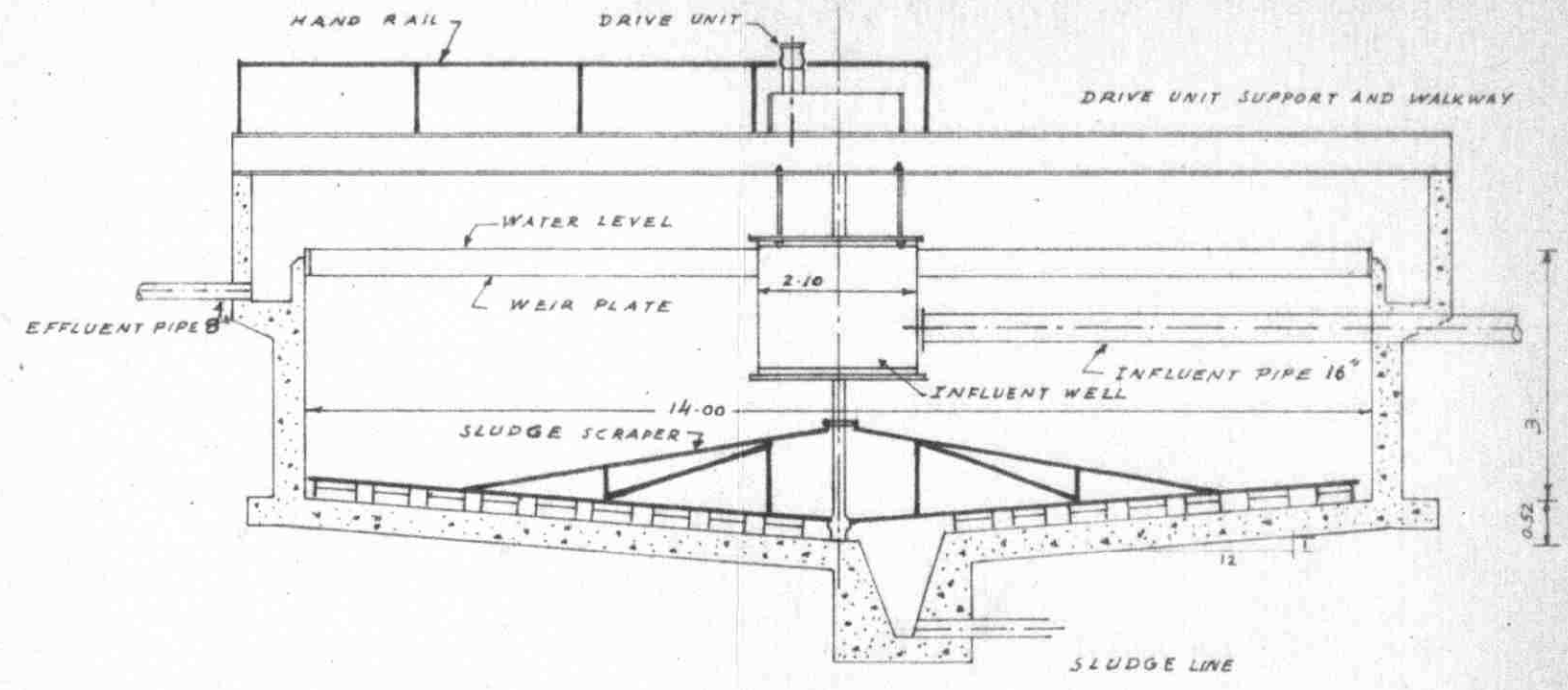


HEBRON SEWERAGE SCHEME	
HYDRAULIC PROFILE	
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 13
ADVISOR PROFESSOR - SAMIR - EL - KHURI	SCALE 1/200 | DATE APRIL 1965

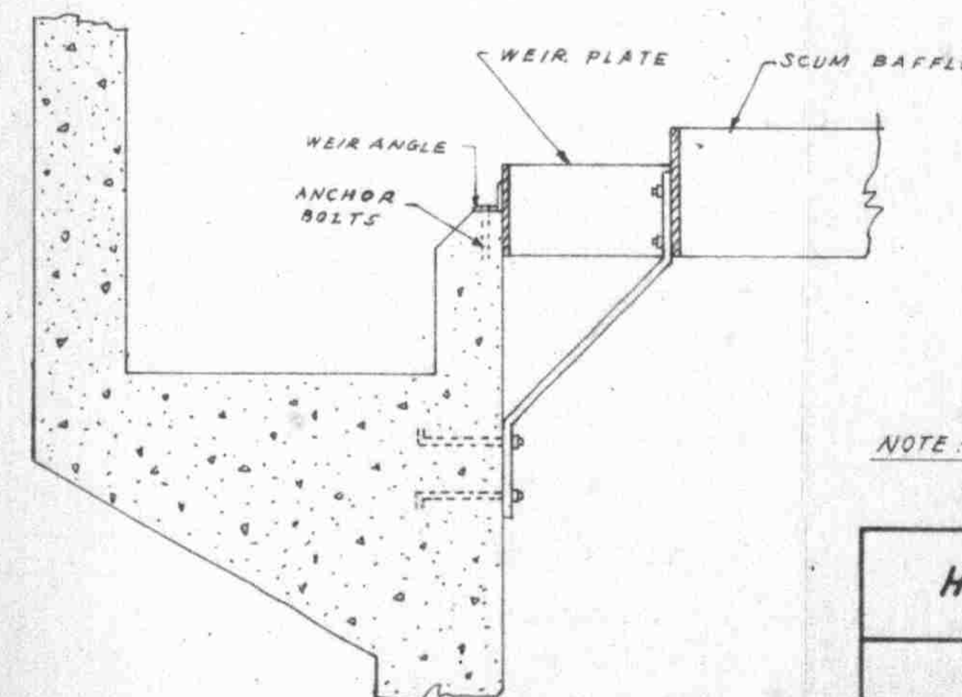

PLAN
SCREENS 1cm x 4cm SPACED WITH CLEAR DISTANCE OF 2.5 cm
PERFORATED METAL PLATFORM FOR SCREENINGS
PLATFORM FOR STANDING FOR MAN
PARTIAL FLUME WITH 6" or 15.25 cm THROAT
SECTION B-B
SECTION A-A
MAX. WATER LEVEL
R.L 812.40
R.L 811.65
R.L 811.31
MAX. W.L 810.88
R.L 810.68
NOTE: ALL DIMENSIONS SHOWN IN CENTIMETERS UNLESS OTHERWISE INDICATED.
HEBRON SEWERAGE SCHEME
SCREENING & GRIT CHAMBER
ENGINEER: SAIYED HASAN AKHTAR
ADVISOR: PROFESSOR SAMIR-EL-KHURI
DRAWING NO. 14
SCALE 1/25
DATE APRIL 1965



PLAN



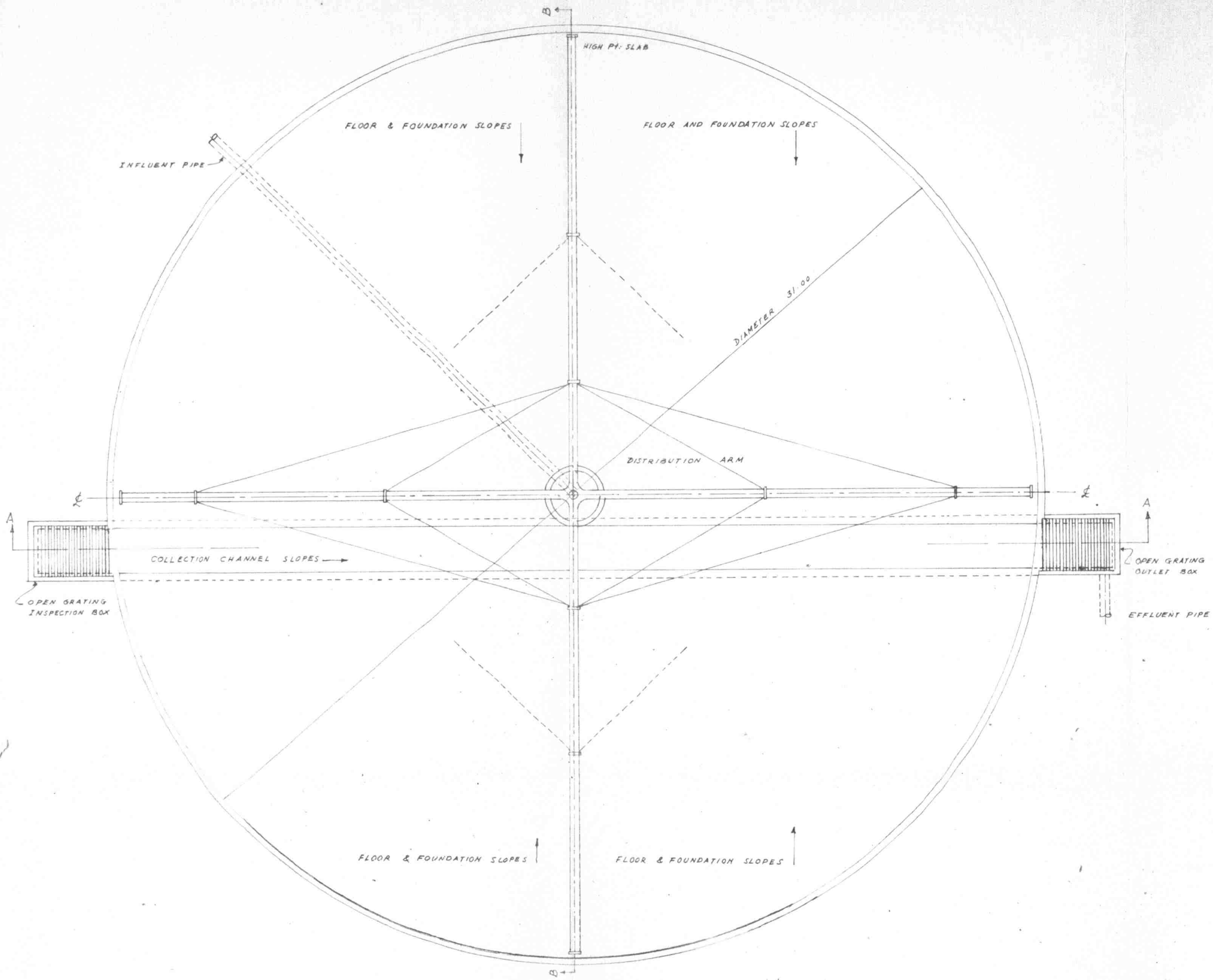
SECTIONAL ELEVATION



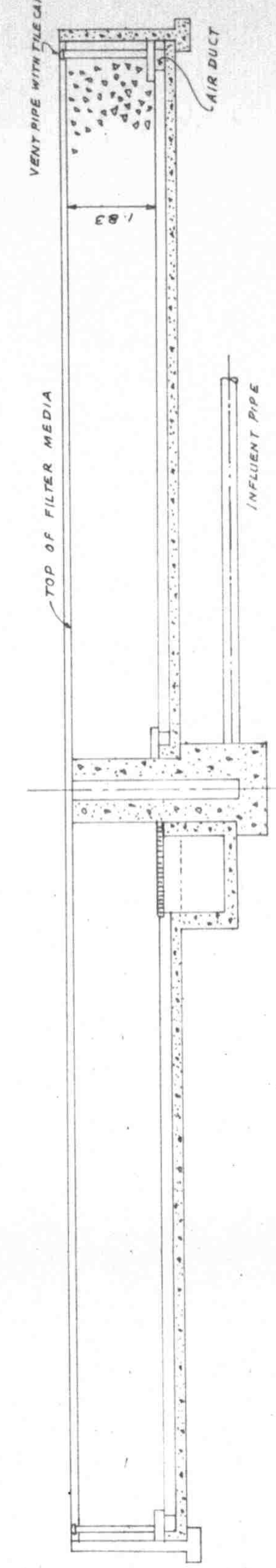
WEIR AND SCUM BAFFLE DETAIL

NOTE: ALL DIMENSIONS SHOWN IN METERS, UNLESS OTHERWISE INDICATED.

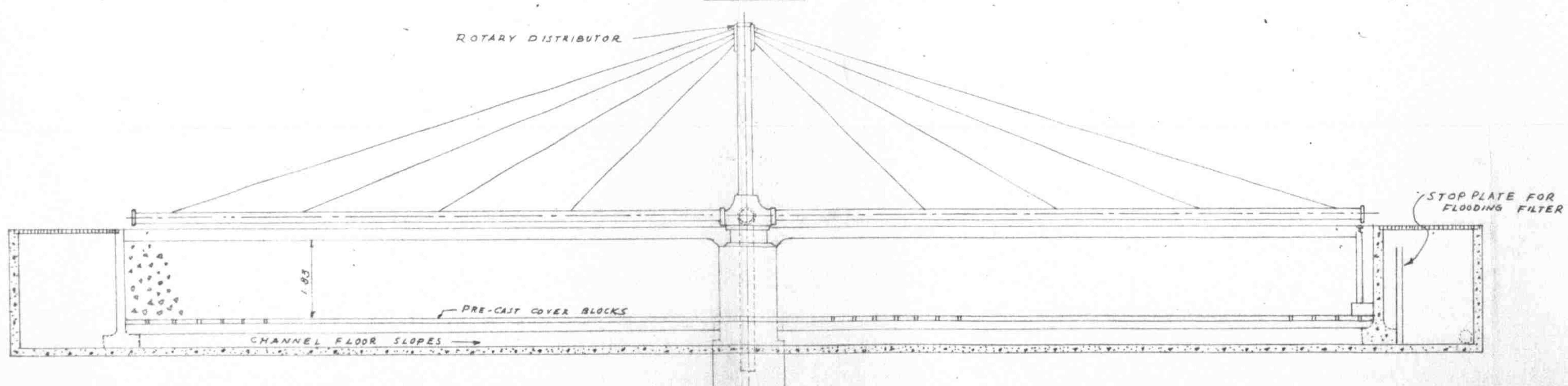
HEBRON SEWERAGE SCHEME		
PRIMARY SEDIMENTATION TANK		
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 15	
ADVISOR PROFESSOR SAMIR- EL- KHURI	SCALE 1/100	DATE APRIL 1965



PLAN



SECTION B-B

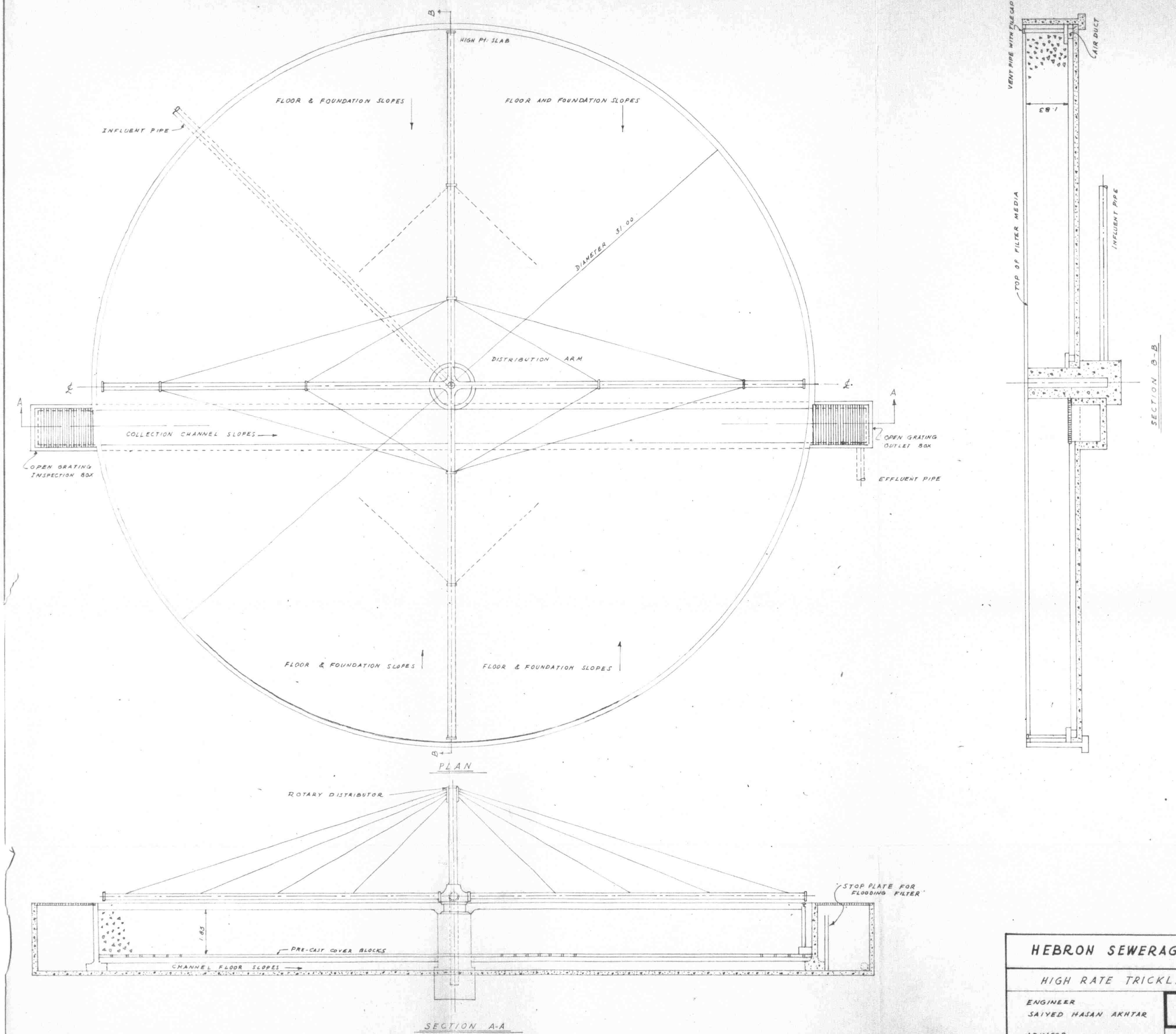


SECTION A-A

HEBRON SEWERAGE SCHEME

HIGH RATE TRICKLING FILTER

ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 16	
ADVISOR PROFESSOR SAMIR-EL-KHURI	SCALE 1/100	DATE APRIL 1965



HEBRON SEWERAGE SCHEME

HIGH RATE TRICKLING FILTER

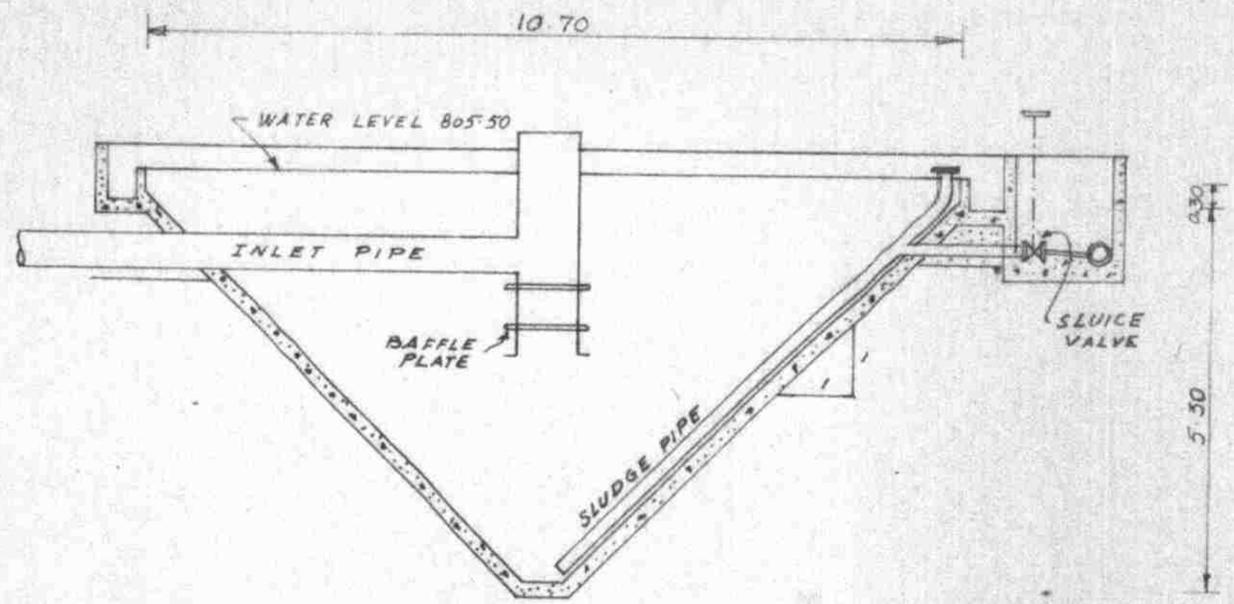
ENGINEER
SAIYED HASAN AKHTAR

DRAWING NUMBER
16

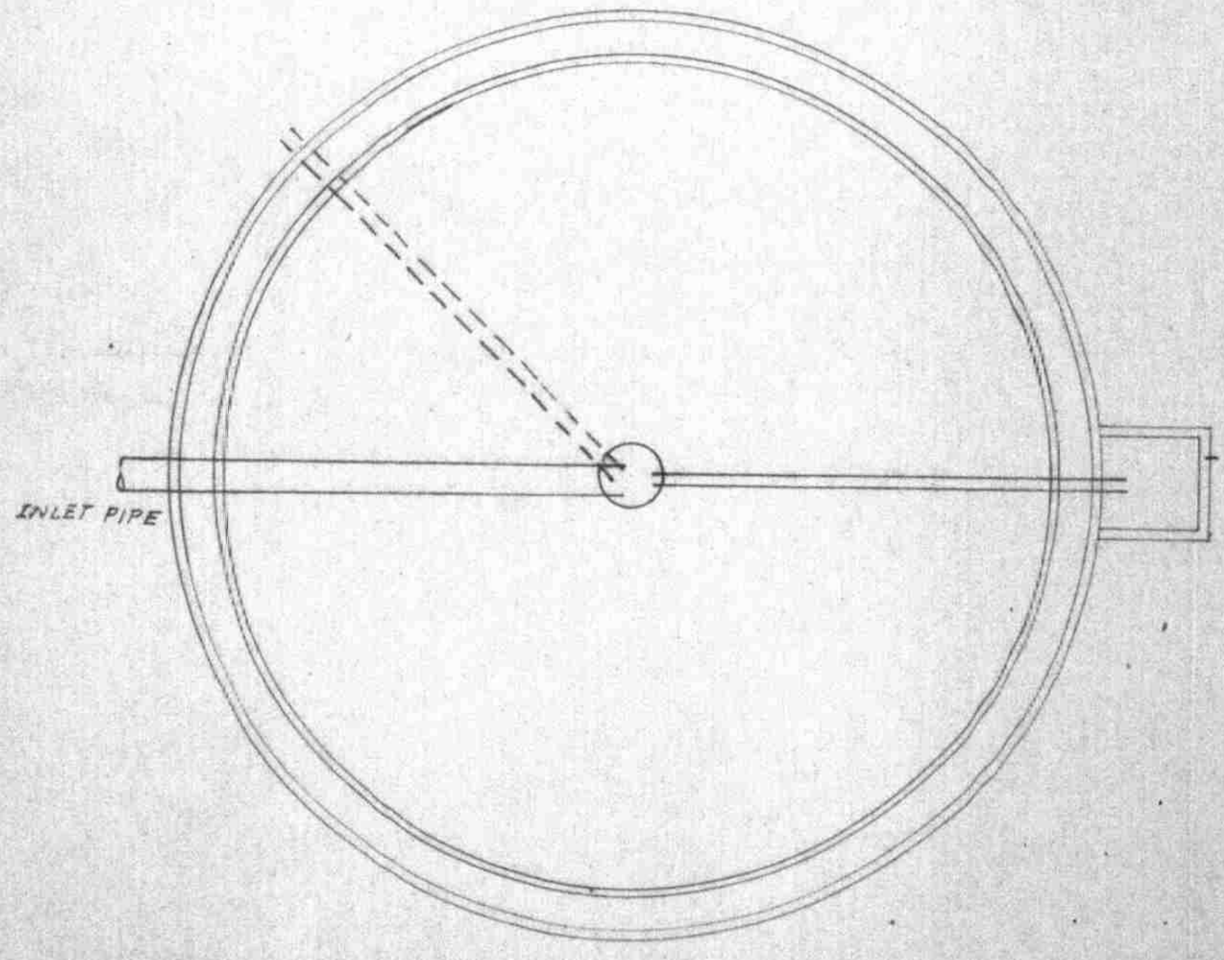
ADVISOR
PROFESSOR SAMIR-EL-KHURI

SCALE
1/100

DATE
APRIL 1965



SECTIONAL ELEVATION

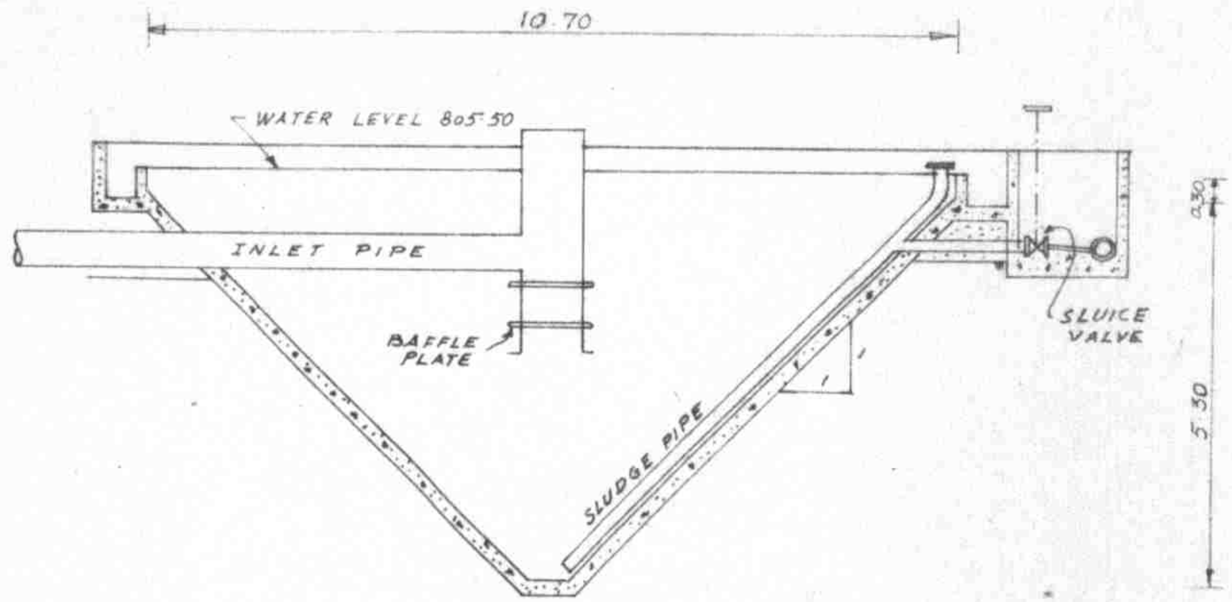


PLAN

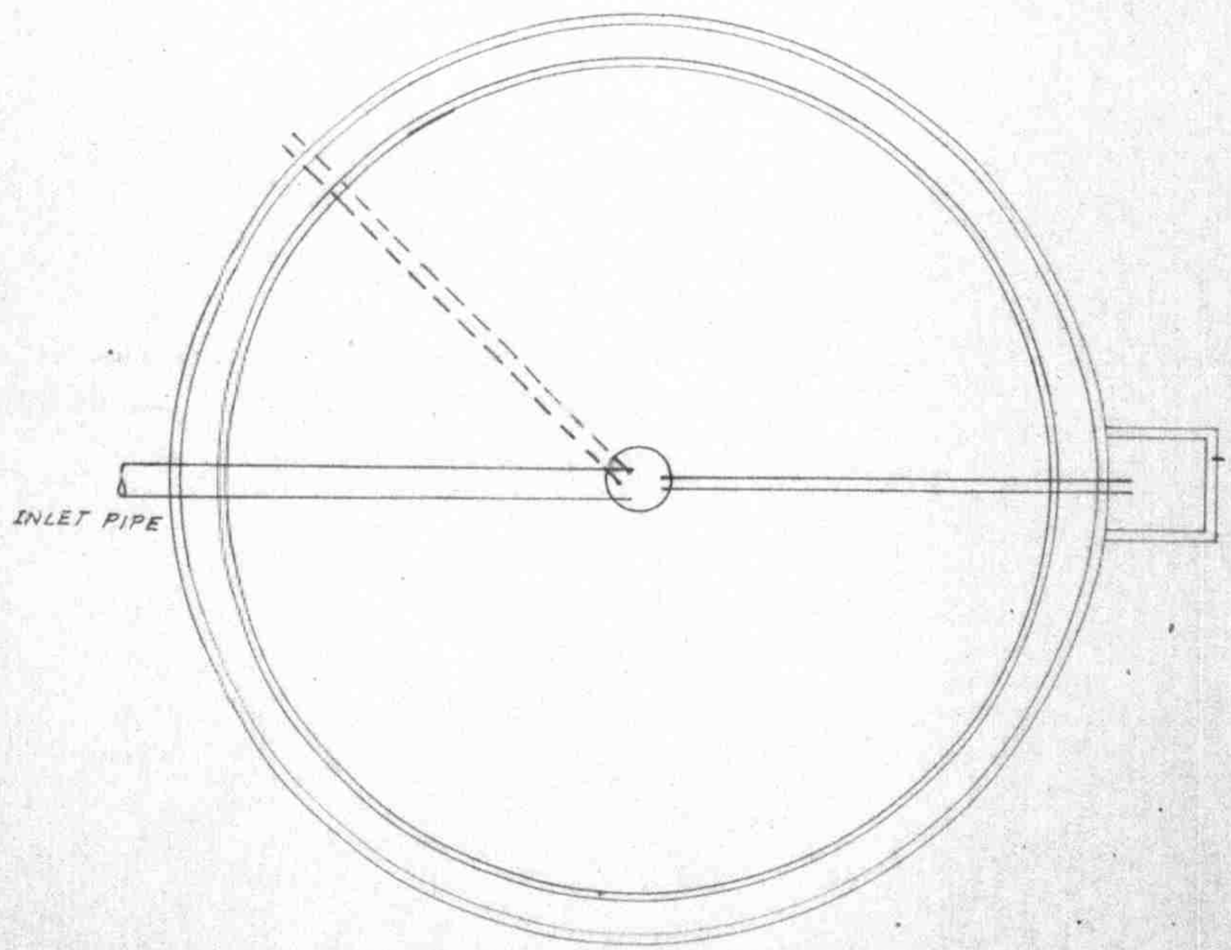
HEBRON SEWERAGE SCHEME

FINAL SEDIMENTATION TANK

ENGINEER SAIYED HASAN AKHTAR ADVISOR: PROFESSOR SAMIR-EL-KHURI	DRAWING NUMBER 17	
	SCALE 1/100	DATE APRIL 1965



SECTIONAL ELEVATION

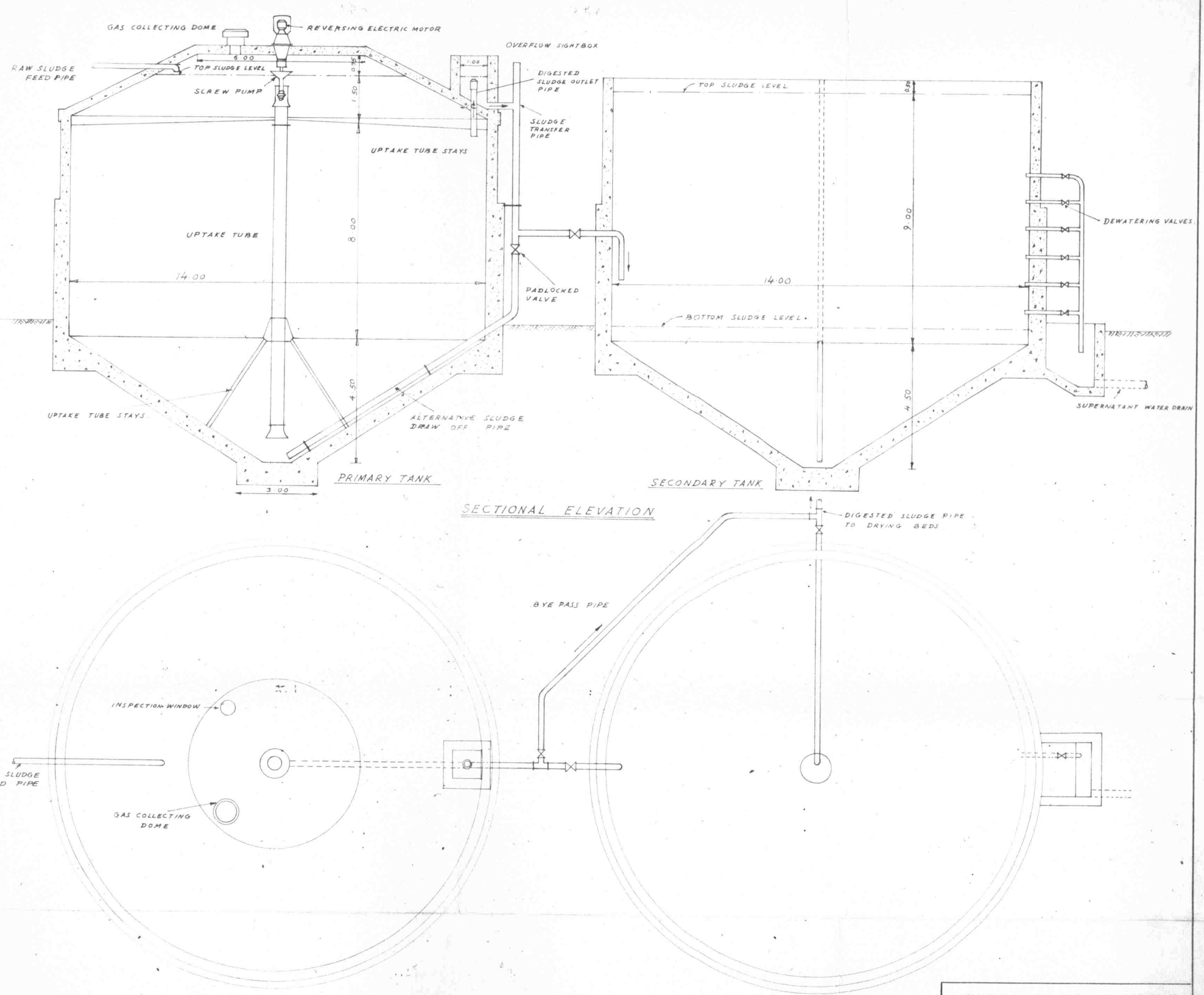


PLAN

HEBRON SEWERAGE SCHEME

FINAL SEDIMENTATION TANK

ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 17	
ADVISOR: PROFESSOR SAMIR-EL-KHURI	SCALE 1/100	DATE APRIL 1965

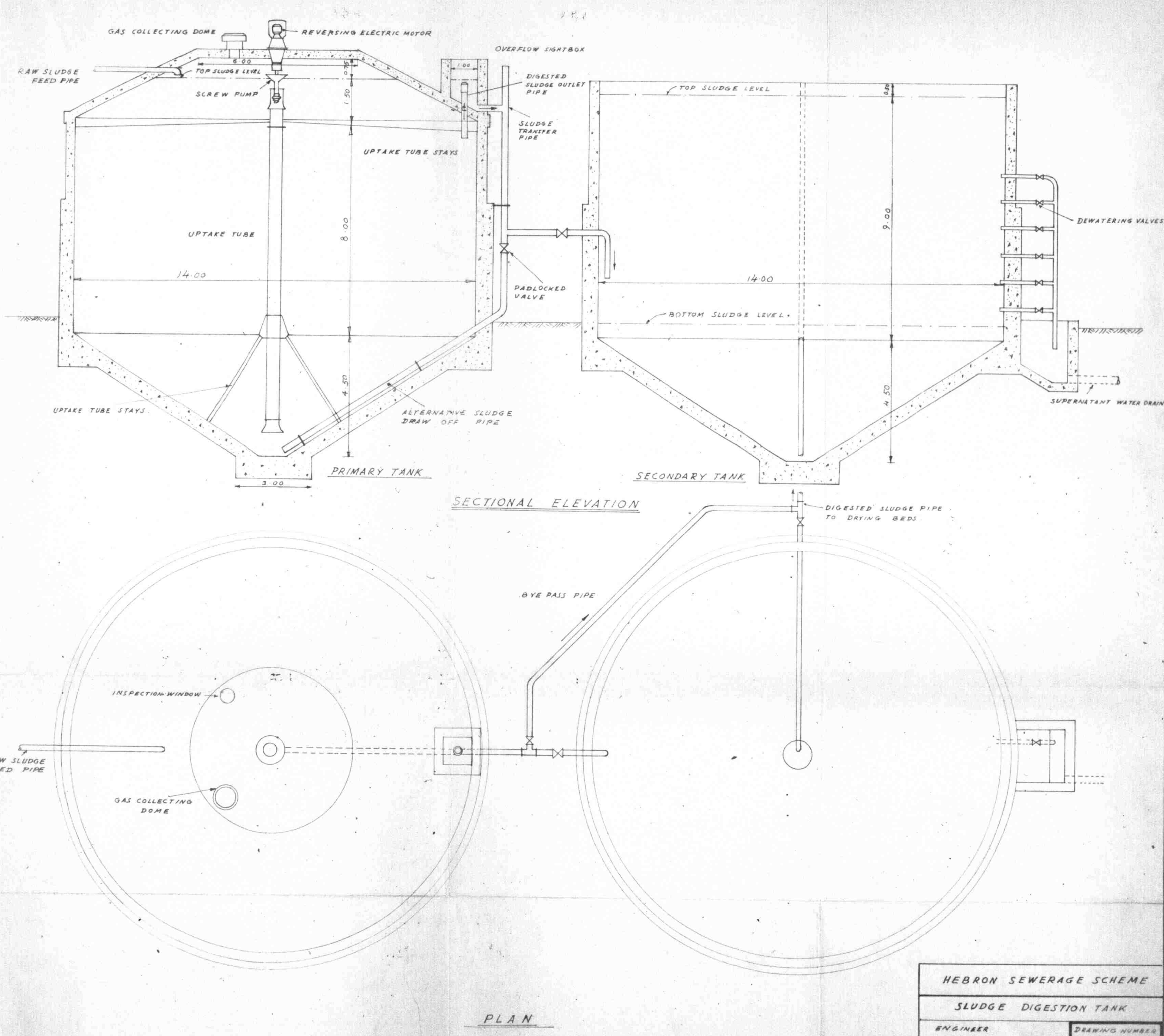


PLAN

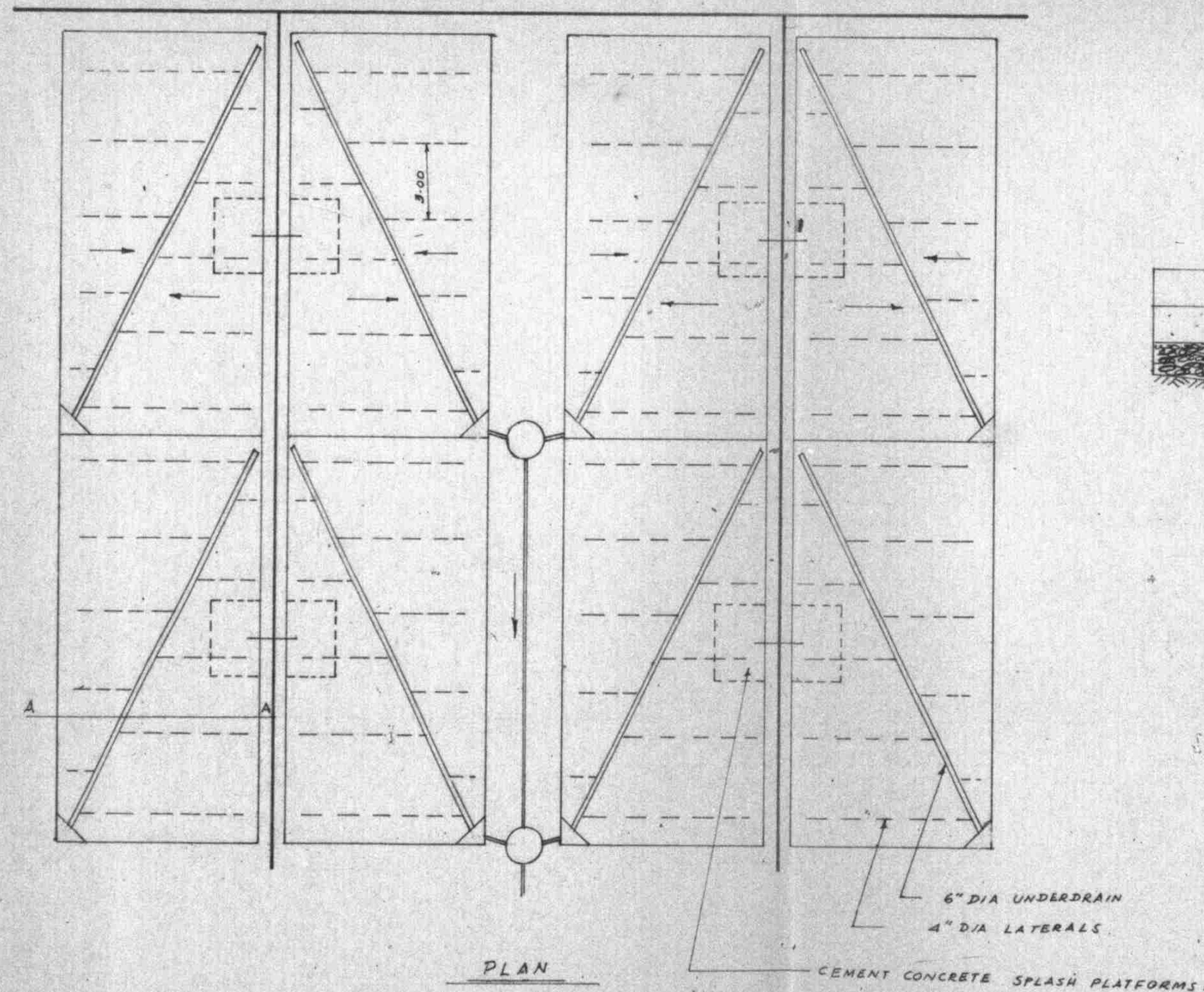
HEBRON SEWERAGE SCHEME

SLUDGE DIGESTION TANK

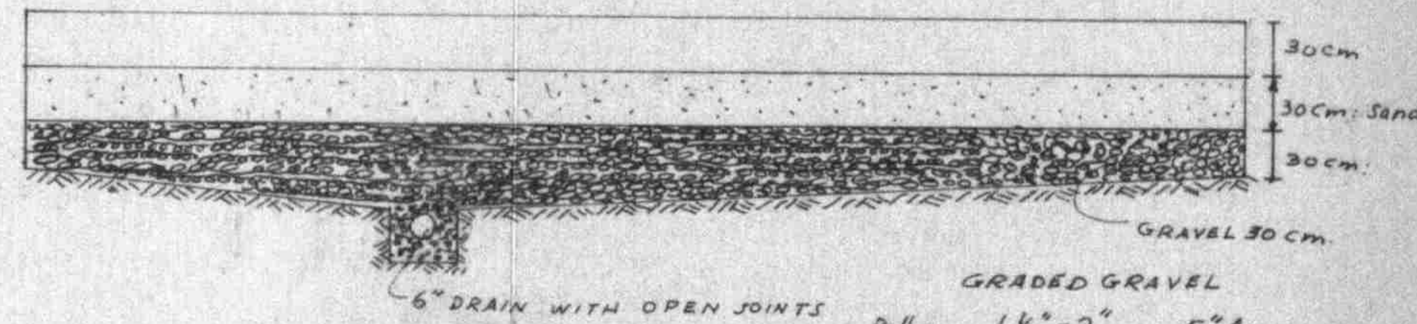
ENGINEER	DRAWING NUMBER	
SAIYED HASAN AKHTAR	18	
ADVISOR	SCALE	DATE
PROFESSOR SAMIR-EL-KHURI	1/100	APRIL 1965



HEBRON SEWERAGE SCHEME		
SLUDGE DIGESTION TANK		
ENGINEER SAIYED HASAN AKHTAR ADVISOR PROFESSOR SAMIR-EL-KHURI	DRAWING NUMBER 18	
	SCALE 1/100	DATE APRIL 1965



PLAN
SCALE 1/200

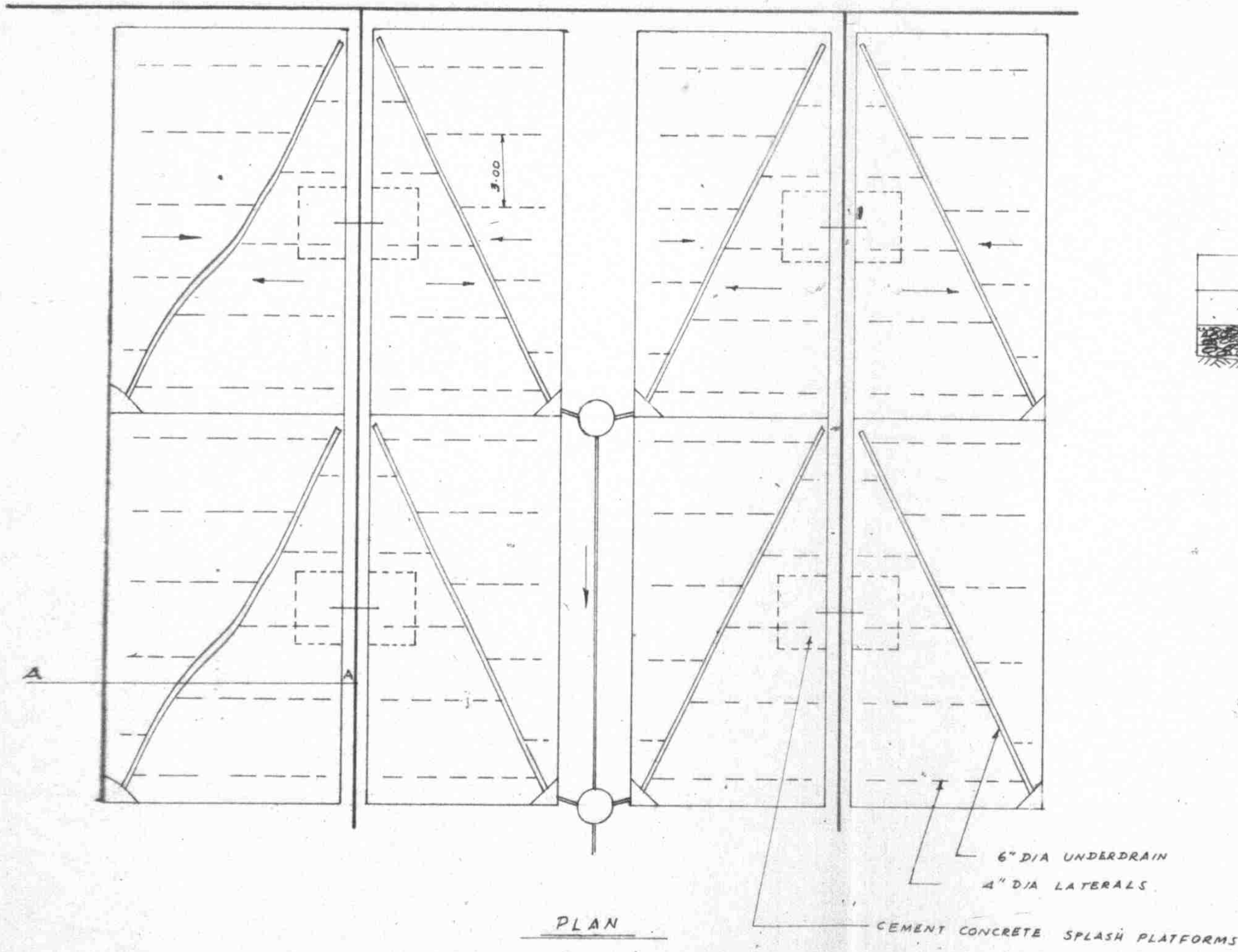


GRADED GRAVEL

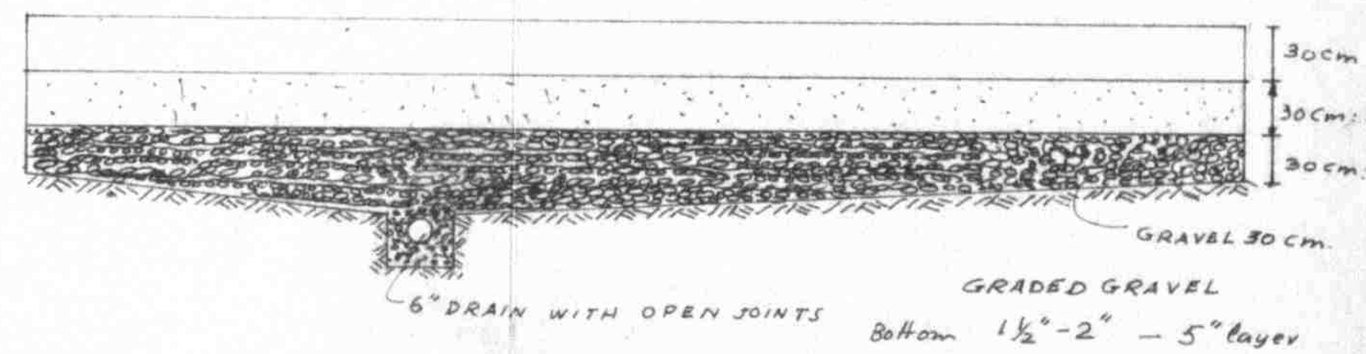
Bottom	1½"-2"	– 5" layer
Middle	1"	– 3" layer
Top	½"	– 4" "
	TOTAL = 12"	

SECTION AT AA
SCALE 1/25

HEBRON SEWERAGE SCHEME	
SLUDGE DRYING BEDS	
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 19
ADVISOR PROFESSOR SAMIR-EL-KHURI	SCALE AS SHOWN · DATE APR. 1965



PLAN
SCALE 1/100

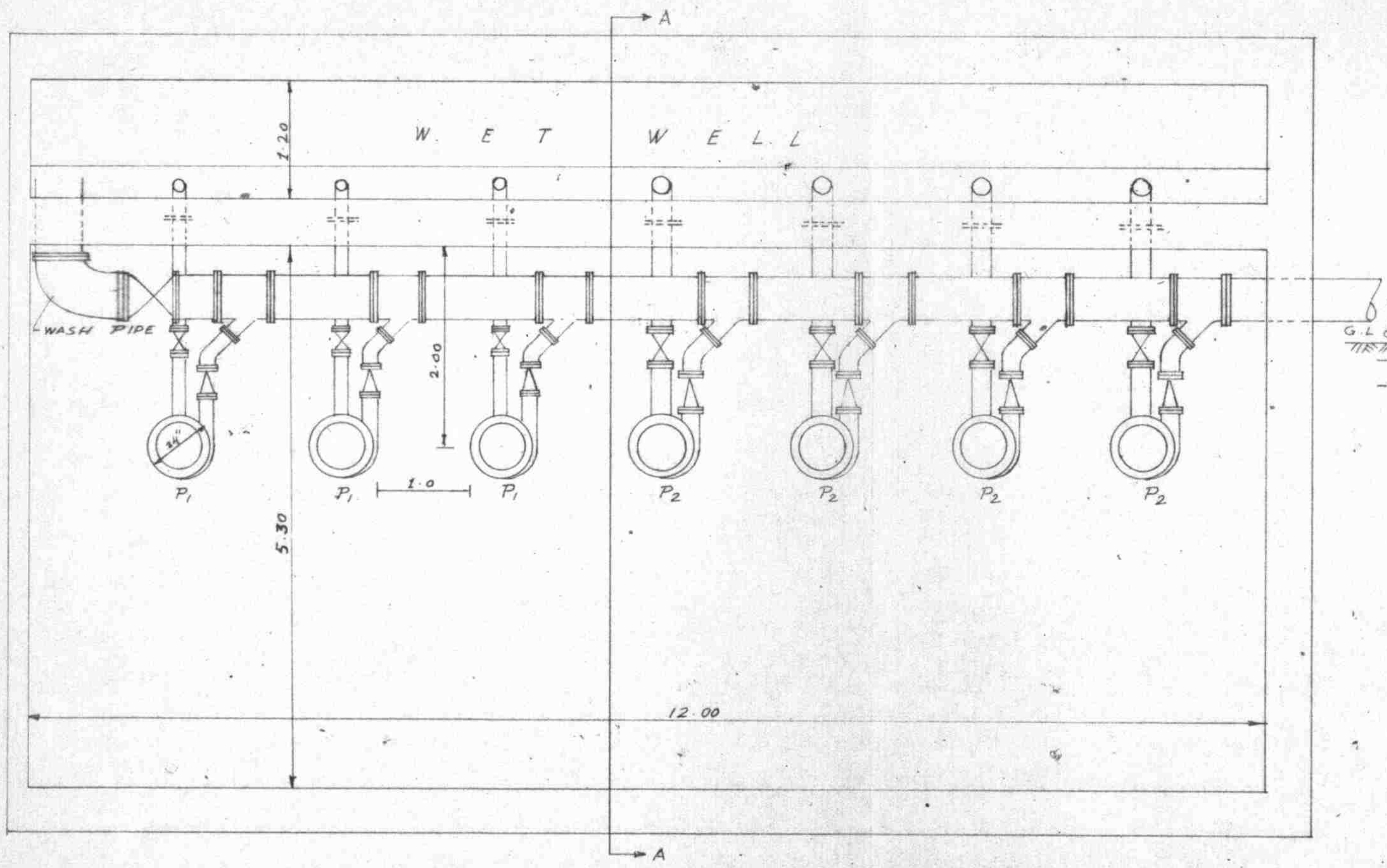


GRADED GRAVEL

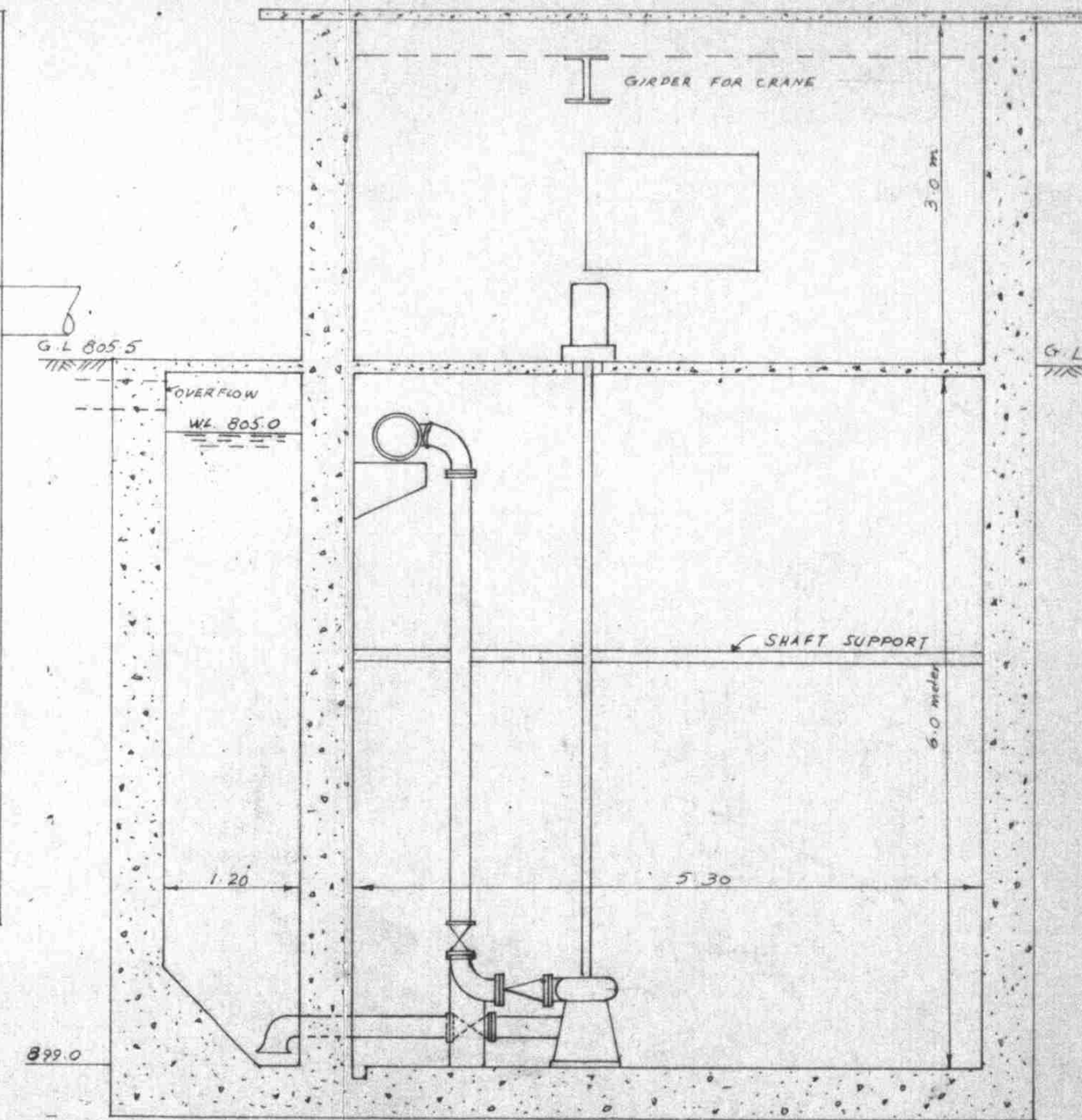
Bottom	1½" - 2"	– 5" layer
Middle	1"	– 3" layer
Top	½"	– 4" "
	TOTAL = 12"	

SECTION AT AA
SCALE 1/25

HEBRON SEWERAGE SCHEME		
SLUDGE DRYING BEDS		
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 19	
ADVISOR PROFESSOR SAMIR-EL-KHURI	SCALE AS SHOWN	DATE APR: 1965

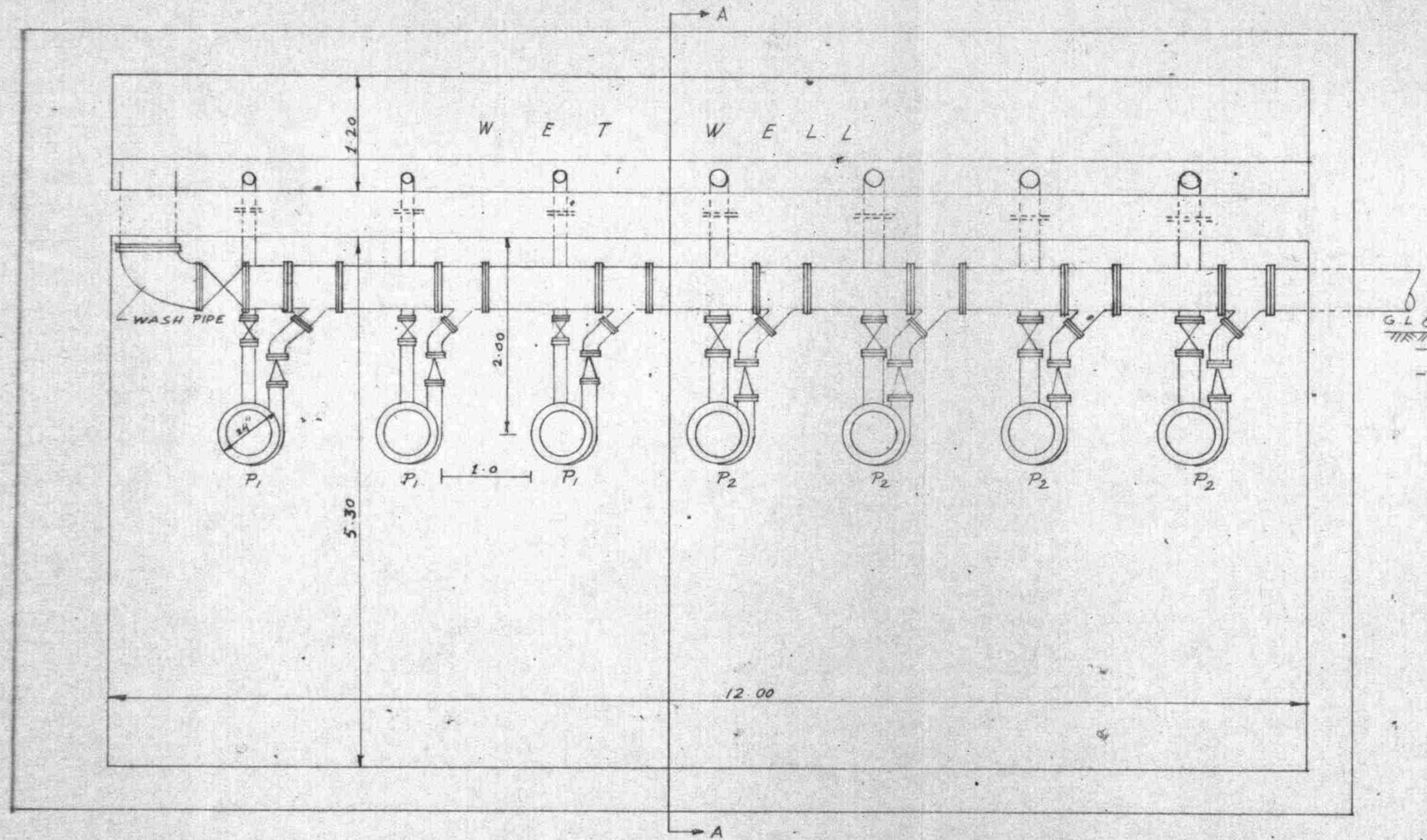


PLAN

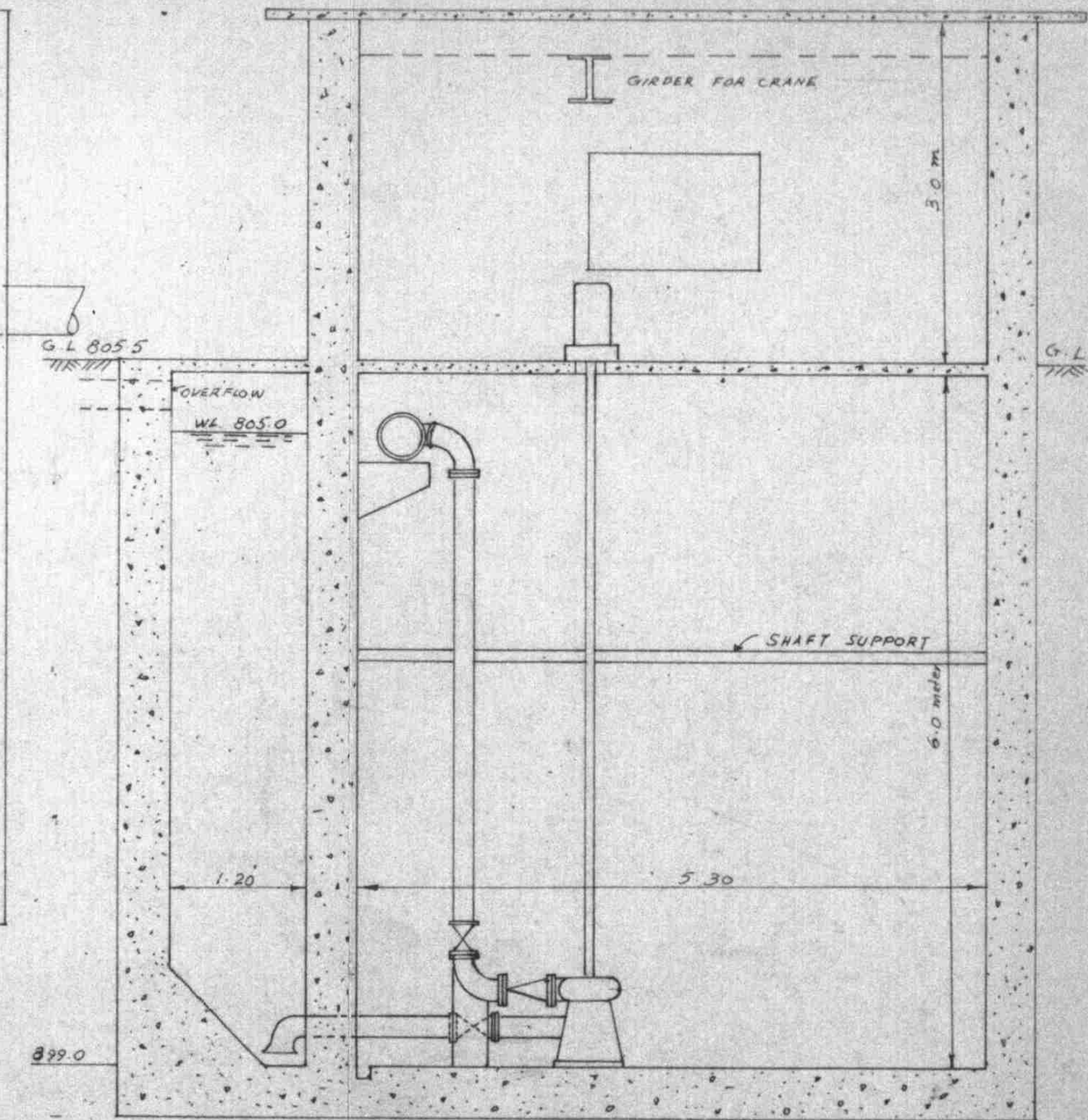


SECTION AT AA

HEBRON SEWERAGE SCHEME	
RECIRCULATION PUMPING STATION	
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 20
ADVISOR PROFESSOR SAMIR-EL-KHORI	SCALE 1/50 DATE APR. 1965



PLAN



SECTION AT AA

HEBRON SEWERAGE SCHEME	
RECIRCULATION PUMPING STATION	
ENGINEER SAIYED HASAN AKHTAR	DRAWING NUMBER 20
ADVISOR PROFESSOR SAMIR-EL-KHORI	SCALE DATE 1/50 APR. 1965