PRELIMINARY DESIGN

OF

SEWERAGE SYSTEM FOR SULAIMANIYA, IRAQ

By

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A Thesis

Submitted to the School of Engineering
American University of Beirut in
partial fulfillment of requirements
for the Degree of Master of Engineering
with major in Sanitary Engineering.

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Beirut,
December 1965.

ACKNOWLEDGEMENT

The author wishes to acknowledge his sincere indebtedness to Professor Samir El-Khuri and Professor Aftim Acra for their able guidance, encouragement and constructive criticism. In fact, it would have been difficult to complete this project, had the writer not received the whole-hearted cooperation of his advisors in all stages of the work.

The author is also grateful to Professor C.J. Inglisses for his help and guidance in the finalization of this project. Also, gratitude is due to the Associated Consulting Engineers, Beirut, and their staff for their technical help.

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Sulaimaniya is one of the fourteen large cities in Iraq situated in the northeastern part of the country, and has an estimated population of 57,000.

The problem created by the undesirable conditions arizing from the present situation regarding liquid waste in Sulaimaniya has prompted this project which is intended to improve the existing situation.

The available data has been studied, analized and used as a basis for the proposed design involving the sewerage systems and treatment plant.

A conventional system for sewage treatment consisting of primary and secondary sedimentation and trickling filter has been adopted. The selection of the system is primarily based on its practicality and economy in addition to other criteria. Consideration has also been given to various other treatment systems, but preference has been given to the trickling filter for various reasons given in the text.

The approximate quantities of material and work needed to accomplish the project has been estimated and the cost determined. The estimated cost of the whole scheme comes to about 646,734 I.D. ($ 1,790,000), or $ 16.00 per head for the total anticipated population.

1 INTRODUCTION

1.1. Scope of the Work

This project is concerned with the design of the sewerage system for Sulaimaniya, Iraq, and includes a discussion on the materials, location and sizes of mains and trunks, establishment of all basic criteria, feasibility of methods of sewage treatment and design of all parts of the treatment works.

Details of the design, construction, drawings, estimates and specification are not provided for they are considered beyond the scope of this project.

2.1. Historical Background

Sulaimaniya, the capital city of Northern Iraq, is believed to have been erected on the site of a small village - Malik Kindi. One of the state rulers (Wali) by the name of Mahmoud Pasha had built adjacent to it a palace (saray) before 1784. His nephew, Ibrahim Begh who succeeded him, constructed around the palace houses, a mosque, a public bath, a market and an inn. When the work was finished in 1784 he named the small town Sulaimaniya as a tribute to the Ottoman Ruler at Baghdad, Sulaiman Pasha the Great[1].

No important historical incidents are attributed to this city, except for the local Kurdish government established through British support after World War I. This government, however, did not survive for long as it was abolished in 1926 (2).

1.3. Location

The city of Sulaimaniya, one of the largest cities in Iraq, with an estimated population of 60,000, is located in the northeastern part of the country about 40 km from the Iranian border. The distance to Baghdad is 270 km and to Kirkuk, the nearest large city, 100 km. The city is located on a high plateau which is to a large extent cultivated and lies 850 m above sea level. Its latitude is $35^{\circ} 33'$ north and longitude $45^{\circ} 26'$ east. The city is surrounded by high mountain chains on the east and southwest. The distance from the city to the eastern mountain chain is 3 km and to the southwestern chain 12 km. About 6 km west of the city the high plateau is crossed by a small tributary of the Diyala River. On this tributary at Surchinar, 7 km northwest of the city, is the newly built cement factory with its own living quarters for the plant employees. The location of Sulaimaniya is shown in Figure 1.1.

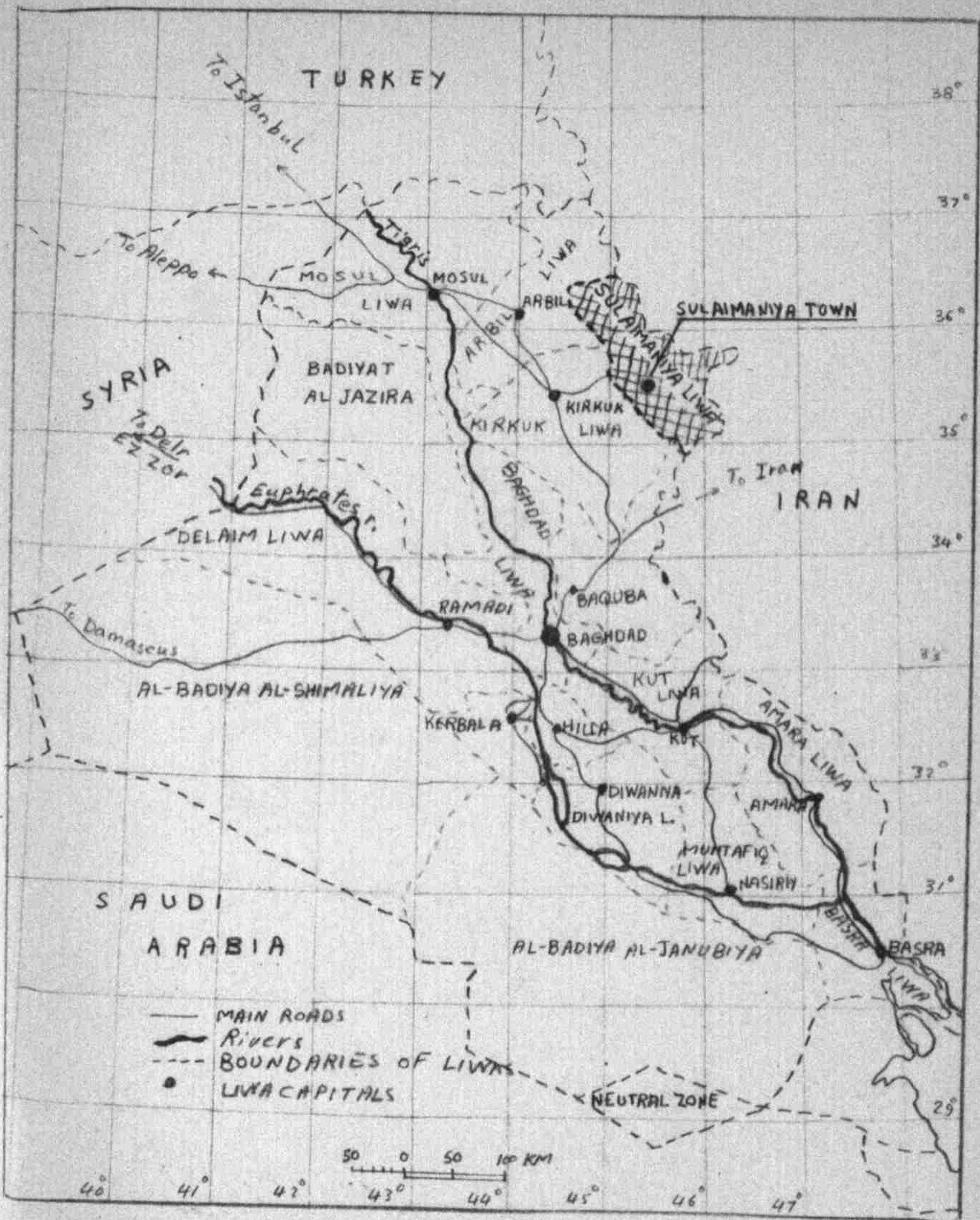

FIG. 1.1.

LOCATION OF SULAIMANIYA

1.4. Living Quarters

Sulaimaniya consists mainly of old buildings situated in the centre of the city, which is characterized by its narrow roads, covered alleys and joined dwellings. Modern buildings are found mainly in the western and northern parts of the city. Some have been erected on the eastern side following the filling up of an older flood control canal which was replaced by a new one located 500 m further east.

1.5. Communications

Communications with Sulaimaniya are maintained entirely by highways. The main road, which is first class, is the recently completed one to Kirkuk. A highway to Baghdad via Derbendi Khan is under construction. The requiring construction is the one from Sulaimaniya to Arbat, a distance of 20 km. A road to the northwest joining the Kirkuk road links Sulaimaniya with the Dokan Dam; another road to the east meeting the road to Derbendi Khan connects with roads to Halabeha and Penjwin.

There are no railway connections with Sulaimaniya, nor have any been planned. Similarly, there is no airport in Sulaimaniya or the vicinity and, therefore, no air communications with the city.

1.6. Topography

The topography of the city and its surroundings is shown in Drawing No. 1. The contours shown are 2m. apart. The terrain within the city slopes largely from northeast to southwest. The highest parts in the northeast are 910 m above sea level. The lowest parts of the city, in the south at Wadi Walobah, are 800 m above sea level. Within a stretch of 3 km from the northeastern to the southwestern part of the city the ground level falls 100 m. Within the city the ground level slopes evenly without any greater hilliness. In the areas south of the city (south of Wadi Walobah) the terrain is hilly. In certain parts of the areas nearest the city, in the west and southwest, there are few hills which are noticeably higher than the surrounding ground.

The terrain is crossed by several smaller and fairly distinct valleys (wadis) which, during the rainy periods, are water-bearing. The most noted of these is the previously mentioned Wadi Walobah, which runs south of the city in a northeast - southwest direction, and Wadi Karaiyah Washak west of the city running in a direction parallel to Wadi Walobah.

1.7. Climatic Conditions

1.7.1 Temperature

No official statistics pertaining to temperature conditions in Sulaimaniya are available. The data on temperature given here refer to the Bakrajo Farm, situated about 7 km west of Sulaimaniya. This data can, however, be regarded as representative of the city of Sulaimaniya also[4].

Sulaimaniya has, as a consequence of its geographical location, the mildest summer of all the important cities of Iraq. The annual mean temperature is 18°C. The warmest month is June with a mean temperature of 31°C and with a mean maximum temperature of 39°C. The coldest month is December with a mean temperature of 6°C and with a mean minimum temperature of 0.5°C. The monthly mean temperature as well as the mean maximum and minimum temperatures are shown in Table 1.1. The highest and lowest recorded monthly temperatures are not available.

The monthly mean temperatures are presented graphically in Figure 1.2.

TABLE 1.1

MONTHLY MEAN TEMPERATURES[4]

Month	Mean Values °C		
	Daily	Maximum	Minimum
January	8.5	12.0	5.0
February	13.5	19.5	8.5
March	22.0	30.0	14.5
April	27.0	34.5	19.5
May	30.0	39.0	22.0
June	31.0	39.5	23.0
July	27.0	35.5	19.5
August	19.5	28.0	11.5
September	14.5	20.0	7.5
October	10.5	16.0	5.0
November	8.0	11.5	2.5
December	6.0	10.0	0.5

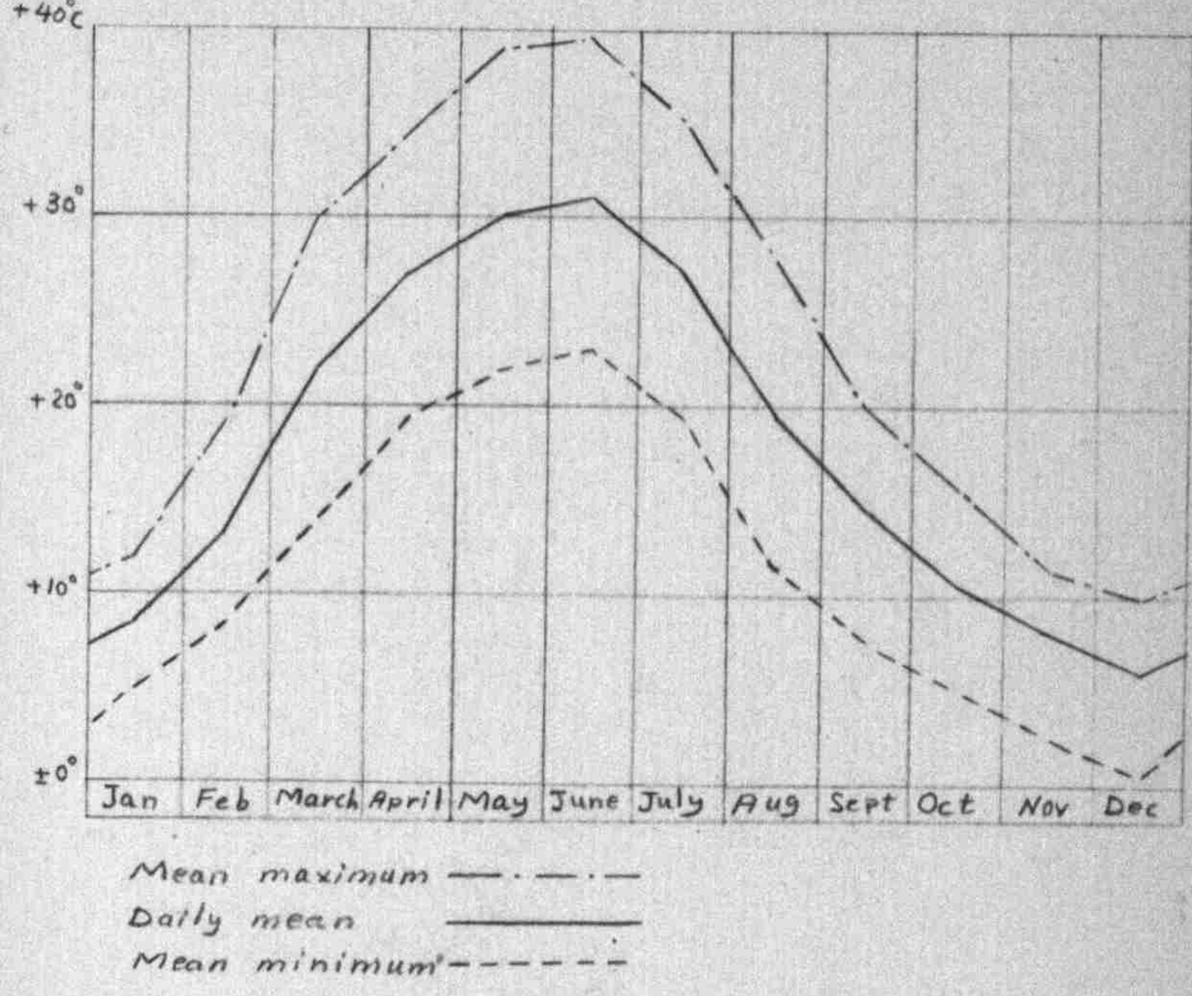

FIG. 1.2.

TEMPERATURE

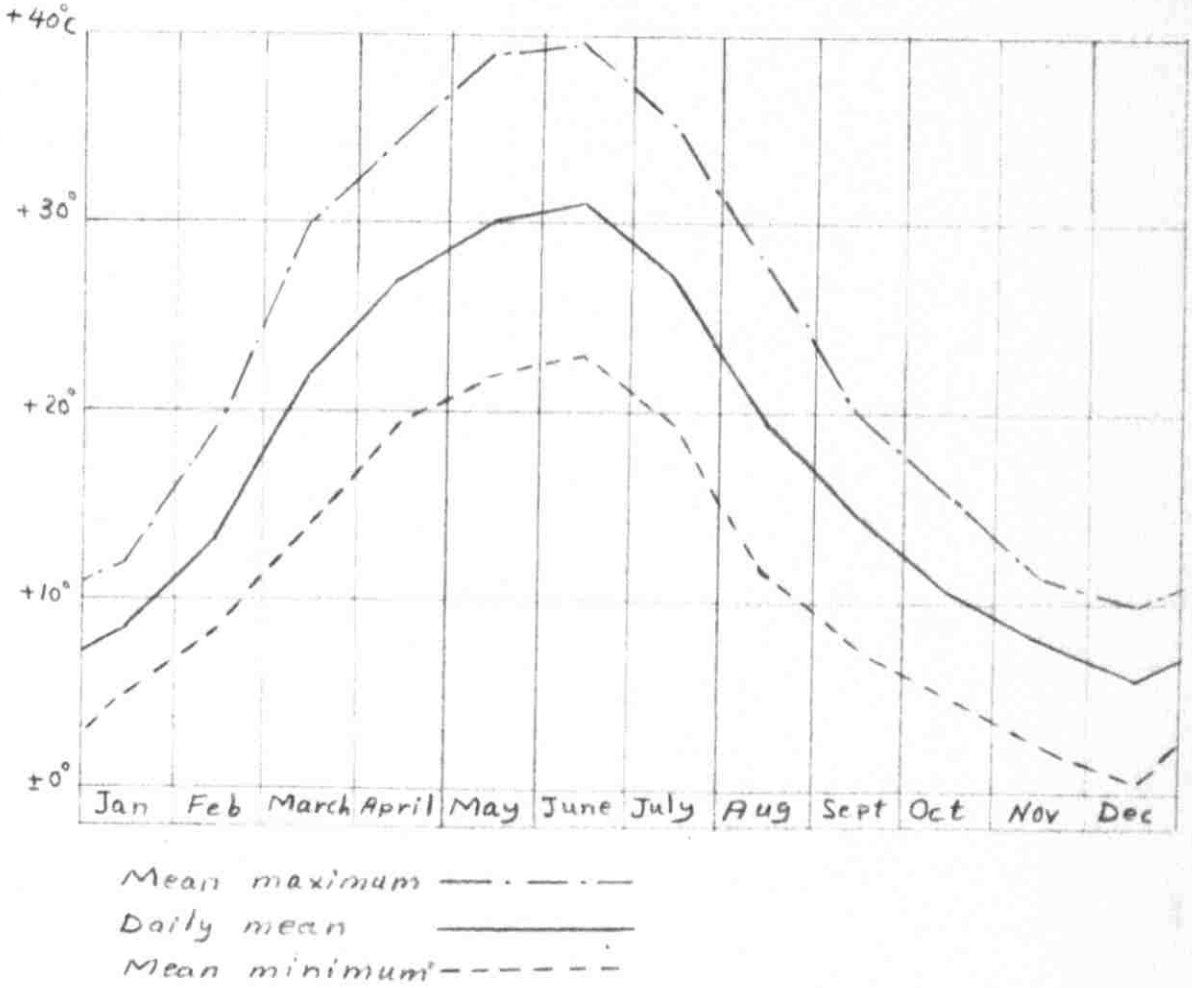

FIG. 1.2.

TEMPERATURE

1.7.2 Precipitation

The annual mean precipitation in Sulaimaniya, according to the Ministry of Communications, amounts to 720 mm. The monthly mean precipitation values are shown in Table 1.2.

TABLE 1.2.[5]

MONTHLY MEAN PRECIPITATION

Month	Precipitation (inches)	Month	Precipitation (inches)
January	4.66	July	0
February	4.55	August	0
March	4.81	September	0
April	4.60	October	0.37
May	1.63	November	3.12
June	0	December	4.57

The monthly mean precipitation values are presented graphically in Figure 1.3.

1.7.2 Precipitation

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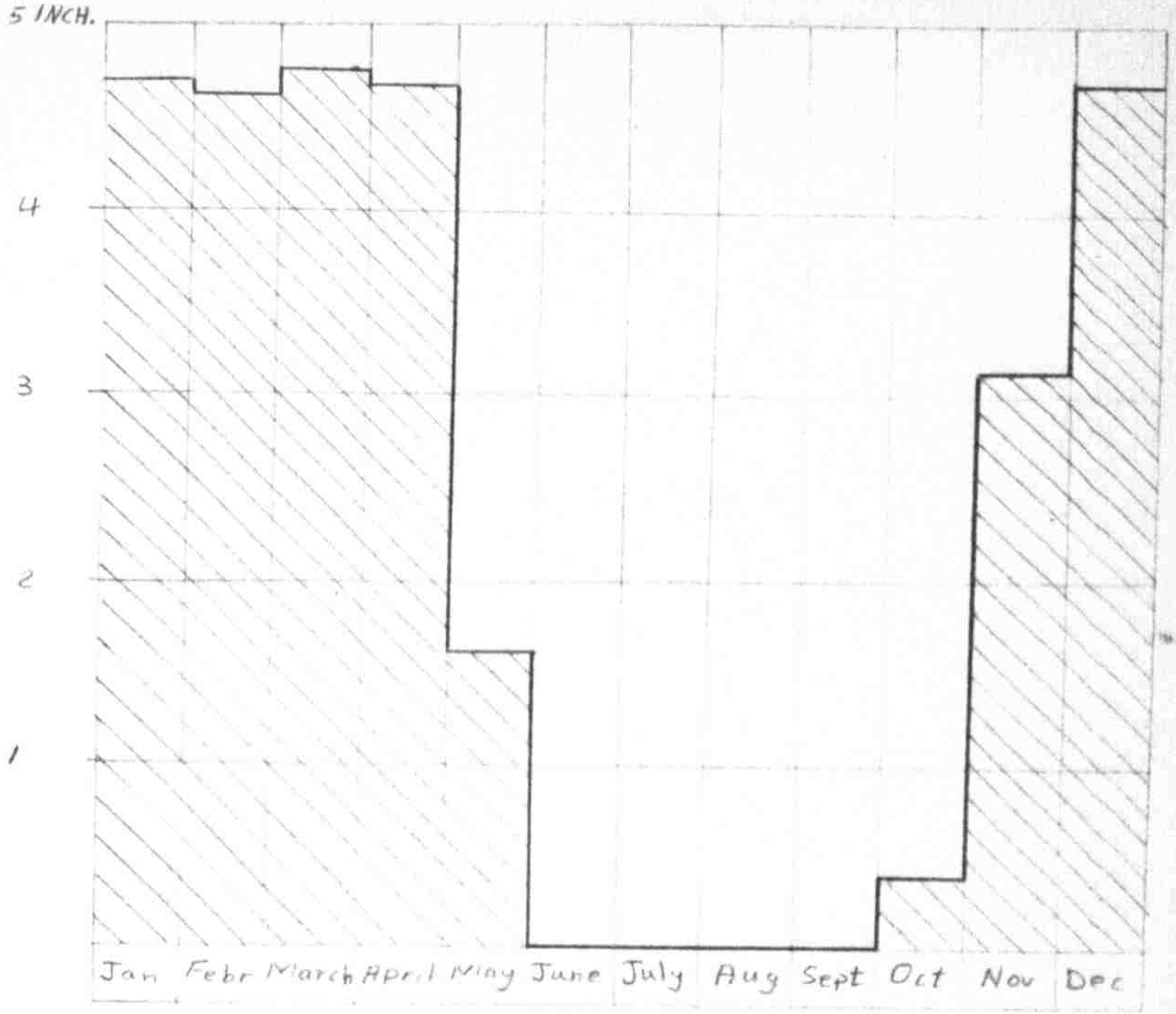

FIG 1.3.

RAINFALL

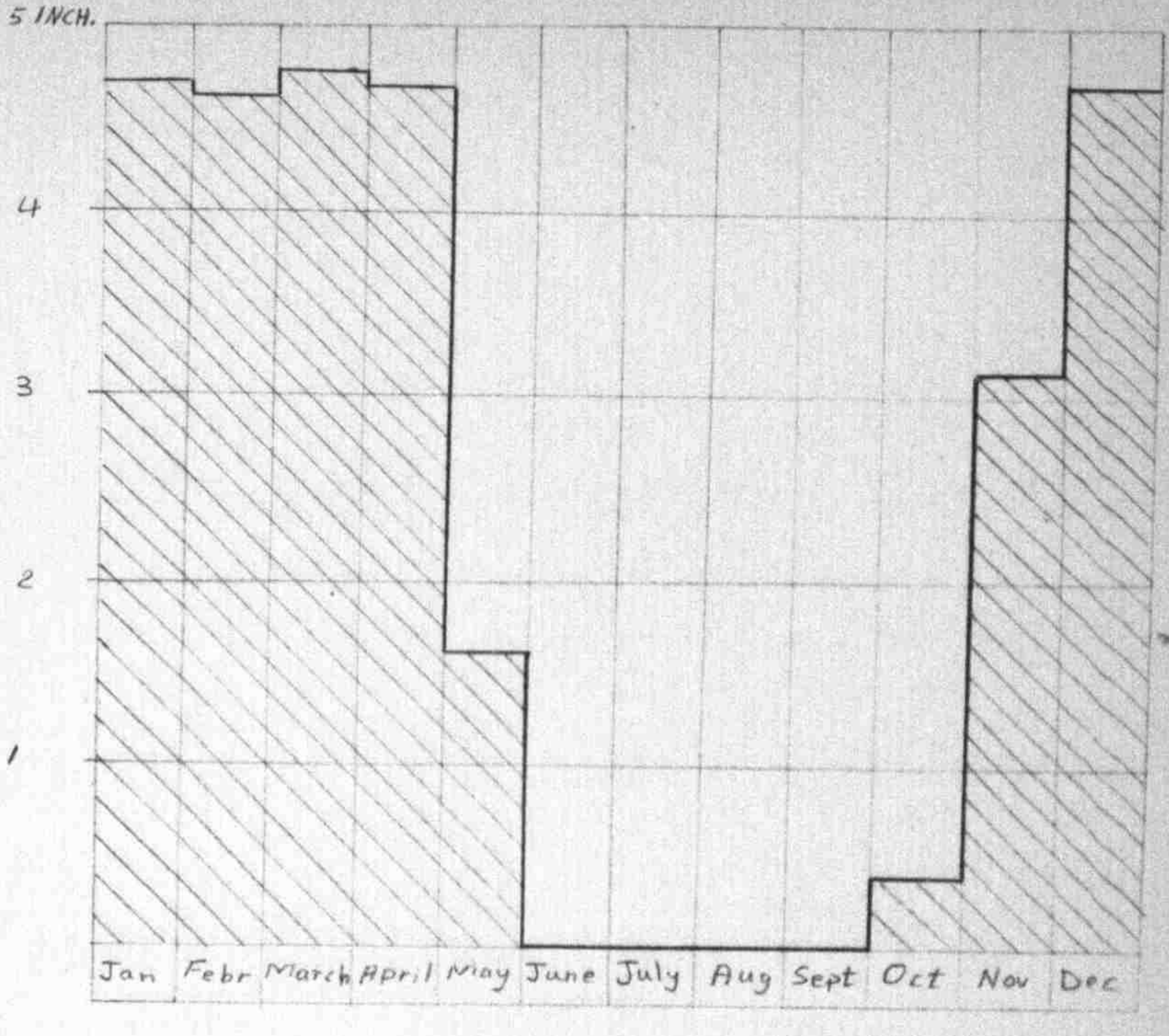

FIG 1.3.

RAINFALL

The maximum recorded precipitation in 24 hours during the period from 1955 to 1962 is shown in Table 1.3.

TABLE 1.3.

MAXIMUM PRECIPITATION IN 24 HOURS
1955 - 1962[5]

Year	Precipitation (in)	Year	Precipitation (in)
1955	3.10	1959	2.00
1956	1.41	1960	4.95
1957	2.56	1961	1.81
1958	2.72	1962	1.42

1.7.3 Relative Humidity

The monthly mean values for the relative humidity are shown in Table 1.4.

TABLE 1.4.(5)

MONTHLY MEAN RELATIVE HUMIDITY

Month	Relative humidity %	Month	Relative humidity %
January	56	July	16
February	56	August	15
March	67	September	17
April	60	October	25
May	42	November	50
June	22	December	65

The lowest humidity, around 15 percent, is noted for July to September, whereas the highest humidity, around 60 percent, is noted for the months of December to April. The variation in the relative humidity during the year is also shown in Figure 1.4.

1.7.4 Winds

The prevailing winds in Sulaimaniya are northeasterly. On certain occasions these can become rather strong and are known locally as the "black winds".

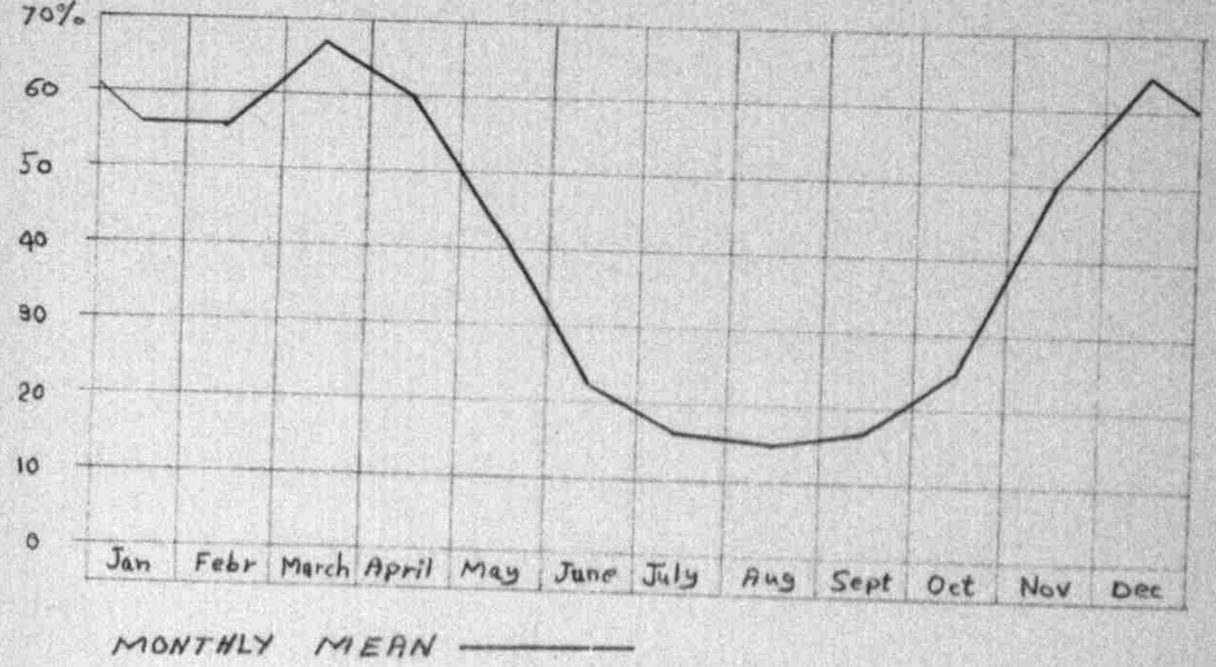

FIG. 1.4.

RELATIVE HUMIDITY

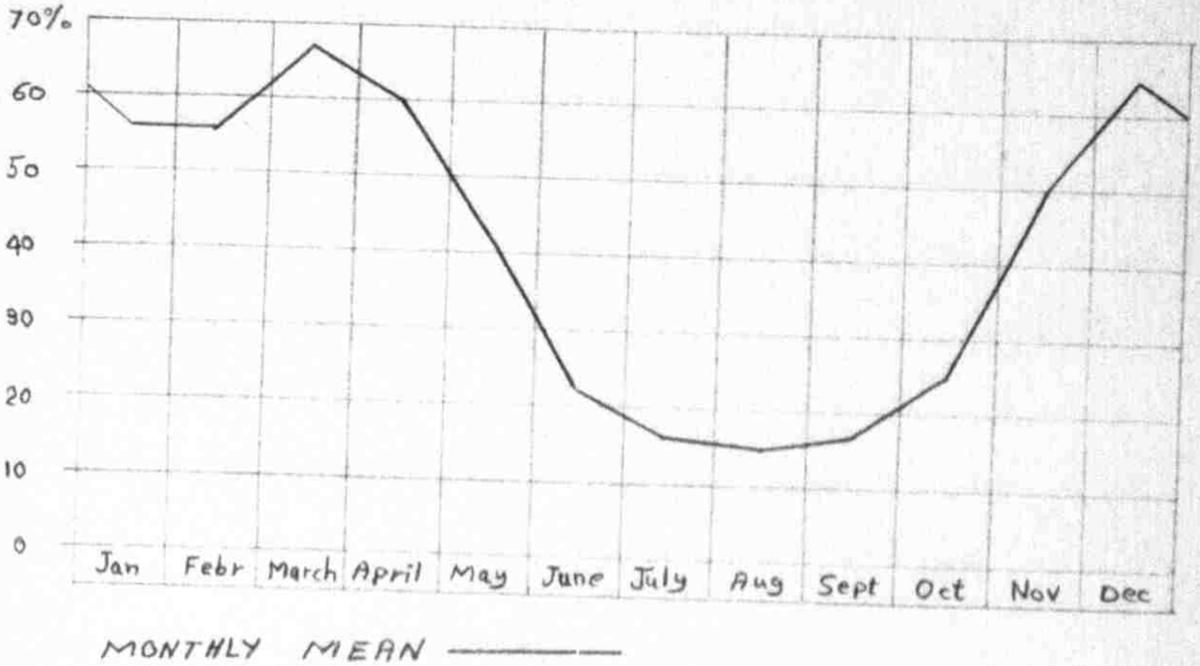

FIG. 1.4.

RELATIVE HUMIDITY

1.8. Industries

Sulaimaniya is the important commercial and administrative centre for Sulaimaniya Liwa - an administrative district comprising 12.000 km^2 with a population of 300,000. This Liwa is an important agricultural region with about one third of its area being cultivated. The important agricultural products include tobacco, cotton, lentils and peas. Animal husbandry is also important, especially the raising of goats. Lumber trade is also significant.

Handicrafts and small industries constitute a considerable part of the occupations in the city. Here, there is only one large industry, a newly built cigarette factory, which employs 200 men and has a daily production of 4 million cigarettes.

1.9. Soil Conditions

1.9.1 Soil Properties

In order to determine the soil properties within the built up part of the city and the neighboring regions, a ground investigation was carried out by the ground water section of the Directorate General of Planning and Design, comprising machine drilling to normal construction depth or to the rock, if this was encountered earlier. The drilling was performed with a gasoline motor driven bore. From the drillings it could be established that the loose soil layers are composed predominantly of fine sandy clay interspersed with sandy soil within certain parts. The bedrock is composed of limestone which lies deep below the ground surface. In most parts of the area southwest of the city the bedrock is found at a comparatively slight depth below the surface. The ground properties were found to be such that they should not present great difficulties in the construction of pipe lines and sewage treatment plant.

1.9.2 Ground Water Conditions

In order to determine the ground water conditions a total of five exploratory 2-inches pipes were sunk within the city and the neighboring regions. The level of the water table was found in this way to be so low that the ground water is of no importance in the planning and dimensioning of the sewerage [3].

1.10. Master Plans

As yet no definite master plan for the development and expansion of the city has been agreed upon. In 1958 Doxiadis Associates, Consulting Engineers, Athens, commissioned by the Development Board of the Ministry of Development, drew up a master plan for the city and its immediate surroundings. The plan was presented in an interim report called "The Future of Sulaimaniya". This plan has, however, never been ratified. Nor have the main lines and recommendations for the development of the city construction of new throughfares, residential districts, etc.. The construction of new throughfares taking place at present are being carried out according to plans drawn by the Directorate General of Planning and Design.

The initial steps for the establishment of a master plan for the city were instituted in the fall of 1963 by the auther at the Directorate General of Planning and Design, Ministry of Municipalities, Baghdad.

1.11. Available Map

In this project a contour-map with a scale of 1/5000, 1/10,000, 1/20,000, made by Hunting Aerosurveys Ltd. in 1962, has been used since it is the only available map.

2 WATER CONSUMPTION AND EXISTING SEWAGE DISPOSAL

2.1. Water Consumption

2.1.1 General

Sulaimaniya has well-organized water works that supply most of the drinking water to the city. There is a well-branched pipeline system in the city with a high reservoir in the northern part. The water is taken partly from drilled wells in the immediate vicinity of the city, the so-called Sulaimaniya Kehrizes, and partly from a spring situated in the village of Surchinar, 7 km west of the city. The total capacity of the wells in the city is limited to 2000 m^3 per 24 hours, whereas the water supply in Surchinar has a capacity that can meet the water requirements of the city for sometime to come.

2.1.2 Present Water Consumption

Information concerning the measured water consumption is available since 1957[(3)]. The monthly values of water consumption during the period January 1957 - August 1962 are shown in Table 2.1. However, values of water consumption beyond August 1962 are not possible to obtain since they are not available at the Ministry of Municipalities.

TABLE 2.1.

MEASURED WATER CONSUMPTION FROM JAN., 1957
TO AUG., 1962. (m^3 PER MONTH). (3)

Month	1957	1958	1959	1960	1961	1962
Jan.	79,000	99,000	107,000	123,000	129,000	153,000
Feb.	78,000	83,000	106,000	124,000	128,000	148,000
March	76,000	109,000	108,000	133,000	141,000	161,000
April	99,000	117,000	150,000	149,000	161,000	188,000
May	98,000	163,000	141,000	182,000	210,000	166,000
June	129,000	187,000	185,000	177,000	255,000	275,000
July	130,000	164,000	192,000	191,000	253,000	295,000
Aug.	147,000	169,000	181,000	198,000	258,000	247,000
Sept.	130,000	157,000	173,000	176,000	209,000	-
Oct.	109,000	146,000	146,000	168,000	201,000	-
Nov.	99,000	135,000	111,000	135,000	161,000	-
Dec.	82,000	119,000	107,000	125,000	140,000	-
Total	1,256,000	1,648,000	1,707,000	1,881,000	2,246,000	

The measured water consumption has increased from 1,256,000 in 1957 to 2,246,000 m^3 in 1961, amounting to an increase of 79 percent. The average and maximum values of the monthly consumption as well as the average values of the 24-hour consumption for the years 1957 - 1961 are given in Table 2.2.

TABLE 2,2.

MEASURED WATER CONSUMPTION FROM 1957 TO 1961
AVERAGE AND MAXIMUM MONTHLY VALUES AND
AVERAGE 24 - HOUR VALUES.

Year	1957	1958	1959	1960	1961
Monthly Aver. (m^3/mon.)	105,000	137,000	142,000	157,000	187,000
Monthly Max. (m^3/mon.)	147,000	187,000	192,000	198,000	258,000
24-hr. Aver.(m^3/24 hr.).	3,450	4,500	4,700	5,200	6,200

The amount of water pumped from the spring in Surchinar to Sulaimaniya has been registered since August 1961. In Table 2.3. are shown the monthly values for the period August 1961 to September 1962.

TABLE 2.3.

AMOUNT OF WATER PUMPED FROM SURCHINAR SPRING
FROM AUGUST 1961 TO SEPTEMBER 1962 (3)

Month	1961	1962
January	-	147,000
February	-	151,000
March	-	200,000
April	-	206,000
May	-	229,000
June	-	231,000
July	-	245,000
August	243,000	237,000
September	233,000	222,000
October	245,000	-
November	230,000	-
December	207,000	-

The amount of water pumped from Surchinar Spring to Sulaimaniya during the 12-month period from August 1961 to July 1962 as calculated from Table 2.3 was 2,557,000 m^3. There is reason to believe that the Sulaimaniya Kehrizes during this period were utilized at their full capacity, i.e. 2000 m^3 per 24 hours. Thus during the 12-month period these wells have supplied 730,000 m^3. The total amount of water supplied from all sources during this period is then 3,287,000 m^3. The measured water consumption during the same period as calculated from Table 2.1 was 2,355,000 m^3. The difference between the calculated and the measured amount of water supplied is 932,000 m^3. This water quantity thus relates to the unmeasured consumption which includes irrigation, sundry uses, and losses. These calculated values expressed as 24-hour averages are given in Table 2.4.

TABLE 2.4.

WATER PRODUCTION AND MEASURED WATER CONSUMPTION FROM AUGUST 1961 TO JULY 1962.

Water production	9,000 m^3/24 hr.
Measured water consumption	6,500 m^3/24 hr.
Unmeasured water consumption (irrigation, losses, etc.)	2,500 m^3/24 hr.

According to the 1956 housing census, 86 percent of the dwellings were connected to the central water supply system(3), which is a remarkably high figure compared to other cities in Iraq. According to information from the water works, the number of consumers (i.e. service connections) in 1960 was 5,685 m^3, in 1961 6,102 m^3, and in 1962 6,484 m^3. It is known that each connection serves 8.5 persons. Assuming, with the guidance of these figures, that from August 1961 to July 1962 the number of connections to residential buildings amounted to 6,000, then during this period 93 percent of the population have been receiving water services - an increase of 7 percent since 1956.

The percentage of the population served with municipal water is represented in Fig. 2.1. It is estimated that 96% of the population have been receiving piped water during 1965.

The water consumption for household purposes and industries has been estimated to average 7500 m^3 per 24 hours. With 54,720 consumers this corresponds to a 24-hour average specific consumption of 137 liters per person, which is equivalent to 36 gallons per capita.per day.

2.1.2 Future Water Consumption

In view of the comparatively rapid rise in the standard of living in Iraq today, which is expected to rise even more rapidly in the near future, the specific water consumption -

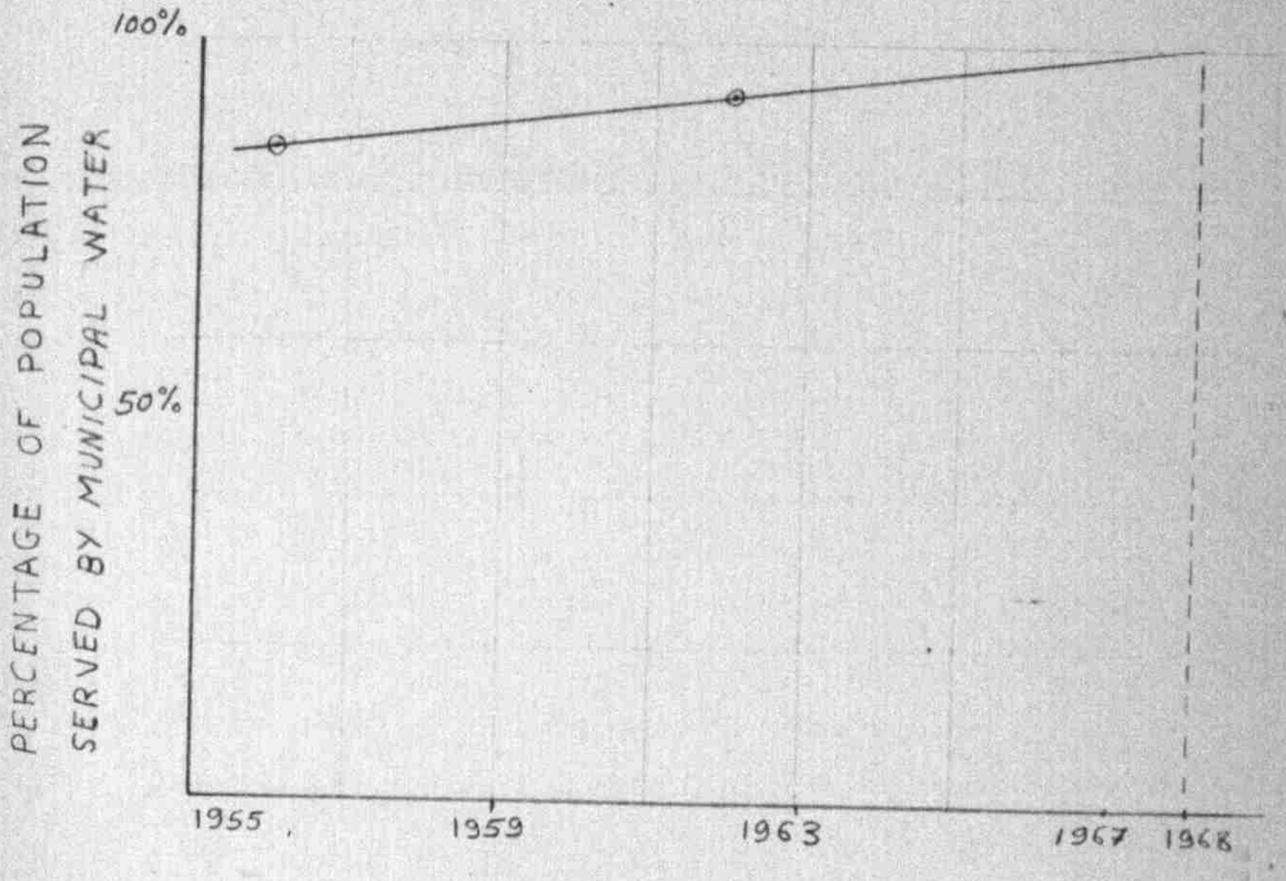

FIG. 2.1.

MUNICIPAL WATER SERVICES

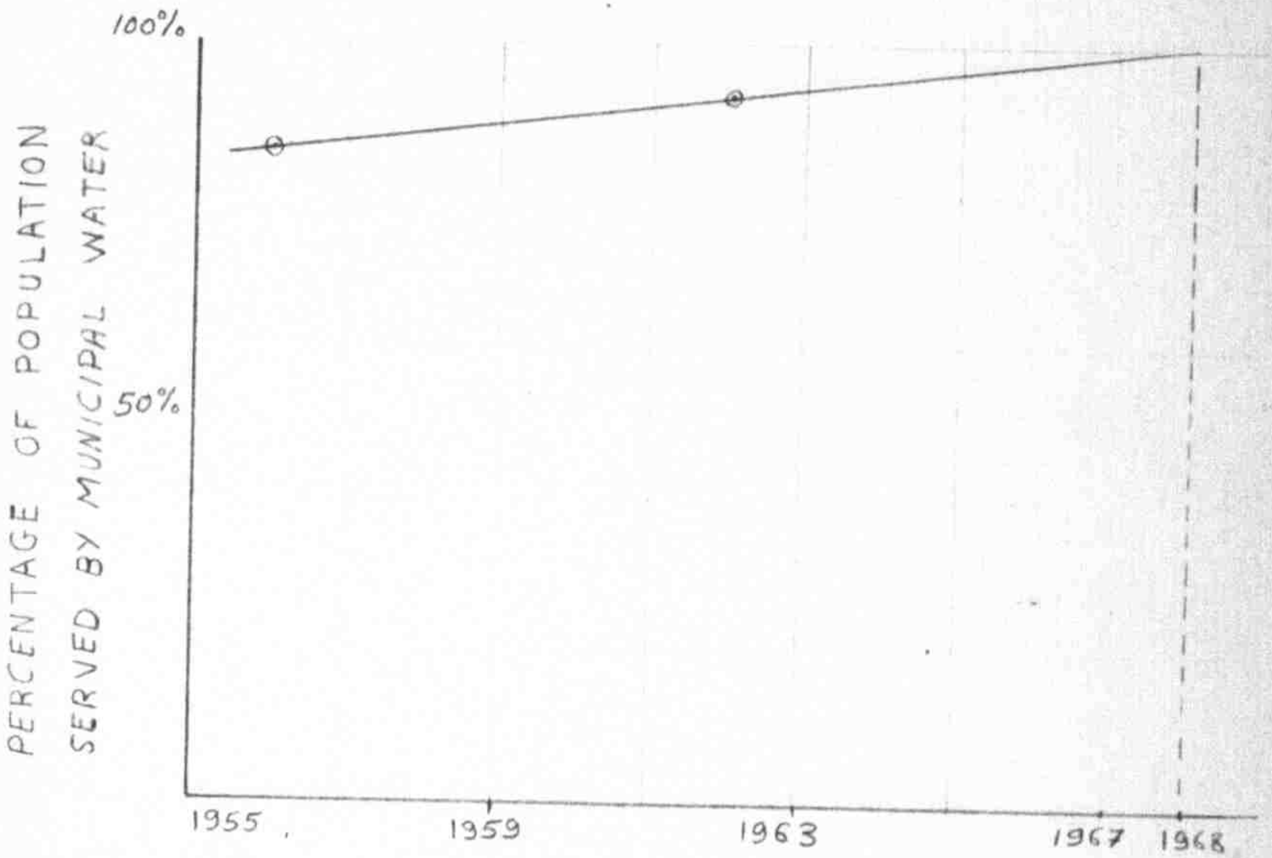

FIG. 2.1.

MUNICIPAL WATER SERVICES

i.e. the total water consumption for household purposes and industries and losses divided by the number of inhabitants - will increase comparatively rapidly. The specific water consumption is estimated at present at 137 liters per capita per 24 hours and can be assumed for a period up to the year 2015 to rise to the values given in Table 2.5.

TABLE 2.5.

SPECIFIC WATER CONSUMPTION 1965 - 2015

Year	Specific Consumption L/c/d
1965	137
1975	166
1985	200
1995	232
2005	264
2015	300

The specific water consumption from 1965 to 2015 is represented graphically in Fig. 2.2.

The water consumption for irrigational purposes is at present 2000 m^3 per 24 hours on the average. This figure is expected to rise to 6000 m^3 per 24 hours by the year 2015.

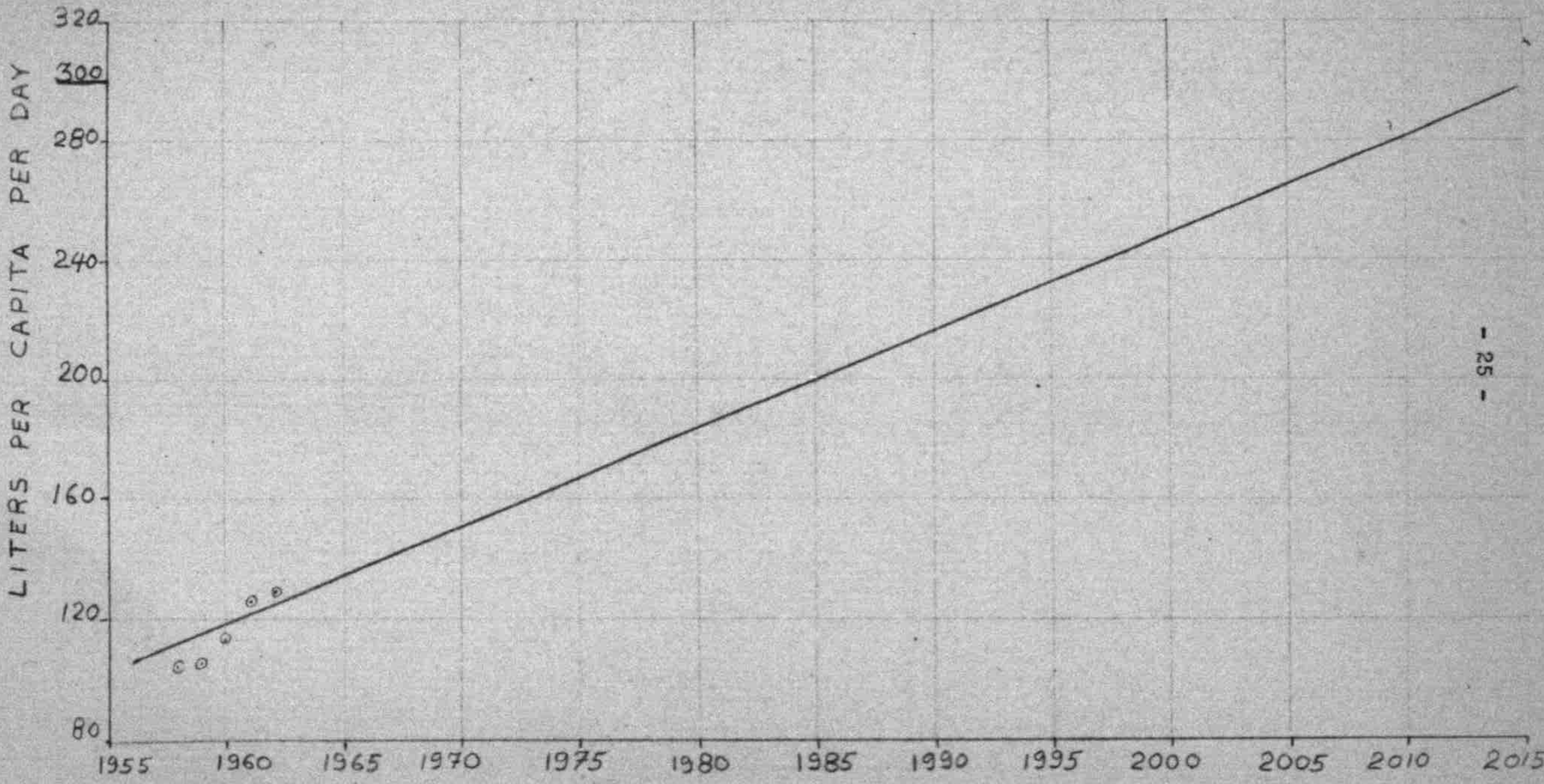

FIG. 2.2.

SPECIFIC WATER CONSUMPTION

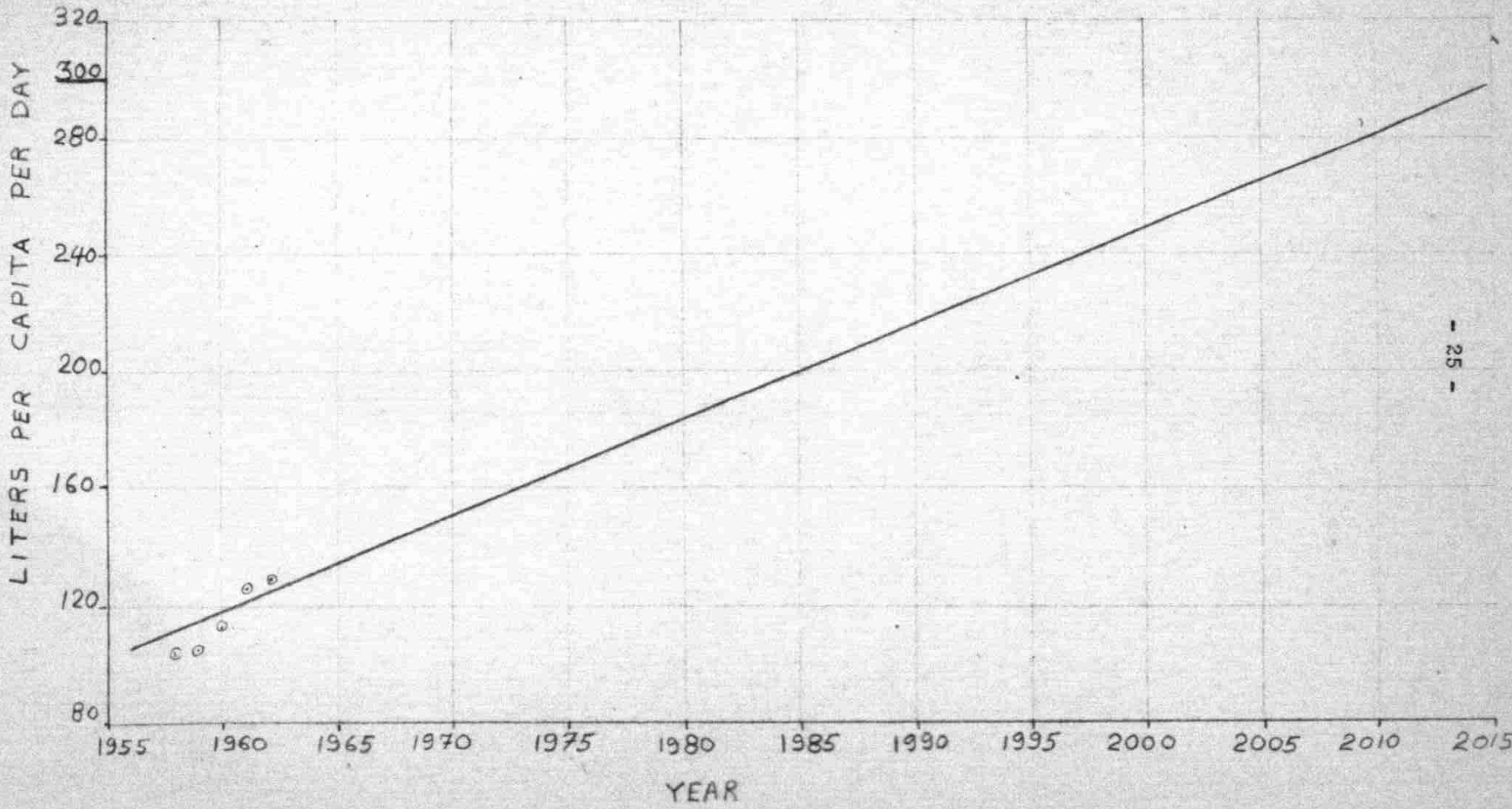

FIG. 2.2.

SPECIFIC WATER CONSUMPTION

On the basis of these assumptions the total water consumption from 1965 to 2015 can be estimated at the values given in Table 2.6, and also represented graphically in Fig. 2.3.

TABLE 2.6.

ESTIMATED AVERAGE 24 - HOUR WATER CONSUMPTION

1965 - 2015 (m^3/24 hr.)

Year	1965	1975	1985	1995	2005	2015
Household, etc.	7,500	11,800	17,200	23,400	28,600	33,800
Irrigation	2,000	2,500	3,500	4,500	5,500	6,000
Total	9,500	14,300	20,700	27,900	34,100	39,800

2.2. Existing Sewage Disposal

No proper Sanitary sewerage system exists in the old district of Sulaimaniya. Here the waste water is simply discharged into cesspools and, in few places, septic tanks are used.

In the newly built district of the city there is an incomplete sewerage system. Most of the pipelines in this

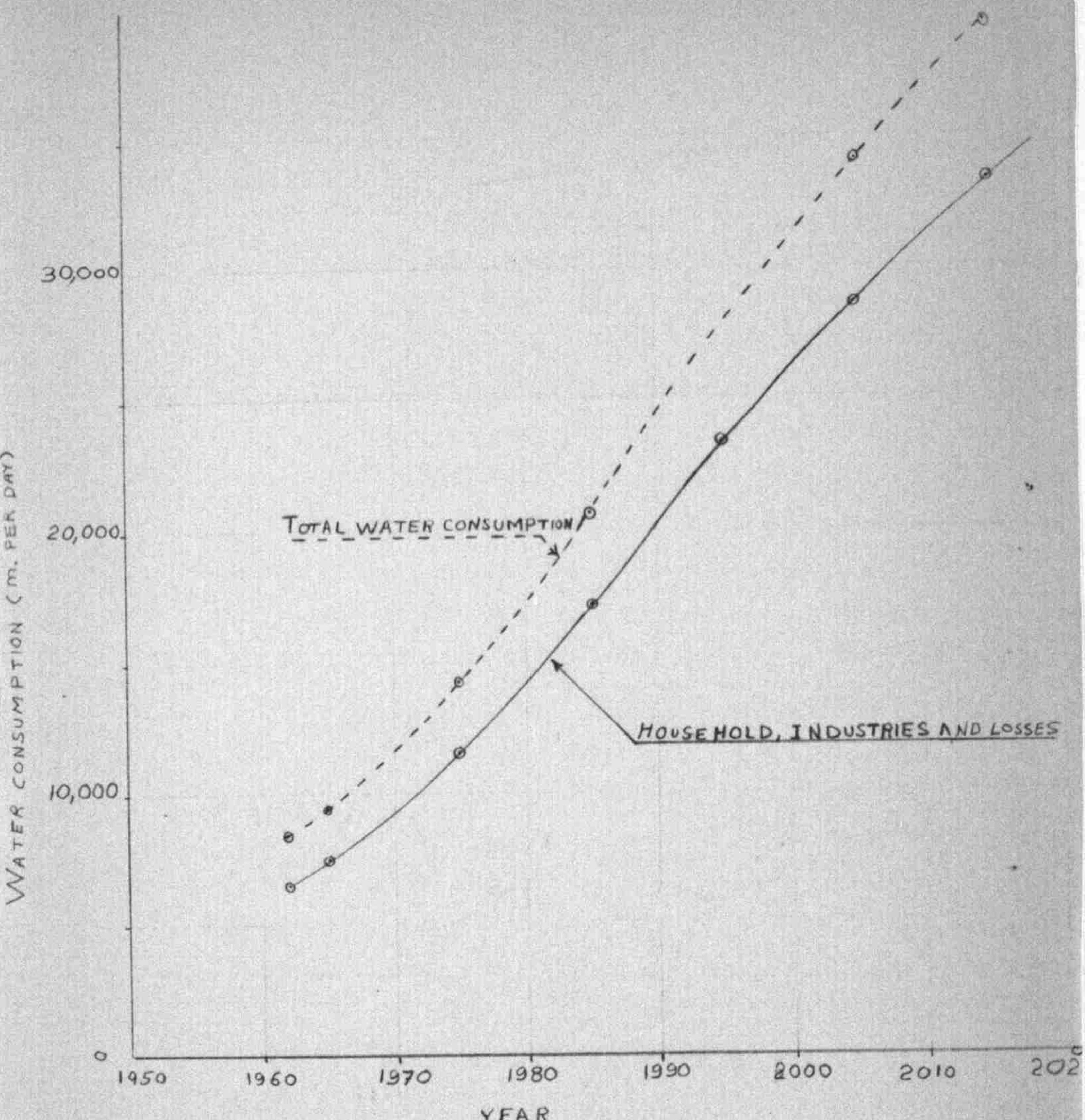

FIG 2.3.

WATER CONSUMPTION

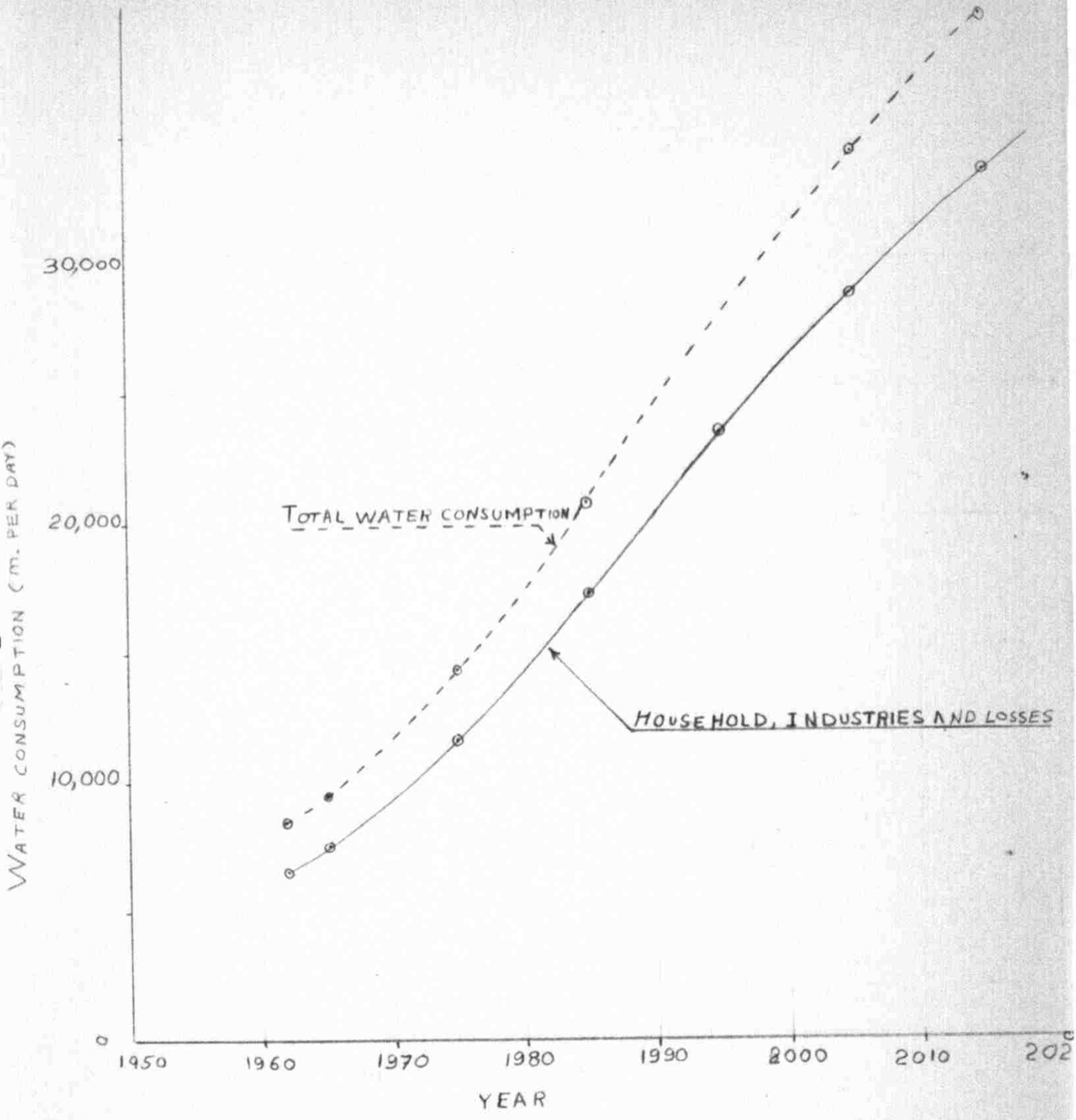

FIG 2.3.

WATER CONSUMPTION

district were constructed originally for the drainage of storm water. However, many pipelines are now used for waste water as well. The sewerage can thus be regarded as a combined system.

The main sewers are usually short and carry the waste water only to the nearest valleys (wadis) where the waste water flows in an open channel.

Diverting flood control canals have been built in order to unload the sewerage system and to protect the city from inundation during heavy runoffs from the slopes to the north and east of the city. There are two such canals in the northern part of the city which convey water masses from the north westward to Wadi Karaizah Washak. In the eastern part of the city, there is another similar canal running from north to south. This conveys water masses from the east southward to Wadi Walobah. The latter canal is of relatively recent construction and replaces an old one situated 500 m to the west of the new one and runs in the same direction. The location of the canals is shown in Drawing No. 1.

The pipelines of the old district are generally in poor condition. They are constructed mostly with rectangular sections of cement blocks or natural stone. They are often poorly maintained and in many places the pipelines have collapsed and become more or less filled. Many of these have been destroyed and intentionally filled up in order to dam up the waste water for irrigational purposes.

Manholes and gullies are often poorly maintained and consequently they do not serve their intended function. As a result of the poor condition of the pipelines inudations occur at times within certain parts of the city during rainfall. There is not any sewage treatment plant in the city. For these reasons, the existing sewerage system is considered useless and a new sanitary sewerage system is recommended for the city of Sulaimaniya.

Storm sewers are not required for the city of Sulaimaniya for the following reasons:

a) The existing storm sewers were constructed properly but their maintenance is poor. It is, therefore, recommended to repair and maintain the existing storm sewers and cut off any cross-connections with waste water.

b) Since there is no planning for the city, no practical proposal for the future storm sewers can be made at present. However, this matter will be discussed briefly in Chapter 5.

3 POPULATION ESTIMATES - PRESENT AND FUTURE.

The principal factors determining the future water needs of a municipality are its population and its industrial production. As these factors increase, the use of water and the disposal of used water will increase. Thus, in order to plan properly for sewerage schemes, sanitary engineers must be aware of present and expected future population and industrial growth.

3.1. Design Period

The number of years from the date of design to the estimated date when the conditions of design will be reached is the design period. The design period, however, depends upon the following factors[6] :

a) The useful life of structure and equipment employed.

b) Original and maintenance costs.

c) The availability of funds.

d) The expected water consumption at the end of the design period, and the availability of such water.

e) The ease, or difficulty, of extending or increasing the capacity.

f) The change in the purchasing power of money during the period of retirement of indebtedness.

g) 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 full capacity.

h) Rate of growth of population.

All of the above factors are variable, but the rate of growth of population is the most variable one. It depends upon many factors which will be discussed later in this chapter.

In practice, the design period varies between 25 and 50 years. For this project, a design period of 50 years has been adopted for the following reasons:

a) It is expected that the city of Sulaimaniya will be saturated within 50 years. Further increase in population after 50 years will require an extra area outside the municipal boundaries.

b) The topography of the city provides high slopes for the sewers which will eliminate low velocities in the initial stages.

c) From the economical point of view, there will not be appreciable savings due to short design period as there will already be sufficient reduction in the size of the sewers due to high sloping grounds.

d) Most of the materials which will be used in the work have a life span of over 50 years.

3.2. Source of Data

The primary source of population data are the past census records. Recording to the most recent census in 1957 the population of Sulaimaniya was 48,450. In an earlier census (1947) the population was given as 35,510. This increase corresponds to an annual growth of 3.2 percent. Table 3.1 shows the growth and distribution of population in Iraq, while Table 3.2. shows the percentile values together with the corresponding values for all of Iraq, the Liwa capitals (with the exception of Greater Baghdad) and Sulaimaniya Liwa (excluding the city of Sulaimaniya).

TABLE 3.1

GROWTH AND DISTRIBUTION OF POPULATION
IN IRAQ 1947 AND 1957[8]

Administrative Area	Population in 1947	Population in 1957	Percentage increase in 10 years period.
Baghdad	817,205	1,366,604	60
Mosul	595,190	717,500	20
Basra	368,799	502,884	37
Kirkuk	286,005	388,912	41
Sulaimaniya	226,400	299,978	32
Diala	272,493	329,813	21
Ramadi	192,983	234,262	22
Kut	224,938	290,070	29
Hilla	261,206	353,614	36
Kerbella	274,264	217,015	- 21
Amara	307,021	329,647	10
Diwaniya	378,118	507,548	34
Nasiriya	371,867	455,644	24
Arbil	239,776	272,526	19
Not registered Estimated	300,000	332,092	
Total Population of Iraq	4,816,185	6,538,109	36

TABLE 3.2.

PERCENTILE POPULATION INCREASE
1947 - 1957

	Total increase %	Annual increase %
Iraq (as a whole)	37.4	3.27
Liwa Capitals (average values)	51.4	4.30
Sulaimaniya Liwa	11.7	1.13
Sulaimaniya city	36.4	3.20

From Table 3.2. it is seen that Sulaimaniya has grown at a lower rate than the overall average rate of all the liwa capitals during the 10 year period between the two censuses. Also the population increase of Sulaimaniya liwa has been markedly less than the population increase in the entire country.

In 1924 Sulaimaniya had a population of about 20,000[8]. During the period between 1924 and 1947 (23 years) the city population should thus have increased by 13,500 (68 percent). This corresponds to an annual growth of 2.25 percent.

The 1947 census is not considered completely reliable for the following reasons:

a) The 1947 census was the first one to be performed under the Iraqi Government control. The personnel concerned were probably not experienced enough to carry out the necessary work satisfactory.

b) Most of the inhabitants of the arid regions of Iraq (Saharas) are nomads that move about between Syria, Jordan, Saudi Arabia and Iraq, and hence it was very difficult to include all of these tribes in the census. In the region concerned migration between Turkey, Persia, Syria and Iraq may have also affected the results of the census as there is a strong relationship between the Kurdish tribes of the above four countries.

c) Since military service is compulsory in Iraq, and due to the adverse feeling of the inhabitants at the time of the census towards this service, many provided false information inorder to escape eventual enrollment in the army. This is evident from the high increase in population registered by the 1957 census.

If the 1947 figure is 5 percent too low, the annual growth to 1957 has been 2.8 percent. If it is 10 percent too low, the annual growth has been 2.2 percent.

The census figure for 1957 is probably correct. In addition to permanent residents of the city, temporary residents, such as army personnel, hotel guests, etc..., have been included in this figure. It has been impossible to obtain any information concerning the size of the transient group.

In a study concerning the water requirement and water supply of Sulaimaniya, the population in 1961 of the city was estimated at 53,000 which, with regard to the rate of development, agrees well with the 1957 census figure.

The statistics were also taken for the live birth and death rate in the city, and are shown in Table 3.3 from which the annual growth for the Sulaimaniya Liwa is found to be 1.2 percent. This figure is low due to the fact that imigration was not taken into consideration. Table 3.4 shows the number of students in the schools from which the annual growth is found to be 4.8 percent. This figure is high due to the fact that education in Iraq, and especially in Sulaimaniya, is increasing at such a high rate which is not commensurate with the increase in population.

TABLE 3.3

NUMBER OF BIRTHS AND DEATHS
1952 - 1961 (8)

Year	No. of live births	No. of deaths	Increase
1952	1,058	361	697
1953	2,126	970	1,156
1954	1,867	659	1,208
1955	2,875	935	1,940
1956	4,031	717	3,314
1957	1,880	845	1,035
1958	1,639	848	791
1959	8,367	549	7,818
1960	6,192	710	5,482
1961	3,754	650	3,104
Total	33,789	7,244	26,545

TABLE 3.4.

NUMBER OF STUDENTS IN THE SCHOOLS
IN SULAIMANIYA LIWA (8)

Year	No. of boys	No. of girls	Total
1951	3,815	937	4,788
1952	4,237	1,132	5,369
1953	4,595	1,286	5,881
1954	5,396	1,459	6,855
1955	6,129	1,724	7,853
1956	8,130	2,304	10,434
1957	9,431	2,986	12,417
1958	10,118	2,901	13,019
1959	12,631	3,729	16,360
1960	15,704	5,061	20,765

3.3. Present Population

The present population of the city of Sulaimaniya is 57,000. The technique used in the estimation of the present population is discussed briefly in section 3.4.

3.4. Future Population

Prognosis of the future population development is a hazardous attempt. The importance of the city as the commercial and administrative centre of Sulaimaniya liwa will in all probability increase. Lately Sulaimaniya has acquired some importance as an industrial city, which will probably be increased with present and future industrial expansion within the country. The comparatively rapid increase in the standard of living in Iraq, which must be taken into consideration, will probably be attended with some suppression of the population increase. An annual population increase of 2.5 percent is probably plausible during the 50 year period following the 1957 census, i.e., until the year 2015.

Before starting with the discussion on the future population, the following assumptions are implicit(7):

a) No change in political, economic and social organization.

b) No war or internal revolution.

c) No large scale epidemic, earthquake, fire or other disaster will occur within the area or the adjacent areas.

There are many forecasting methods for estimating the present and future population, each of which requires specific and sufficient information which is not available in the present circumstances. V.B. Stanbery suggests that, in the absence of the necessary data and analysis, an arithmetic projection might be used as probable minimum forecast, and geometric projection might be used as a maximum figure[7]. The use of any of the above methods assumes that the population growth has followed some identifiable mathematical relationship in which population change is a function of time, and future changes in population will follow a pattern predictable from this relationship. These methods, therefore, do not lead to an accurate estimation for the future population. They have been modified according to Verhulst's theory which is based on the principle that the population of any area will increase to a terminal number that might be called the saturation limit. The determination of this limit is based on the logistic curve of Peral[6]. Accordingly, the rate of increase in population as reaching the future limit has been slowed down by assuming that the decline of population will start after 20 years from now, i.e., in the year 1985.

Two curves for population growth are represented in Figure 3.1. From curve "A" the future population can be calculated to the values given in Table 3.5. and from curve "B" the future population can be calculated to the values given in Table 3.6.

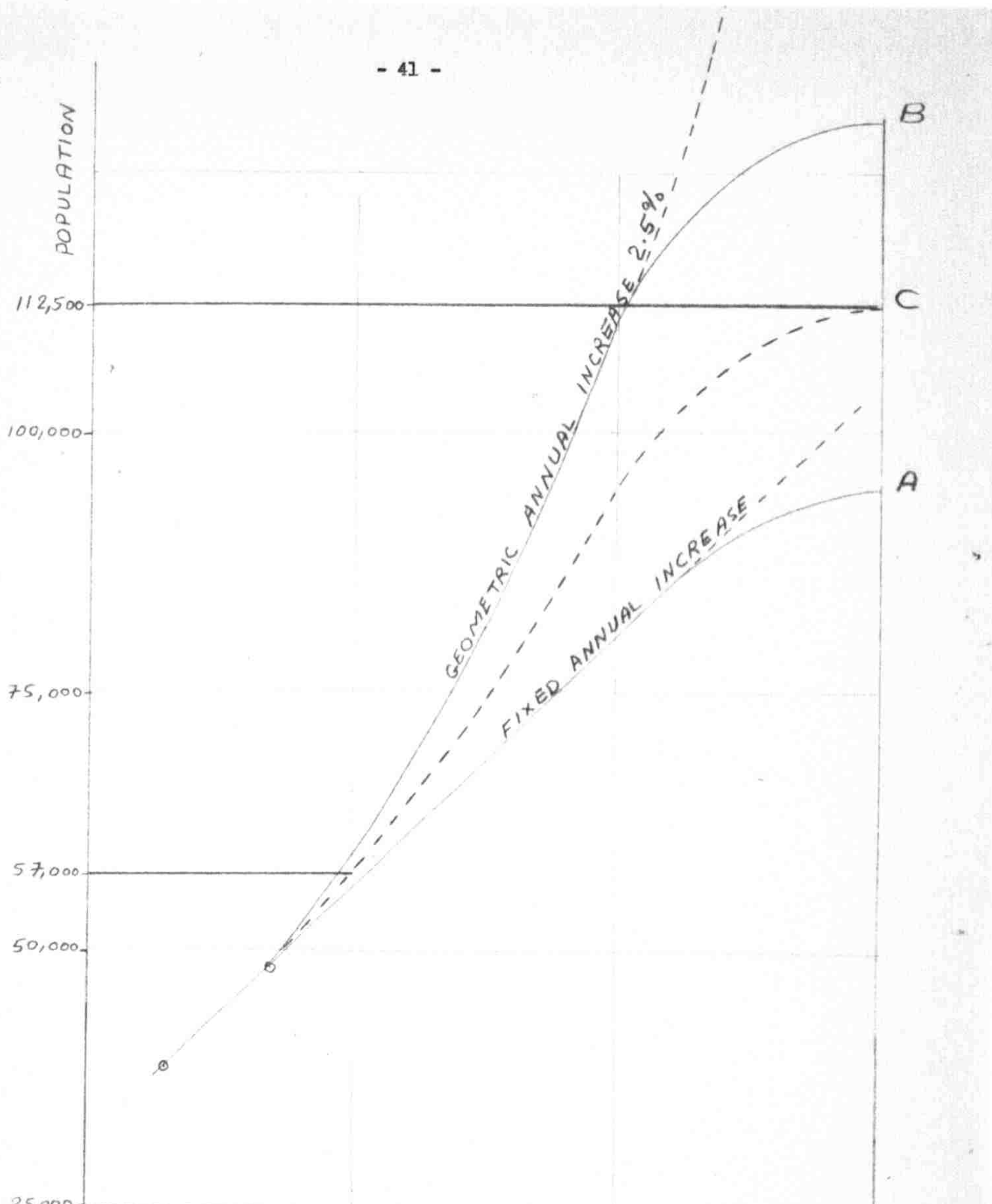

FIG. 3.1

POPULATION GROWTH CURVES

TABLE 3.5.

FUTURE POPULATION ACCORDING TO
ARITHMETIC PROJECTION

Year	Population	Year	Population
1965	56,000	1995	85,000
1975	65,700	2005	92,000
1985	75,200	2015	95,000

TABLE 3.6.

FUTURE POPULATION ACCORDING TO
GEOMETRIC POPULATION

Year	Population	Year	Population
1965	59,200	1995	118,500
1975	76,000	2005	127,100
1985	97,400	2015	130,000

It is seen that if the linear increase method is depended upon, the population in 50 years would be 95,000, whereas if the compounds rate is used, the population during the same period would reach about 130,000. An average figure of 112,500 persons in the year 2015 will be justifiably used as a basis for the design. Consequently, curve C, which has

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TABLE 3.7.

FUTURE POPULATION ACCORDING TO CURVE C

Year	Population	Year	Population
1965	57,000	1995	101,000
1975	71,000	2005	109,000
1985	86,000	2015	112,500

3.5. Direction of Development

3.5.1 Present Direction

Sulaimaniya has developed from its original centre, the village of Malik Kendi situated in the northeastern part of the present city, in a southwesterly direction. The city, therefore, can be assumed to have acquired a long narrow form in the beginning, in a northeasterly - southwesterly direction with Malik Kendi in the northeastern part. From this original form the city has developed towards the northwest and southwest to its present form.

At present the building expansion is taking place mainly to the west and the north. The new highway to the west towards Kirkuk has been of decisive importance for the developmental tendency towards the west. Relatively new flood control canals north of the city, constructed to divert excess surface water from the mountain chain in the northeast, have attracted new settlements in a northly direction.

A rather significant settlement in the eastern part of the city has taken place since an older flood control canal east of the city has been filled in and substituted with a new one located about 500 m east of the old canal and largely parallel to it. The area between these two canals is planned for settlement and new building construction is in progress. A certain construction has also taken place east of the new canal. It is probable that this development will continue.

Developmental possibilities south of the city are largely limited by Wadi Walobah. South of this stream the terrain is slightly hilly which makes it unsuitable for settlement. However, in recent years two residential areas have been developed south of Wadi Walobah. It is improbable that any more marked development of this area will take place.

3.5.2 Future Direction

The future development of the city is dependent principally on the availability of suitable land for construction purposes. The development is further dependent on such factors as closely situated communities, thoroughfares and administrative and commercial connections with the outside. There are suitable areas for construction primarily to the north and northwest of the city. Likewise there are large parts of the areas in a southwesterly direction which can be utilized for construction. The development in a southerly direction is largely limited to Wadi Walobah. To the south the topography is such that construction is not suitable. In a northerly direction development can take place largely towards the existing surface water canals, and in the east towards the new easterly canal and also to a certain extent east therefrom.

The new highway to Kirkuk will be of great importance for future settlement development. The new road skirting the city to the southwest, which connects the Kirkuk road with the highway from Sulaimaniya to Baghdad via Derbendi Khan, will stimulate a settlement development in a southwesterly direction. The new Baghdad highway in the south, however, will probably not have any great importance for the developmental direction of the city.

The development of the city will probably take place in westerly, northwesterly and southwesterly directions. Limited development may also take place to the east and to the north. The military base in the northwestern part of the city will probably, in the light of development plans, be moved to a more suitable area outside the city.

3.6. Population Densities

3.6.1 Present Population Densities

The city of Sulaimaniya has different population densities in different areas. The centre of the city is heavily populated in contrast to the peripheral areas. The estimated present population densities are given in Table 3.8, and shown in Drawing No. 2.

TABLE 3.8.

PRESENT POPULATION DENSITIES

Class	Area in acres	Est. population density (person/acre)	Numbers of persons
I	353.8	81	28,600
II	106.5	70	7,450
III	181.8	50	9,080
IV	454.1	26	11,870

3.6.2 Future Population Densities

Population densities in the year 2015 can be estimated according to the availability of suitable land for construction purposes and according to the Iraqi law of 1954, which classified the developmental areas into five classes, with the average size of building sites being 150 m^2, 250 m^2, 450 m^2, 800 m^2 and 1000 m^2 in the respective classes. The areas for public roads, parks and public buildings, according to the above law, are 40 percent of the total area for class I, II and III, and 35 percent of the total area for class IV and V.

Population densities for the year 2015 are calculated to the values given in Table 3.9, and are shown on Drawing No. 3.

TABLE 3.9

POPULATION DENSITIES FOR THE YEAR 2015

Class	Area in acres	Population density (persons/acre)	No. of Persons
I	360	80	28,800
II	365	58	21,170
III	760	33	25,080
IV	1,110	25	27,450
V	1,250	8	10,000
Total	3,845		112,500

4 DIMENSIONING DATA FOR PLANNING OF SEWERAGE.

4.1. Municipal Waste

4.1.1 Volume

One of the most important dimensioning date for planning of municipal sewerage system is the amount of waste water which requires consideration and determination of the following:

a) Domestic Sewage

The amount of domestic sewage is usually less than the per capita water consumption because of the water used for sprinkling, irrigation, leakage, etc.. The amount of domestic sewage usually varies between 70 to 80 percent of the amount of consumption(14). It is assumed that 70 percent of the water consumption reaches the sewers as there are many open areas in the city of Sulaimaniya where the water will be used for sprinkling and irrigation. The amount of domestic sewage, therefore, will be:

$$0.7 \times 80 = 56 \text{ gallons per capita per day.}$$

b) Commercial Sewage

Commercial areas like stores, hotels, offices, hospitals and schools also contribute waste water. The amount of waste water in such areas is usually more than that of the residential areas. Commercial establishments in the city of Sulaimaniya are scattered all over the city; however, most of them are located in the old district.

As the city of Sulaimaniya is relatively small it is, therefore, assumed that contribution of waste water by commercial areas is amply cared for in the peak allowance for per capita sewage flows[11].

c) Ground Water Inilitration

Water that enters sewers through poor joints, cracks, walls of manholes, perforated manhole covers and drains from flooded cellers is described as infiltration water. The cost of pumping and treating sewage obviously increases as the quantities delivered to the pumps or treatment facilities become greater because of inflitration water. It is, therefore, very important to keep the amount of infiltration water to a minimum by all possible means.

The amount of infiltration water to be expected will depend upon the following factors:

(1) The height of the ground water table with respect to the level of the sewers.

b) Commercial Sewage

Commercial areas like stores, hotels, offices, hospitals and schools also contribute waste water. The amount of waste water in such areas is usually more than that of the residential areas. Commercial establishments in the city of Sulaimaniya are scattered all over the city; however, most of them are located in the old district.

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The amount of infiltration water to be expected will depend upon the following factors:

(1) The height of the ground water table with respect to the level of the sewers.

(2) Porous subsoil conditions.

(3) Type of joints.

(4) Type of pipe material.

(5) Inspection and workmanship in private property construction such as house connections.

(6) Provision to prevent cracking of sewers.

(7) Sewer size - the large sewers present more joint length for leakage.

There is no accepted standard method for the evaluation of the rate of infiltration to be used for specifications and design. However, various criteria and allowances have been used including[14]:

(1) Per mile of sewer: 2000 to 200,000 gpd.

(2) Per inch-diameter mile of sewage: 500 to 2000 gpd.

(3) Per capita: 25 to 200 gpd.

(4) Per acre of sewered area: 300 to 1500 gpd.

The level of water table in Sulaimaniya and neighbouring region is so low that the ground water is of no importance to the contribution of infiltration. Moreover, the percolation of the water into the ground will be very little due to steep slopes. Minimizing infiltration requires the use of water tightness of joints, careful inspection and good workmanship of house connections, as well as adequate inspection of the construction of public sewers.

In spite of all measures adopted, sanitary sewers have to be designed to carry the unavoidable amounts of infiltration water. Allowance for this has been made in the capacity of the sewers at the rate of 25 gallons per capita per 24 hours.

The total average flow will, therefore, be based on the following:

(a)	Domestic sewage	56 gpd
(b)	Infiltration	25 bpd
		81 gpd.

This figure is almost equal to the per capita water consumption and, therefore, a rate of the average flow of 80 gallons per capita per 24 hours has been adopted in the design.

4.1.2 Strength of Sewage

It has not yet been possible to carry out investigations concerning the amount of pollution in the municipal waste water. However, there is reason to believe that future pollution may reach a value similar to that found at present in the waste water of European cities.

An anticipated amount of future pollution of 0.17 pound BOD per capita per 24 hours is thus probable. The total amount of future pollution can thus be estimated to be:

$$0.17 \times 112{,}500 = 19{,}100 \text{ pounds BOD per 24 hours.}$$

4.2. Industrial Waste

4.2.1 Wastes at Present

Although Sulaimaniya is not an industrial city, the limited number of small industries includes a cigarette factory, a slaughterhouse and a few tanneries.

The volume and strength of the liquid wastes produced by these factories is computed hereunder:

a) Cigarette Factory:

The cigarette factory employes 200 men and has a daily production of 4 million cigarettes. The liquid waste produced is not significant and is mostly derived from the domestic activities occurring at the factory. Contribution of the small amount of industrial waste is attributed to the following processes[15]:

(1) Tobacco is allowed to ferment subsequent to steam treatment which produces a liquid waste containing nicotine.

(2) Floor drainings which contain grease.

(3) Water used to wash machines.

Volume of liquid waste per sheep[19] = 4 gallons

Volume of liquid waste water from 140 sheep = 4 x 140

= 560 gallons

Volume of liquid waste per cow[19] = 360 gallons

Volume of waste water from cows = 360 x 10

= 3,600 gallons

Total volume of waste from tannery = 3600 + 560

= 4,160 gallons

The strength of the waste water is considered to be 1500 ppm. BOD[16].

The total pollution from the tannery

$$= \frac{1500 \times 8.34 \times 4160}{10^6}$$

= 52 lbs. of BOD.

The total daily volume of waste water contributed by all industries in the city = 53,850 + 4160

= 58,010 gallons

The total amount of pollution per day = 52 + 298

= 350 pounds of BOD.

Industrial waste water in the city of Sulaimaniya does not need special treatment because:

(1) The amount of waste liquid contributes less than 1% of the domestic wastes.

(2) The pollutional effect of slaughter house wastes are similar to those of domestic sewage. Tannery wastes in small quantities have no significant effect on the sewage strength or volume, and do not harm the sewer material or the operation of sewage treatment plant (16).

4.2.2 Future Industrial Waste

It is very difficult to foresee the type and size of future industries in Sulaimaniya. The dimensioning future population has been estimated to be 112,500. In future larger industries of importance from the point of view of volume and pollution are expected to be established. Hence, a value of 5,000 is given as a rough estimate of the population equivalent which corresponds to less than 5 percent of the estimated population of the year 2015. This estimate is based upon the following facts:

a) Sulaimaniya Liwa is an important agricultural region but is not industrialized.

b) The Iraqi Government has no plan for establishing large governmental industries in the city (4).

c) The comparatively rapid increase in the standard of living in Iraq which would require further development of industry in Sulaimaniya.

The volume of industrial waste in the year 2015 = 400,000 gpd.

The amount of pollution in the year 2015 = 850 lb. of BOD.

4.3. Storm Water

4.3.1 Intensity of Rainfall

There are no statistics available concerning the relation between the intensity, duration and frequency of rainfall in Sulaimaniya. Nor is there any official meteorological station where such observations are made in the city. On the other hand, observations of rainfall are made at a nearby experimental farm - Bakrajo Farm. Automatic recording of rainfall has been carried out since 1959. With the guidance of the data from this source a relation between the intensity, duration and frequency has been calculated and the results are shown in Table 4.1 and presented graphically in Figure 4.1.[4].

TABLE 4.1

RELATION BETWEEN INTENSITY, DURATION AND FREQUENCY OF RAINFALL [4]
(inch/hour)

Frequency (n)	Duration (min.)				
	5	10	15	60	120
1.0	1.10	0.90	0.64	0.43	0.29
0.5	1.25	1.01	0.71	0.49	0.33
0.2	1.30	1.07	0.75	0.54	0.39

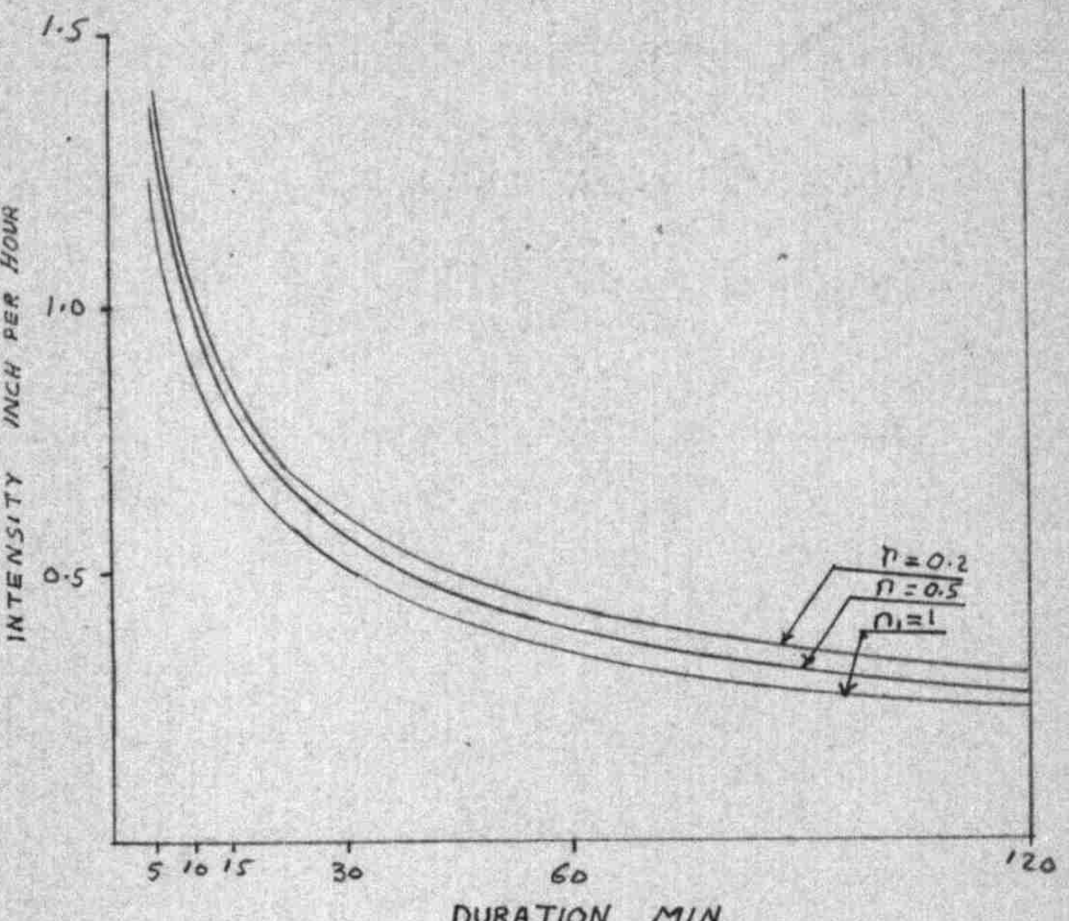
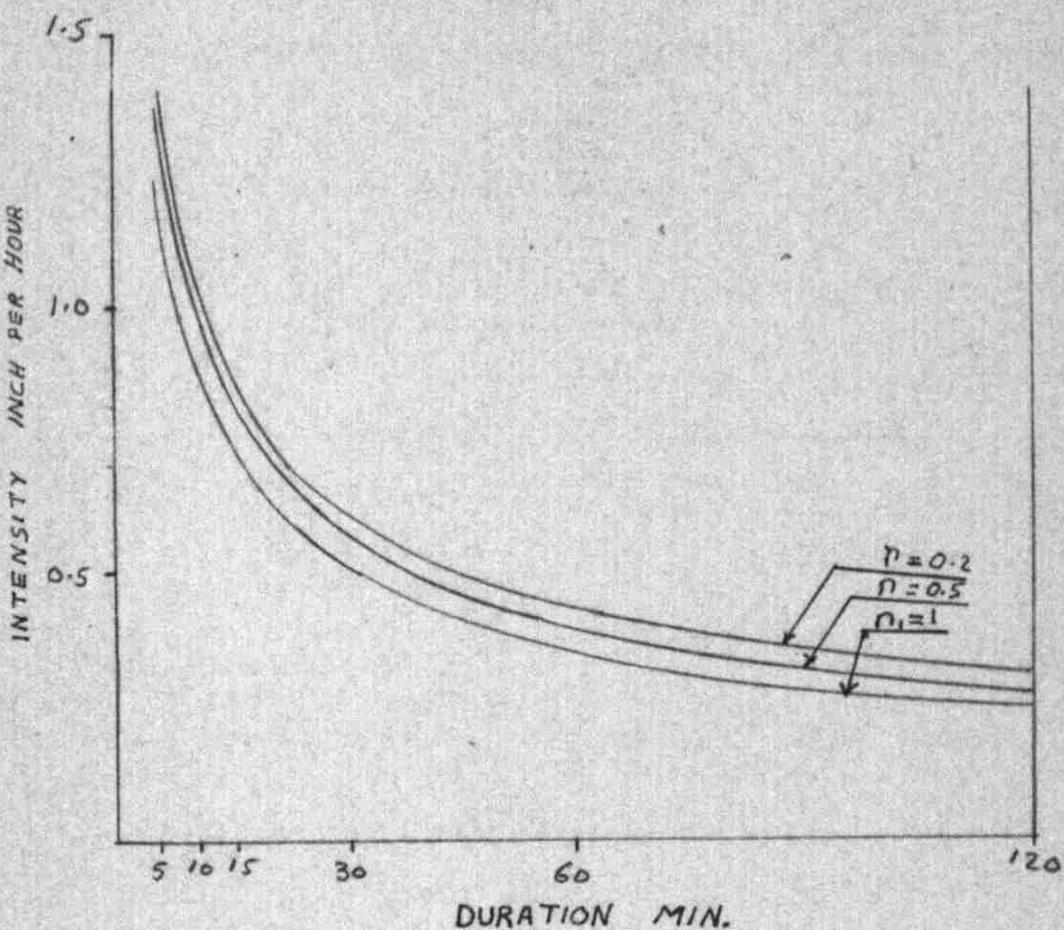


FIG. 4.1

INTENSITY OF RAINFALL



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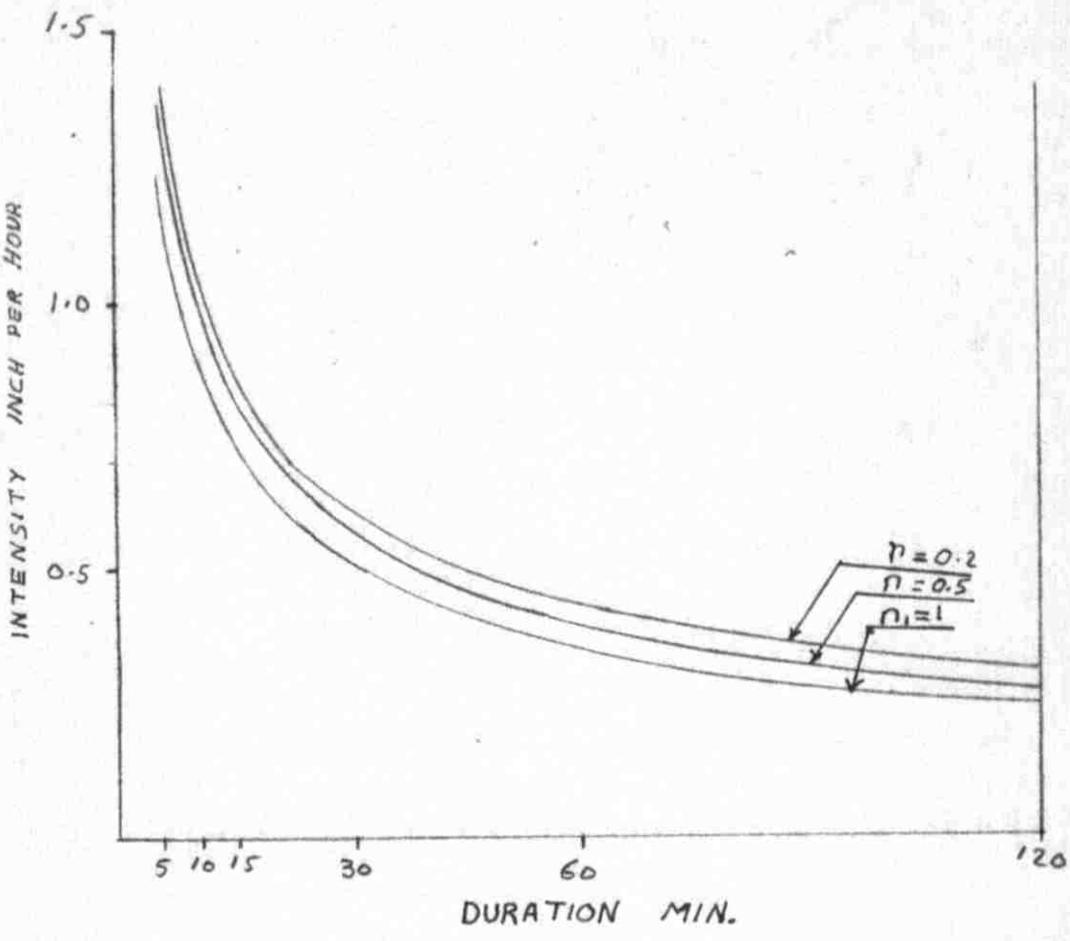

FIG. 4.1

INTENSITY OF RAINFALL

It should be observed that these values have been calculated from material unsatisfactory from a statistical point of view. It is important that after further observations have been made a new statistical treatment be carried out to obtain more reliable values. Those given here have, however, been recommended for the dimensioning of the system of storm sewers in the near future as new street will be open.

4.3.2 Values of Relative Imperviousness

Only a part of the rainfall flows off the ground in the form of storm water. The rest evaporates or is infiltrated into the soil. The magnitude of the part that flows off in the form of storm water is indicated by the so-called value of relative imperviousness, which is dependent on the following factors:

a) Vegetation in the area.

b) Building density of the area.

c) Topography of the area.

d) Extent of hard surfaces.

In Table 4.2 are given some values of relative imperviousness for different types of surfaces(17).

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In Table 4.2 are given some values of relative imperviousness for different types of surfaces[17].

TABLE 4.2

VALUES OF RELATIVE IMPERVIOUSNESS[17]

Description of area	Relative Imperviousness
Roof surface assumed to be watertight	0.7 - 0.95
Asphalt pavements in good order	0.85 - 0.90
Stone, brick, and wood-block pavements with tightly cemented joints	0.75 - 0.85
Ditto, with open or uncemented joints	0.5 - 0.70
Inferior block pavements with open joints	0.4 - 0.50
Macadamized roadways	0.25 - 0.60
Gravel roadways and walks	0.15 - 0.30
Unpaved surfaces, railroad yards, and vacant lots	0.10 - 0.30
Parks, gardens, lawns, and meadows - depending on surface slope and characteristics of subsoil	0.05 - 0.25
Wooded areas of forest land - depending on surface slope and characteristics of subsoil	0.01 - 0.20
Most densely populated or built-up portion of a city	0.70 - 0.90

For the dimensioning of the pipelines an average value of relative imperviousness of 0.80 has been recommended. This high value has been chosen in view of the strongly sloping terrain within the region. Furthermore, it is reckoned with the fact that the unpaved streets in the city will be paved in the near future and that the new exploitation areas will be built with hard-surfaced streets. In the detailed planning of the pipeline system a determination of the size of the value of relative imperviousness within each part of the area should be undertaken, thus giving consideration to local variations with regard to vegetation, topography, hard surfaces, etc..

The above dimensioning data for storm sewers is given here as a guide for the designer of new storm sewers in the future and especially after the planning of the city.

5 PROPOSED SEWERAGE SYSTEM

5.1. Choice of Sewerage System

One of the most important problems to be considered when planning a sewage disposal scheme is whether to use a combined system for sewage and storm water, or to use a separate system for each.

The deciding factor in the choice of the system is the economy and the volume and intensity of rainfall.

a) Economy

Combined sewers may be less costly to construct than separate sanitary and storm sewers because of the requirement of one sewer which usually needs to be very little larger than that required for storm flow. Savings due to less costly initial construction should, however, be weighed against probable increase in future costs for separation, pumping, and treatment. Initial savings may well prove to be a false economy.

b) Volume and Intensity of Rainfall

In some cases the total annual rainfall does not exceed a few inches, but this amount may be concentrated in a few heavy storms yielding appreciable runoff. The runoff may by far exceed the domestic sewage flow. Hence, if a combined system is used, the required diameter of the sewers should be relatively large to drain both the domestic sewage and the storm water. In dry weather the flow would consist of the sewage only. This would result in shallow depths of flow and small velocities accompanied by septicity of sewage and clogging of sewers.

In the light of the preceeding discussion and on the basis of the following reasons, a separate system is proposed for Sulaimaniya:

a) Since Sulaimaniya is located on a number of hills which slope towards the valleys (wadis), it will be more economical to let all surface water run through short and small storm sewers to the nearest wadi. The use of a combined system means that the long pipeline to the sewage treatment plant should have dimensions larger than necessary.

b) The separate system does away with the problem of surface treatment resulting in savings in the initial and operational cost of the treatment plant.

c) The existing combined system in the newly built district, which cannot be made available for the proper conveyance of sewage, can yet be used for storm water as the municipality is incapable of providing funds for the construction of new storm water sewers.

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c) The existing combined system in the newly built district, which cannot be made available for the proper conveyance of sewage, can yet be used for storm water as the municipality is incapable of providing funds for the construction of new storm water sewers.

d) The street-side gutters can take care of some of the storm water flow with savings in pipe length and diameter.

e) In the separate system the diameter of the sewage sewers are small enough to facilitate self-cleansing at peak flows unlike the combined sewers which should be large enough to accomodate storm water and have gradients which produce self-cleansing velocities at times of storm only,

5.2. Pipe Material

Some of the important factors that affect the selection of pipe materials are:

a) Type of liquid to be conveyed.

b) Durability.

c) Resistance to erosion.

d) Weight of the material.

e) Resistance to corrosion.

f) Resistance to structural failure.

g) Cost of material, handling and installation.

h) Type of joint-watertightness and ease of assembly.

i) Availability of sizes required.

j) Site condition.

k) Local material.

l) Maintenance cost.

It is evident that no one material can possibly fulfill all the above requirements. It is, therefore, necessary to balance between the advantages and disadvantages of different types of pipes to decide on any one of them. The local availability would probably be a decisive factor in the choice as customs on imported goods would be high.

Types of pipes usually are described hereunder:

a) Vitrified-clay pipes:

Vitrified clay pipes have the advantage of high resistance to crown corrosion and to agressive constituents of sewage and ground water. They also possess good hydraulic properties due to their smooth and impervious inner surfaces. The main disadvantage of such pipes, however, is their liability to damage in transit and handling due to their brittleness. Their cost is also high due to the fact that they are not available locally. It is evident, therefore, that such pipes are not suitable for use as sewers for Sulaimaniya city.

b) Bituminous fiber pipes:

Bituminous fiber pipes are made of wood fiber impregranted with coaltar-pitch. Among the advantages claimed for them are: lightness, ease of construction, water-tightness, resistance to absorption, tightness of joints, high crushing.strength, durability, resistance to corrosion and to

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corrosive chemicals, and flexibility or plastic flow. The disadvantages of such pipes, however, are: if the pipes are permanently laid on the ground surface exposed to direct sunlight deterioration may occur; the pipes are available in sizes only between 2 and 8 inches in diameter and they are not available locally. It is evident, therefore, that such pipes are not suitable for use as sewers for Sulaimaniya.

c) Cast-iron pipes:

The advantages of the cast-iron pipes are: absolute water-tightness, high resistance to structural failure and availability in all diameters and lengths. These pipes are not available locally and are costly. Moreover, they are subject to corrosion. These pipes, therefore, are not suitable for sewer construction.

d) Asbestos cement pipes:

Among the advantages claimed for asbestos cement pipes for sewers are: lightness, long sections to minimize the number of joints and maintain alignment; easy cutting and fitting as asbestos cement pipes can be cut with a wood saw; locally manufactured and comparatively cheap. These pipes, however, are subject to crown corrosion and agressive ground water. Asbestos cement pipes can, therefore, be used for sewers construction in this project.

e) Cement Concrete pipes:

Cement concrete pipes are used extensively for the construction of sewers. They are proposed for the construction of sewers for Sulaimaniya because of the following advantages:

(1) Cement concrete pipes are the cheapest available pipes in Iraq.

(2) No transportation is required as the cement factory is located just outside the municipal boundaries.

(3) The availability of a wide range of sizes.

(4) Ease of manufacture on site.

(5) The availability of many types of joint designs depending on the degree of water tightness required.

(6) Relative ease in obtaining the required strength.

(7) Long laying lengths and the rapidity of opening and backfilling of trenches.

(8) Concrete pipes have a self-healing property.

(9) Crown corrosion of sewers, which takes place under anaerobic conditions, can be prevented by adequate ventilation and proper slopes.

(10) Errosion of sewers is not expected as long as the velocity of flow remains within the allowable range. However, with excessive velocities protective lining of clay linear blocks will have to be provided.

5.3. Design Criteria

The purpose of any sewer system is to convey the sewage or storm water from its many origins to its point of disposal in the most economical manner. In a sense, the sewer system is a means of conveyance aimed at achieving the best utilization of the available energy which is inherent in the flowing liquid. In order to achieve this purpose proper design of the sewer system is necessary. Among the objectives of such a design is the determination of the size, slope and depth of the sewers.

5.3.1 Maximum and Minimum Flow of the Sewage

The maximum and minimum rate of sewage flow are the controlling factors in the design of sewers. The sewers must be of sufficient capacity to carry the maximum flow and they must have such a grade as to prevent sediment deposition and crown corrosion, especially during periods of minimum flow.

The amount of sewage is a variable factor and depends upon the hour of the day, the day of the week, and the season of the year, with extreme low flows usually occuring between 2 and 6 a.m. in winter, and peak flows occuring during the daylight hours in summer.

Since the daily or hourly fluctuation in rate of water consumption is not available, the estimates of the peak flow or the minimum flow can not be made. In such a case it is of common practice to have the following criteria for the maximum and minimum flows(11):

Peak flow in laterals	4 x average daily flow
Peak flow in main and trunk sewers	2.5 x average daily flow
Minimum flow in all sewers	0.5 x average daily flow

5.3.2 Velocities of Sewage Flow in Sewers

a) Minimum Velocity:

The minimum velocity of flow in sewer should be sufficient to prevent the deposition of suspended solids which may damage the sewer material as a result of decomposition of the organic mater in the depositions. It is common practice to use a minimum velocity, when flowing full, of 2 feet per second for sanitary sewers and 3 feet per second for storm drains. However , it is recommended that the velocity should be increased to a value of 3 feet per second where the sewage is strong and the temperature is relatively high(11).

Since the topography of the area is favorable to allow steep slopes, a minimum velocity of 2.5 feet per second will be adopted.

b) Maximum Velocity:

The maximum velocity is determined from a consideration of the ero ion which may be caused by the grit or other inorganic solids which are transported along the invert at high speed. For this reason, it is usual practice to limit velocities in sewers flowing full to not more than 10 feet per second[11].

5.3.3 Slope of Sewers

In practical design of a sewer, efforts should be 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 basic principle will be adopted in the design of the sewers and, hence, there will be no fixed limits for maximum and minimum slopes, provided the maximum and minimum velocities are within the allowable limits[11].

5.3.4 Type of Sewer

Circular section of the sewer will be adopted for the following reasons:

a) Circular shape gives maximum cross sectional area for the amount of material in the wall.

b) Circular pipes have excellent hydraulic properties.

c) Convenience in manufacturing of pre-cast pipes.

d) Fairly stable in place.

5.3.5 Size of Sewer

a) Minimum Size:

The adoption of a minimum size is necessary because experience has shown that some comparatively large objects such as scrubbing brushes, for instance, often get into sewers and the resulting stoppage is less likely if sewers larger than 8 inches are used[18].

A minimum size of 8 inches will, therefore, be adopted in the design of sewers.

b) Maximum Size:

No maximum size for a sewer has even been laid down, but it has been suggested that diameters exceeding 15 to 16 feet would not be economical to construct[13]. However, such a diameter is not likely to be utilized in this project.

5.3.6 Depth of Sewer

The depth of sewers must be such as to provide an adequate outlet for all connections. Since the houses have no basement a minimum depth of 4 feet (1.25 m) below street surface to the top of sewer is sufficient and will be adopted.

Summary of Design Criteria:

The following design criteria will be adopted:

a) Maximum flow of laterals and submain

= 4 x average daily flow.

b) Minimum flow of mains and trunks

= 2.5 x average daily flow.

c) Absolute minimum flow = 0.5 x average daily flow.

d) Minimum velocity flowing full

= 2.5 feet per second.

e) Maximum vlocity = 10 feet per second.

f) The cross-section of sewers will be circular.

g) Absolute minimum diameter of sewers

= 8 inches.

h) Minimum depth below street surfact to the top of sewers

= 4 feet.

Other criteria include:

i) Invert of laterals at a junction should be above those of the main by at least half the difference in diameter of small pipes and three-fourth the difference in the diameter of large pipes.

j) Manholes will be provided at all changes in grade, direction and size, at street junctions, at all the upper ends of sewers, at intermediate points and at spacings preferably not more than 250 feet (75 meter).

5.4. Embedment of Sewers

Since sewers are ordinarily burried underground they would be subjected to loads due to backfill and external sub-charges of the moving traffic. In the project under considera-tion all pipes are to be of relatively small diameters and of concrete. When properly laid, such pipe is sufficiently strong to bear the pressures exerted by the above mentioned loads. Since the load transmitted to the pipe is proportional to the width of trench, it has been recommended by Hardenburgh that this width should not exceed $1\frac{1}{2}$ times the outside diameter of the pipe[10].

The method of laying has much to do with the strength developed by the pipe. The cheapest method, obviously, is to lay the pipe directly on the bottom of the trench[9]. But this method is undesirable as it adds less supporting strength to the pipe than other methods. In Fig. 5.1 this method is designated as Type 1. Type 2 has been widely recommended, but the hand labour required for shaping the bottom makes the method compare unfavourably in cost to Type 3. Type 4 has a higher load factor, but the increase in pipe strength does not justify the higher cost of this type of bedding. Types 5 and 6 are to be used with soft soils that do not give the necessary support for pipes. It is believed that the soil conditions in the city

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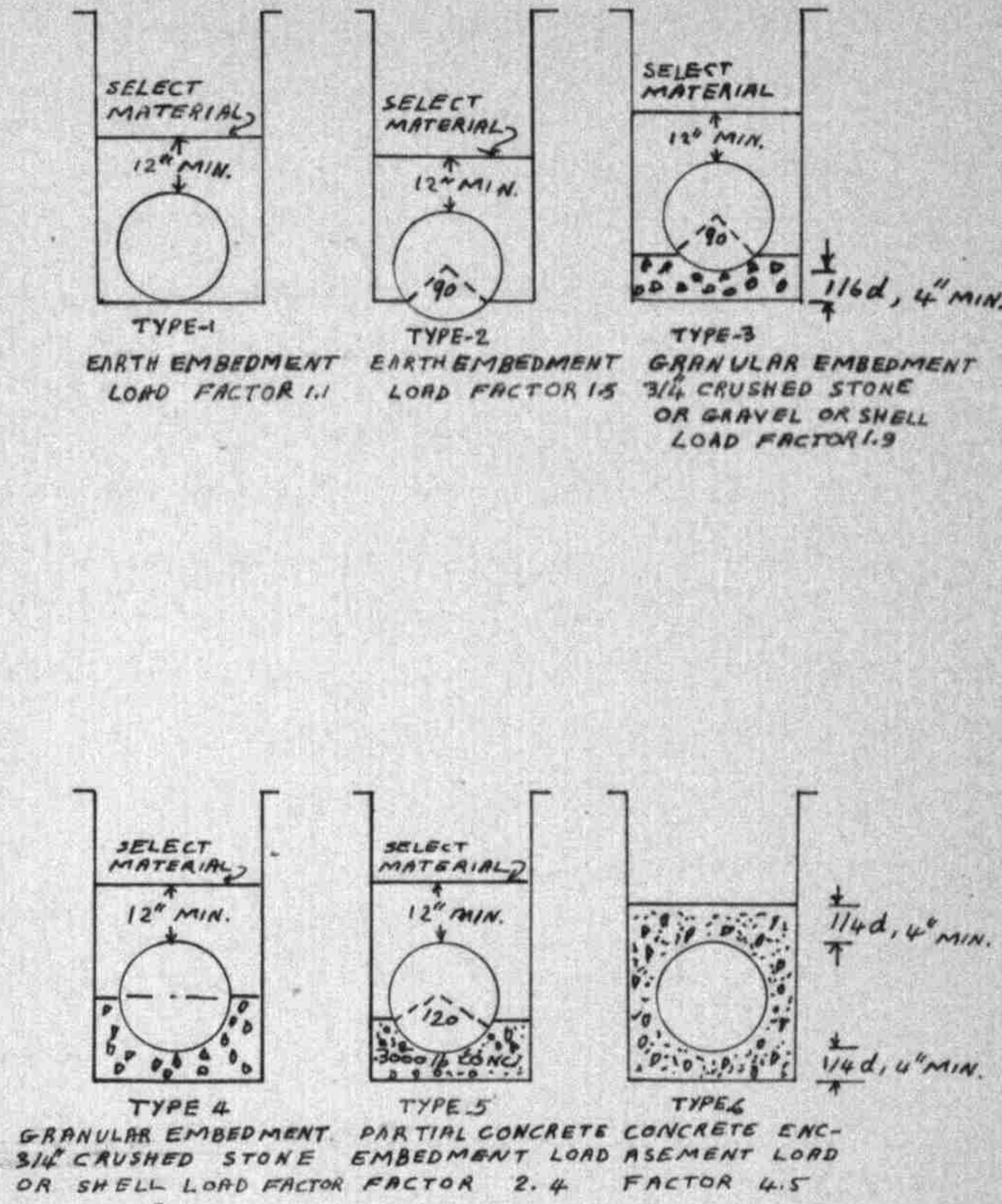

FIG. 5.1

EMBEDMENT OF SEWERS

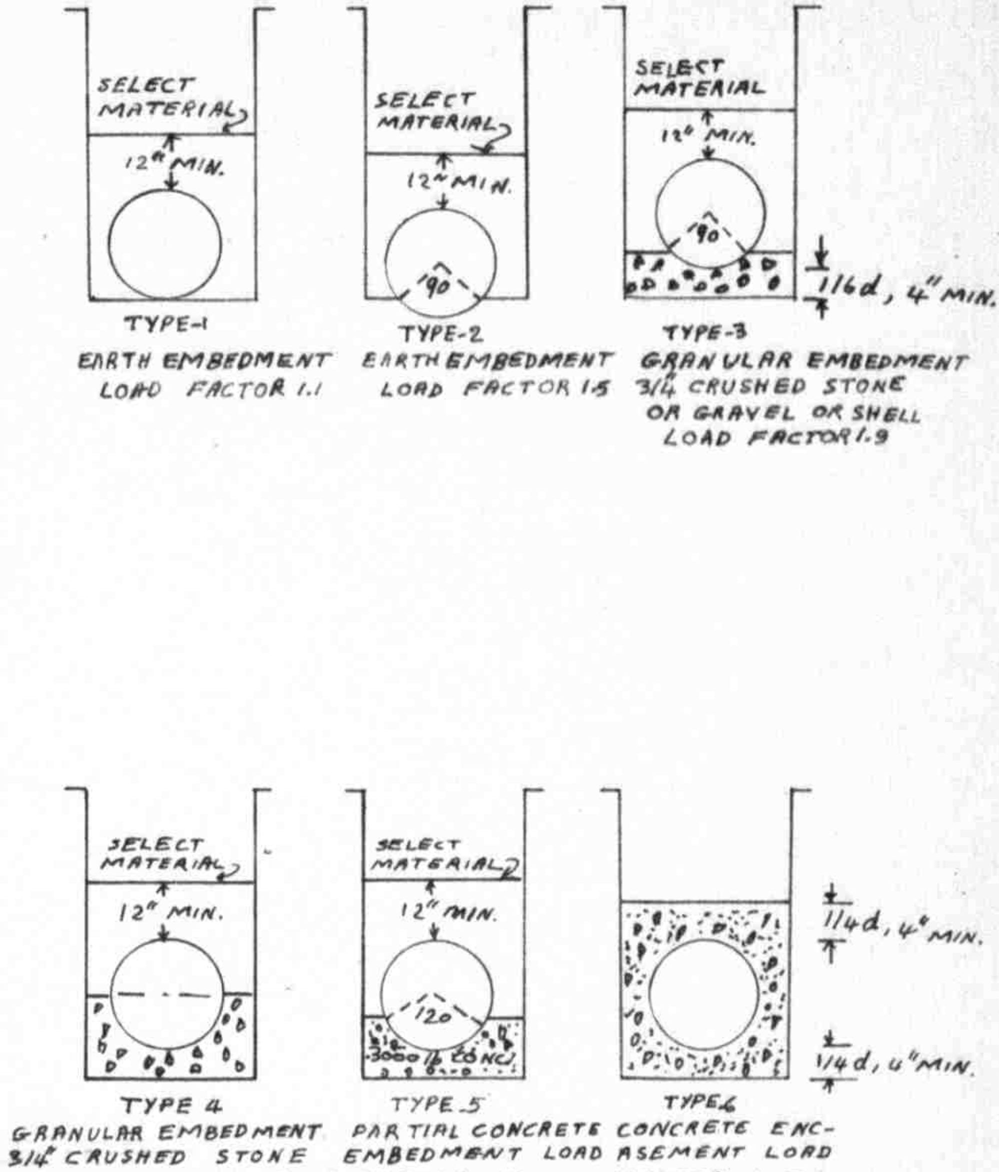

FIG. 5.1

EMBEDMENT OF SEWERS

of Sulaimaniya will permit the use of Type 3 bedding.

The granular material to be used may be either crushed stone or pea gravel which will pass a half-inch sieve, but will be retained on a No. 4 sieve. As it is presumed that crushed stone will be more available and less expensive than gravel stone, such material is recommended for use. The depth of the layer of granular material below the sewer should not be less than 1/6 the pipe diameter or 4 in., whichever is the larger[9]. The remainder of the side fills and a minimum depth of 12 inches over the top of the pipe should be filled with carefully compacted select material. This select material should be either earth or sand which can be placed in 6 in. layers and tampered. This will equally distribute the load on the pipe sewer and allow its transmission to the bed of trench without causing any great stresses to be developed in the pipes that may cause their breakage.

5.5. Manholes

Manholes are probably the most common and most necessary of all sewer appurtenances. They are used as a means of access for inspection and cleaning.

Manholes should be placed at all points where there is a change in alignment or size of sewer pipes, where there is an appreciable change in grade, at all junctions, at the upper ends of all laterlas and at intermediate points where the spacing between two consecutive manholes exceeds 250 ft.

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Manholes will be provided with a cast iron frame and cover with a clear opening of 22 in.. This opening will either be placed at the centre or side of the manhole, depending on the diameter of the latter. Inorder to prevent the entrance of rain water, the cover should be solid without perforations or openings. It should also be roughened inorder not to present a smooth surface that may cause slipping. The top of the cover should also be constructed flush with the finished road surface inorder not to hamper the passage of vehicles.

The manhole will be circular in shape inorder to achieve better stability and greater economy in construction. The minimum inside diameter will be 4 ft. which is sufficient for a man to perform inspection and cleaning easily.

The material that will be used in the construction of manholes is concrete. It is believed that this material will serve the purpose very satisfactorily whether from the point of view of structural strength or economy. The thickness of the wall will depend on the depth up to 6 ft.; the thickness is to be made 6 in. and to increase it to 8 in. for greater depths.

The bottom of the manhole is to be made of an 8-in. concrete slab which is slightly sloped on the upper surface toward the open channel constituting a continuation of the sewer pipes. The channel depth should be made at least equal to the diameter of the outgoing pipe inorder to prevent sewage from spreading over the manhole bottom.

Steps are also to be provided, set vertically at about 15 in. apart, in order to facilitate entrance to and exist from the manhole. They are to be made of tardipped cast-iron so as to be resistant to corrosion which, otherwise, would proceed quite rapidly in the moist, gas-laden air of the sewer. These steps should be firmly anchored to the wall in order to prevent any accidents.

5.6. Design of Sewers

5.6.1 Design of Storm Water Sewer

The division of the future city into catchment areas, according to Drawing No. 4, is also valid with regard to the drainage of storm water. Within most of these catchment areas it is proposed, as previously mentioned, that there is no need for a new storm sewer system.

It is proposed that storm water mains in future should be dimensioned for one year's rainfall, i.e., for intensities of rainfall occurring once a year.

The present city is protected by flood control canals from storm water flowing from the mountains north and east of the city. These canals are presumed to be so dimensioned that they can receive and drain all storm water thereby preventing it from reaching the sewer system.

It is proposed that the existing open water courses (wadis) remain unchanged as far as possible in order to reduce the costs for the storm water system. This should be observed in all general and detail planning for development. Eventually, of course, a limited amount of excavations and corrections will be necessary to meet reasonable solutions or other demands.

Catchment areas A, B, C, D and E are proposed to be furnished with storm water mains according to the city plans. The receiving body of water will be Wadi Karaizah Washak or its tributaries. These pipelines are to be drawn as soon as possible to the open water-courses. Wadi Karaizah Washak is kept open all the way through areas A and C as well as its tributary through area D. This tributary receives water from the western of the two northern flood control canals. The tributary through area B is kept open at east upto area F and preferably as high up as is shown on the drawing. North of this point the valley is more level and can be culverted, thus enabling a better utilization of the region for building purposes.

Catchment areas H and I are proposed to be furnished with the storm water mains according to the city plans with the tributary to Wadi Karaizah Washak through area I and south thereof as the receiving body of water. The tributary is to be kept open and certain excavations and corrections may be necessary.

The storm water from area P is drained to the flood control canal located on the eastern boundary of the area. This canal empties into Wadi Walobah. Depending on the future development, it may eventually prove advisable in the western part of the area to construct a smaller diverting canal to free the area from drainage from the high regions on the east. Areas S and Q are proposed to be furnished with storm water mains joining some smaller wadis west of these areas. A main sewer through area Q must be dimensioned for draining a smaller catchment area to the east. The other catchment areas are drained by the existing storm sewer system.

5.6.2 Design of Sanitary Sewers

The city has been divided into 18 catchment areas, designated A through S. The catchment areas are shown in Drawing No. 4. The present built-up area of the city lies mainly within the areas F, G, K, L, M, N, O, R, S and Q. There is some settlement within the areas E and P. The other areas do not have any buildings of importance to this subject.

Proposals for a system of main sewers are presented in Drawing No. 5. All sewage will be collected at one point in the southwestern part of the proposed development area of the city close to Wadi Karaizah Washak. Here the waste water will be subjected to treatment in a sewage treatment plant before being discharged into Wadi Karaizah Washak, which thus constitutes the receiving body of water. No pumping of the waste water will be necessary. All waste water will be conveyed to the sewage treatment plant by gravity.

The analysis of the sanitary sewer system is discussed hereunder.

The area, the present and future population, and the amount of waste water of each catchment area is given in Table 5.1.

a) Catchment area A is located in the western part of the city. There is no building of importance within this area. The waste water is collected in a main sewer running along Wadi Karaizah Washak to the proposed sewage treatment plant. On the plan of the main sewers, Drawing No. 5, this sewer is not shown since its extension and dimensions are dependent on the building plan within the area.

TABLE 5.1

AREA, POPULATION AND AMOUNT OF WASTE WATER OF THE CATCHMENT AREAS

Catchment area	Area (acres)	Present Population	Future Population	Expected waste water (cu.ft./day)
A	400	0	3,200	34,200
B	366	1,200	3,320	35,500
C	284	0	6,950	74,200
D	172	0	4,300	46,000
E	260	950	6,700	71,600
F	85	2,350	2,800	29,900
G	240	4,250	6,150	65,700
H	220	0	1,760	18,800
I	230	0	2,200	23,500
K	182	7,600	9,440	101,000
L	279	13,800	18,900	205,000
M	264	17,500	18,500	197,800
N	166	2,700	7,460	79,700

TABLE 5.1 (Cont'd)

Catchment area	Area (acres)	Present Population	Future Population	Expected waste water (cu.ft./day)
O	168	2,200	5,550	59,300
P	326	1,300	8,150	87,100
R	38	950	1,250	13,370
S	111	1,100	3,660	39,100
Q	69	1,100	2,210	23,600
Total		57,000	112,500	1,202,500
				= 9 mgd.

b) Catchment area B is located east of area A. The municipal electrical sub-station is situated within area B. In the eastern part of this area there is some residential housing. A small tributary to Wadi Karaizah Washak runs centrally through the area, from which the waste water is collected by main B that runs parallel to the tributary and leads to the proposed sewage treatment plant. The extension of this sewer is shown in Drawing No. 5. The sewer also receives waste water from area E and F located up-stream. The extension of the sewer can eventually be revised to a certain degree depending on how the building plan works out.

c) Catchment area C is located north of area A. At present area C is completely undeveloped. The waste water main largely following Wadi Karaizah Washak, which flows through the region. The outlet is connected with the main sewer through area A.

d) Catchment area D is located immediately east of area G. At present it is completely unsettled. The waste water sewer following the tributary to Wadi Karaizah Washak flows through the area. However the extension of this sewer depends on how the building plan is worked out and, therefore, it is not shown on the plan over the system of main sewers. The outlet is connected with the main sewer through area A and conveyed through this to the sewage treatment plant.

e) Catchment area E lies southeast of area D. In its northern part, on both sides of the flood control cannal, there is a smaller residential district. The waste water sewer is connected to the main sewer B.

f) Catchment area F is located south of area E and immediately north of the highway to Kirkuk. It is largely built up with modern residential houses of high quality and modern street planning. The waste water from this catchment area is connected to main B.

g) Catchment G is situated north-east and south of area F and where the cigarette factory, municipal park and army camp are located. It is proposed that the waste water of area F be conveyed to a main sewer which is to be connected to the trunk at manhole T 37, 350 m inside area I.

h) Catchment area H lies southwest of area G and south of area B. In this area there are no buildings of any importance. It is proposed that the waste water draining be conveyed to the trunk along the southern boundary of the area. Immediately southwest of the area is the site of the proposed sewage treatment plant. The formation of the sewage within the area is dependent on the building plan. The area is suitable for the localization of industrial plants.

i) Catchment area I is situated south of area G and has the municipal athletic field. The pipe line from area G is proposed to be used as the main sewer as in addition to the aformationed trunk along the southern boundary of the area. It is recommended that the southern parts of the area should be utilized for the localization of new industrial plants.

j) Catchment area K lies immediately east of area G, and is largely built up. It is proposed that the sewage of this area is conveyed to the main sewer K which is connected to the main sewer KL and the latter connected to the trunk at the manhole.

k) Catchment area L includes the central parts of the city and within this area lies the mutasarafia (provincial governing body), the hospital and the central market. The waste water is conveyed to main L which is connected to KL.

l) Catchment area M lies immediately east of area L which embodies the old parts of the city. The area is almost completely built up. The waste water is conveyed to mains M_1 and M_2 which are joined together to form M which is connected to the trunk at the manhole.

m) Catchment area N is situated along Wadi Walabah, south of area M. About half of the area is built up. It is proposed that the waste water of this area is conveyed to the main sewer N which also receives waste water from areas O and P located northeast of area N.

n) Catchment areas O and P are situated in the eastern part of the city. The waste water from these areas is conveyed to main N. The extension of the main sewer within the areas are dependent on the area plan.

o) Catchment area R lies in the southern part of the city, south of Wadi Walobah. The area is built up with newly erected residential houses. The waste water of the area will be conveyed by main sewer R along Wadi Walobah, and will join the main sewer N at manhole No. 10, Wadi Walobah is crossed by means of a sewer bridge.

p) Catchment area S lies west of area R and south of Wadi Walobah. The buildings within this area are rather new, and the waste water is conveyed to main sewer S which is connected to the trunk at manhole T-46.

q) Catchment area Q lies, like areas R and S, south of Wadi Walobah. It is proposed that main sewer S be constructed to carry out the waste water from the area. This sewer will be connected to trunk sewer at manhole T-46.

The design of sewers is represented in Tables 5.2 and 5.3. The calculation for velocities and slopes have been based on the nomogram based on Mannings Formula for circular pipes flowing full with $n = 0.013$[9]. Profiles of the trunk sewer and one main sewer are shown in Drawings No. 6 and No. 7.

The minimum depth of flow during lowest discharge has been checked to be not less than 2 inches, and the velocity not less than 2 ft./sec. by using the curves for partial flow.[9]

TABLE 5.2.

FLOW IN THE SANITARY SYSTEM

LINE	From manhole	To manhole	Length m	Increment of area acres	Increment Population	Total Population	Sewage flow (gpm)	Sewage flow (cfs)
B	-	31	-	345	9500	9500	760,000	1.175
	31	20	723	119	2144	11644	925,000	1.430
	20	15	530	54	432	12076	966,000	1.495
	15	8	592	60	480	12556	1,005,000	1.555
	8	T1	600	33	264	12820	1,028,000	1.590
I	23	17	365	178	5600	5600	448,000	0.695
	17	5	894	62	550	6150	492,000	0.760
	5	1	345	122	1500	7650	612,000	0.946
K	26	23	185	57	3680	3680	294,400	0.457
	23	17	428	28	1975	5655	452,400	0.700
	17	10	486	42	1685	7340	587,200	0.910
	10	KL1	-	55	2100	9440	755,200	1.170
L	29	23	350	180	10,950	10,950	876,000	1.350
	23	18	376	32	2,560	13,510	1,080,800	1.670
	18	11	504	40	3,200	16,710	1,336,800	2.060
	11	6	300	15	1,200	17,910	1,432,800	2.210
	6	1	285	12	990	18,900	1,512,000	2.340

LINE	From man-hole	To man-hole	Length m	Incre-ment of area acres	Incre-ment Popula-tion	Total Popula-tion	Sewage flow (gpm)	Sewage flow (cfs)
M1	29	23	487	48	2700	2700	216,000	0.334
	23	18	338	20	1310	4010	320,800	0.496
	18	9	581	38	3000	7010	560,800	0.868
	9	7	146	16	1230	8240	659,200	1.022
	7	M9	520	25	2000	10240	819,200	1.266
M2	18	15	230	15	1020	1020	81,600	0.126
	15	7	555	50	3160	4180	334,400	0.502
	7	M9	418	13	1040	5220	417,600	0.645
M	9	7	152	224	15460	15460	1,236,800	1.911
	7	3	253	24	1950	17410	1,392,800	2.150
	3	MN9	245	15	1090	18500	1,480,000	2.295
N	40	30	638	326	8150	8150	625,000	1.010
	30	21	580	210	6760	14910	1,192,800	1.848
	21	15	492	50	3000	17910	1,432,800	2.220
	15	10	310	33	1620	19530	1,562,400	2.420
	10	MN9	652	68	2200	21730	1,738,400	2.680

LINE	From man-hole	To man-hole	Length m	Incre-ment of area acres	Incre-ment Popula-tion	Total Popula-tion	Sewage flow (gpm)	Sewage flow (cfs)
MN	9	T42	574	962	40910	40910	3,272,800	5.060
R	8	N10	530	38	1250	1250	100,000	0.155
S	13	T46	865	111	3660	3660	292,800	0.453
Q	16	T46	945	69	2210	2210	176,800	0.273
T	46	42	250	180	5870	5870	469,600	0.725
	42	37	328	547	40910	46780	3,742,400	5.785
	37	21	1158	461	28340	75120	6,009,600	9.315
	21	17	245	108	700	75820	6,065,600	9.375
	17	7	753	362	7650	83470	6,677,600	10.330
	7	1	547	220	1760	85230	6,818,400	10.560
	1	Treatment Plant	80	1557	27270	112500	9,000,000	13.920

TABLE 5.3.

DESIGN OF SANITARY SEWER SYSTEM

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
B	35	34	1.43	3.58	95	812.1	809.6	10	0.0150	1.42	5.0	2.72	809.20	807.78
	34	33	"	"	54	809.6	809.2	"	"	0.81	"	"	807.78	806.97
	33	32	"	"	50	809.2	808.4	"	"	0.75	"	"	806.97	806.22
	32	31	"	"	65	808.4	805.5	"	0.0395	2.56	7.3	3.97	806.22	803.66
	31	30	"	"	"	805.5	802.1	"	"	2.56	"	"	803.66	801.10
	30	29	"	"	68	802.1	800.0	"	0.034	2.32	6.7	3.64	801.10	798.78
	29	28	"	"	65	800.0	798.4	"	"	2.21	"	"	798.78	796.57
	28	27	"	"	"	798.4	795.3	"	"	2.21	"	"	796.57	794.36
	27	26	"	"	63	795.3	796.0	"	0.0172	1.08	5.4	2.94	794.36	793.28
	26	25	"	"	65	796.0	794.7	"	"	1.10	"	"	793.28	792.18

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
B	25	24	1.43	3.58	65	794.7	790.5	10	0.0172	1.10	5.4	2.94	792.18	791.08
	24	23	1.495	3.74	80	790.5	791.2	12	0.0090	0.77	4.7	3.70	791.08	790.31
	23	22	"	"	80	791.2	787.9	"	0.0370	2.96	7.3	5.75	790.31	787.35
	22	21	"	"	50	787.9	787.3	"	0.011	0.55	4.8	3.77	787.35	786.80
	21	20	"	"	78	787.3	787.7	"	"	0.86	"	"	786.80	785.94
	20	19	"	"	60	787.7	787.0	"	"	0.66	"	"	785.94	785.28
	19	18	"	"	"	787.0	785.7	"	"	0.66	"	"	785.28	784.62
	18	17	1.555	3.900	57	785.7	784.2	"	0.0390	2.20	7.0	5.5	784.62	782.42
	17	16	"	"	55	784.2	779.5	"	"	2.16	"	"	782.42	780.26
	16	15	"	"	64	779.5	778.8	"	"	2.50	"	"	780.26	797.76
	15	14	"	"	65	778.8	776.3	"	"	2.54	"	"	797.76	795.22

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
B	14	13	1.555	3.900	66	776.31	775.82	12	0.008	0.53	4.5	3.54	774.50	773.97
	13	12	"	"	60	775.80	776.00	"	"	0.48	"	"	773.97	773.49
	12	11	"	"	"	776.00	775.00	"	"	"	"	"	773.49	773.01
	11	10	"	"	"	775.00	770.90	"	0.0590	3.54	9.0	7.05	773.00	769.46
	10	9	"	"	53	770.90	770.00	15	0.0050	0.26	3.6	4.42	769.46	769.20
	9	8	"	"	50	770.00	770.41	"	"	0.25	3.6	"	769.20	768.95
	8	7	1.590	3.98	98	770.40	771.00	"	"	0.49	3.7	4.55	768.95	768.46
	7	6	"	"	62	771.00	771.20	"	"	0.31	3.7	"	768.46	768.15
	6	5	"	"	60	771.20	769.32	"	"	0.30	"	"	768.15	767.85
	5	4	"	"	84	769.35	769.80	"	"	0.42	"	"	767.85	767.43
	4	3	"	"	80	769.83	769.10	"	"	0.40	"	"	767.43	767.03
	3	2	"	"	78	769.15	768.15	"	"	0.39	"	"	767.03	766.64

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
B	2	1	1.590	3.98	70	768.12	768.00	15	0.0050	0.35	3.7	4.55	766.64	766.29
	1	T1	"	"	"	768.00	767.90	"	"	"	"	"	766.29	765.94
I	23	22	0.695	1.74	67	817.70	816.15	8	0.0214	1.43	5.5	1.93	816.00	814.57
	22	21	"	"	60	816.15	814.70	"	"	1.28	"	"	814.57	813.29
	21	20	"	"	"	814.70	813.60	"	"	"	"	"	813.29	812.01
	20	19	"	"	62	813.60	812.45	"	0.0158	0.98	4.0	1.4	812.01	811.03
	19	18	"	"	60	812.45	811.65	"	"	0.95	"	"	811.03	810.08
	18	17	0.760	2.15	55	811.65	811.10	10	0.0085	0.46	3.8	2.08	810.08	809.62
	17	16	"	"	50	811.10	810.15	"	0.0114	0.62	5.0	2.74	809.62	809.00
	16	15	"	"	55	810.15	809.44	"	"	0.97	"	"	809.00	808.03
	15	14	"	"	85	809.00	809.00	"	"	1.10	"	"	808.03	806.93

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
I	14	13	0.760	2.15	96	809.00	808.90	10	0.0114	2.04	5.0	2.74	806.93	804.89
	13	12	0.946	2.36	79	808.90	806.30	"	0.0264	1.98	5.5	3.02	804.89	802.91
	12	11	"	"	75	806.30	804.60	"	"	1.98	"	"	802.91	800.93
	11	10	"	"	"	804.6	801.70	"	"	1.98	"	"	800.93	798.95
	10	9	"	"	"	801.70	800.00	"	"	1.98	"	"	798.95	796.97
	9	8	"	"	"	800.00	797.95	"	"	1.98	"	"	796.97	794.99
	8	7	"	"	"	797.95	795.90	"	0.0254	1.90	"	3.00	794.99	793.09
	7	6	"	"	"	795.90	794.65	"	"	1.90	"	"	793.09	791.19
	6	5	"	"	"	794.65	792.40	"	"	1.90	"	"	791.19	789.29
	5	4	"	"	69	792.40	789.50	"	0.0254	1.76	"	"	789.29	787.53
	4	3	"	"	70	789.50	788.30	"	"	1.78	"	"	787.53	785.75
	3	2	"	"	60	788.30	787.70	"	"	1.52	"	"	785.75	784.23

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
I	2	1	0.946	2.36	60	787.70	785.90	10	0.0254	1.52	5.5	3.00	784.23	782.71
	1		"	"	84	785.90	782.40	"	0.0149	1.25	4.5	2.56	782.71	781.46
K	25	24	0.457	1.140	65	849.30	846.45	8	0.0277	1.80	5.0	1.75	846.80	845.00
	24	23	"	"	60	846.45	844.60	"	"	1.67	"	"	845.00	843.33
	23	22	"	"	"	844.60	843.30	"	"	1.67	"	"	843.33	841.66
	22	21	0.70	1.70	70	843.30	841.95	"	0.0165	1.15	4.4	1.54	841.66	840.51
	21	20	"	"	72	841.95	840.85	"	"	1.19	"	"	840.51	839.32
	20	19	"	"	65	840.85	839.90	"	"	1.07	"	"	839.32	838.25
	19	18	"	"	75	839.90	838.70	"	"	1.23	"	"	838.25	837.02
	18	17	"	"	"	838.70	837.70	"	"	"	"	"	837.02	835.79
	17	16	"	"	"	837.70	836.00	"	"	"	"	"	835.79	834.56

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
K	16	15	0.910	2.27	82	836.00	833.00	8	0.0275	2.26	6.1	2.13	833.56	831.30
	15	14	"	"	78	833.00	830.40	"	0.0275	2.14	6.0	2.10	830.95	828.83
	14	13	"	"	80	830.40	828.70	"	"	2.20	"	"	828.83	826.63
	13	12	"	"	"	828.70	825.80	"	0.0263	2.11	5.9	2.06	826.30	824.19
	12	11	"	"	"	825.80	824.00	"	"	"	"	"	824.15	822.04
	11	10	"	"	"	824.00	822.00	"	"	"	"	"	822.04	819.93
	10	9	"	"	87	822.00	815.00	"	0.0450	3.92	7.4	2.58	818.93	815.01
	9	8	1.17	2.92	83	815.00	811.90	10	0.0314	2.61	5.9	2.06	815.01	812.40
	8	7	1.17	"	75	811.90	810.00	"	0.0314	2.35	5.9	3.22	812.40	810.05
	7	6	"	"	74	810.00	807.70	"	"	2.32	"	"	810.05	807.73
	6	5	"	"	75	807.70	805.20	"	"	2.35	"	"	807.73	805.38
	5	4	"	"	62	805.20	805.70	"	"	1.95	"	"	805.38	803.39

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
K	4	3	1.17	2.92	55	805.70	803.80	10	0.0314	1.72	5.9	3.22	803.39	801.67
	3	2	"	"	83	803.80	799.30	"	"	2.60	"	"	801.67	799.07
	2	1	"	"	80	799.30	800.20	"	"	2.50	"	"	799.07	796.57
	1	KL1	"	"	40	800.20	796.00	"	"	1.25	"	"	796.57	796.32
L	29	28	1.35	3.38	60	850.20	848.00	10	0.0358	2.15	7.0	3.82	848.80	846.65
	28	27	"	"	58	848.00	846.50	"	"	2.08	"	"	846.65	844.57
	27	26	"	"	52	846.50	845.00	"	"	1.86	"	"	844.57	842.71
	26	25	"	"	60	845.00	842.00	"	"	2.15	"	"	842.71	840.56
	25	24	"	"	80	842.00	841.10	"	0.020	1.60	5.8	3.17	840.56	838.96
	24	23	"	"	37	841.10	840.20	"	"	0.74	"	"	838.96	838.22
	23	22	1.67	4.17	67	840.20	835.80	"	0.0437	2.92	8.1	4.42	838.22	835.30
	22	21	"	"	75	835.80	832.90	"	"	3.28	"	"	835.30	832.02

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
L	21	20	1.67	4.17	77	832.90	829.60	10	0.0437	3.36	8.1	4.42	832.02	828.66
	20	19	"	"	76	829.60	828.40	12	0.018	1.37	6.1	4.80	828.66	827.29
	19	18	"	"	78	828.40	828.20	"	"	1.40	"	"	827.29	825.89
	18	17	2.06	5.15	104	828.20	822.10	"	0.045	4.70	9.2	7.23	825.89	821.19
	17	16	"	"	30	822.10	821.40	"	0.022	0.66	6.7	5.27	821.19	320.53
	16	15	"	"	75	821.40	819.80	"	"	1.65	"	"	820.53	818.88
	15	14	"	"	"	819.80	818.30	"	"	"	"	"	818.88	817.23
	14	13	"	"	100	818.30	815.90	"	"	2.20	"	"	817.23	815.03
	13	12	"	"	75	815.90	812.40	"	0.049	3.70	9.0	7.07	815.03	811.33
	12	11	"	"	"	812.40	810.70	"	0.029	2.17	7.4	5.80	811.33	809.16
	11	10	2.21	5.52	67	810.70	808.90	"	0.029	1.94	7.5	5.90	809.16	807.22
	10	9	"	"	60	808.90	807.30	"	"	1.74	7.5	"	807.22	805.48

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
L	9	8	2.21	5.52	60	807.30	805.40	12	0.029	1.74	7.5	5.90	805.48	803.74
	8	7	"	"	57	805.40	802.00	"	0.0395	2.26	8.3	6.52	803.74	801.48
	7	6	"	"	58	802.00	800.30	"	"	2.30	"	"	801.48	799.18
	6	5	2.34	5.85	50	800.30	879.70	15	0.0090	0.45	4.9	6.00	799.18	798.73
	5	4	"	"	55	879.70	879.00	"	"	0.49	"	"	798.73	798.24
	4	3	"	"	55	879.00	778.80	"	"	"	"	"	789.24	797.75
	3	2	"	"	87	878.80	776.10	"	"	0.78	"	"	797.75	796.97
	2	1	"	"	35	776.10	775.00	"	"	0.31	"	"	796.97	796.66
KL	KL1	T	3.52	8.77	67	894.90	893.10	18	0.009	0.60	5.4	9.60	796.30	795.70

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
M1	29	28	0.334	0.0835	80	902.20	899.00	8	0.0438	3.52	5.4	1.89	900.80	897.28
	28	27	"	"	"	899.00	895.60	"	"	"	"	"	897.28	893.76
	27	26	"	"	"	895.60	891.60	"	"	"	"	"	893.76	890.24
	26	25	"	"	"	891.60	887.60	"	"	"	"	"	890 24	886.72
	25	24	"	"	87	887.60	884.10	"	0.0405	"	"	"	886.72	883.20
	24	23	"	"	80	884.10	881.00	"	"	3.24	"	"	883.20	879.96
	23	22	0.496	1.240	43	881.00	879.70	"	0.0210	0.90	4.6	1.61	879.96	879.06
	22	21	"	"	62	879.70	878.60	"	"	1.30	"	"	879.06	877.76
	21	20	"	"	84	878.60	875.70	"	0.0406	3.40	3.6	1.26	877.76	874.36
	20	19	"	"	75	875.70	872.60	"	"	3.04	"	"	874.36	871.32
	19	18	"	"	"	872.60	869.30	"	"	"	"	"	871.32	868.28
	18	17	0.868	2.17	105	869.30	864.30	"	"	[illegible].25	6.6	2.31	868.28	864.03

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
M1	17	16	0.868	2.17	85	864.30	861.60	8	0.0326	2.77	6.2	2.17	863.00	860.23
	16	15	"	"	80	861.60	859.50	"	"	2.61	"	"	860.23	857.62
	15	14	"	"	"	859.50	856.80	"	"	"	"	"	857.62	855.01
	14	13	"	"	53	856.80	854.80	"	0.0223	1.18	5.5	1.92	855.01	853.83
	13	12	"	"	34	854.80	854.10	"	"	0.78	"	"	853.83	853.05
	12	11	"	"	25	854.10	853.70	"	"	0.57	"	"	853.05	852.48
	11	10	"	"	90	853.70	851.70	"	"	2.05	"	"	852.48	850.43
	10	9	"	"	28	851.70	851.20	"	"	0.65	"	"	850.43	849.78
	9	8	1.022	2.56	75	851.20	849.30	10	0.0277	2.08	5.3	2.90	849.78	847.70
	8	7	"	"	70	849.30	846.50	"	"	1.94	"	"	847.70	845.76
	7	6	1.266	3.16	75	846.50	842.80	"	0.039	2.93	7.0	3.82	844.46	841.52
	6	5	"	"	"	842.80	840.40	"	"	"	"	"	841.52	838.50

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
M1	5	4	1.266	3.16	64	840.40	839.60	12	0.0053	0.34	3.7	2.90	838.50	838.26
	4	3	"	"	55	839.60	939.30	"	"	0.29	"	"	838.26	837.27
	3	2	"	"	88	839.30	838.90	"	"	0.47	"	"	837.97	837.70
	2	1	"	"	85	838.90	838.60	"	"	0.45	"	"	837.70	837.25
	1	M9	"	"	85	838.60	838.10	"	"	0.45	"	"	837.25	836.80
M2	18	17	0.126	0.360	100	872.30	870.40	8	0.020	2.00	2.8	0.98	871.00	869.00
	17	16	"	"	82	870.40	867.80	"	0.0188	1.54	2.7	0.94	868.00	866.46
	16	15	"	"	55	867.80	867.50	"	"	1.03	"	"	866.46	865.43
	15	14	0.502	1.255	88	867.50	865.90	"	"	1.65	4.2	1.46	865.43	863.78
	14	13	"	"	90	865.90	863.40	"	"	1.69	"	"	863.78	862.09
	13	12	"	"	54	863.40	862.50	"	0.0158	0.85	4.1	1.43	862.39	861.24
	12	11	"	"	58	862.50	861.70	"	"	0.91	"	"	861.24	860.33

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing ft/sec	Cap. flow ing cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
M2	11	10	0.502	1.255	54	861.70	860.70	8	0.0158	0.85	4.1	1.43	860.33	859.48
	10	9	"	"	70	860.70	857.90	"	0.0413	2.90	5.4	1.89	859.48	856.58
	9	8	"	"	"	857.90	854.70	"	"	"	"	"	856.58	853.68
	8	7	"	"	"	854.70	852.20	"	"	"	"	"	853.68	850.78
	7	6	0.645	1.610	80	852.20	851.10	"	0.0194	1.55	4.8	1.68	850.68	849.13
	6	5	"	"	"	851.10	848.80	"	"	"	"	"	849.13	847.58
	5	4	"	"	45	848.80	847.30	"	0.0370	1.66	6.1	2.14	847.58	845.92
	4	3	"	"	42	847.30	845.70	"	"	1.55	"	"	845.92	844.37
	3	2	"	"	70	845.70	843.00	"	"	2.59	"	"	844.37	841.78
	2	1	"	"	"	843.00	838.80	"	0.0500	3.50	6.4	2.24	840.93	837.48
	1	M9	"	"	29	838.80	838.20	"	0.0260	0.75	5.2	1.82	837.48	836.73

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
M	9	8	1.911	4.77	76	838.20	837.20	12	0.0178	1.35	6.2	4.87	836.70	835.35
	8	7	"	"	"	837.20	835.40	"	"	"	"	"	835.35	834.00
	7	6	2.150	5.38	40	835.20	833.90	"	0.0250	1.00	7.1	5.58	832.00	831.00
	6	5	"	"	57	833.90	831.60	"	"	1.43	"	"	831.00	829.57
	5	4	"	"	60	831.60	829.60	"	"	1.50	"	"	829.57	828.07
	4	3	"	"	93	829.60	826.10	"	0.0350	3.26	7.9	6,20	828.07	824.81
	3	2	2.295	5.75	85	826.10	824.20	"	0.0204	1.73	6.3	4,95	824.81	823.08
	2	1	"	"	80	824.20	822.40	"	"	1.63	"	"	823.08	821.45
	1	T	"	"	"	822.40	817.50	"	"	"	"	"	821.45	819,82

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
N	40	39	1.01	2.53	60	900.60	901.70	12	0.003	0.18	2.9	2.28	899.00	898.82
	39	38	"	"	"	901.70	901.30	"	"	"	"	"	898.82	898.64
	38	37	"	"	"	901.30	900.10	"	"	"	"	"	898.64	898.46
	37	36	"	"	70	900.10	899.50	"	0.011	0.77	4.2	3.30	898.46	897.69
	36	35	"	"	75	899.50	898.30	"	"	0.83	"	"	897.69	896.86
	35	34	"	"	88	898.30	896.70	"	0.0033	0.29	6.2	4.87	895.10	894.81
	34	33	"	"	60	896.70	897.00	"	"	0.20	"	"	894.81	894.61
	33	32	"	"	"	897.00	896.90	"	"	"	"	"	894.61	894.41
	32	31	"	"	50	896.90	894.30	"	0.0537	2.68	7.2	5.66	894.41	891.73
	31	30	"	"	54	894.30	890.80	"	"	2.90	"	"	891.73	888.83
	30	29	1.848	4.62	65	890.80	890.50	15	0.003	0.20	3.3	4.05	888.83	888.63
	29	28	"	"	53	890.50	888.10	"	0.0612	3.24	8.2	10.00	888.63	885.39

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
N	28	27	1.848	4.62	50	888.10	884.80	15	0.0612	3.08	8.2	10.00	885.39	882.31
	27	26	"	"	65	884.80	884.40	"	0.0030	0.20	3.3	4.05	882.31	882.11
	26	25	"	"	70	884.40	883.50	"	0.0030	0.21	"	"	882.11	881.90
	25	24	"	"	68	883.50	877.80	"	0.0927	6.30	11.0	13.50	881.90	875.60
	24	23	"	"	40	877.80	876.10	"	0.030	1.20	7.1	8.70	875.60	874.40
	23	22	"	"	84	876.00	874.20	"	"	2.55	"	"	874.40	871.85
	22	21	"	"	85	874.20	867.90	"	0.0675	5.75	9.6	11.80	871.85	866.10
	21	20	2.220	5.55	98	867.90	863.30	"	0.0448	4.40	8.4	10.30	866.10	861.70
	20	19	"	"	60	863.30	863.00	"	0.0200	1.20	6.5	7.95	861.70	860.50
	19	18	"	"	73	863.00	859.50	"	0.0405	2.96	8.9	10.90	860.30	857.34
	18	17	"	"	90	859.50	856.30	"	"	3.68	"	10.90	857.34	853.66
	17	16	2.220	5.55	85	856.30	852.90	"	"	3.44	"	"	853.60	850.16

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
N	16	15	2.22	5.55	85	852.30	849.20	15	0.0405	3.44	8.9	10.90	850.16	846.72
	15	14	2.42	6.05	65	849.20	845.00	"	0.0280	1.82	7.6	9.30	845.80	843.98
	14	13	"	"	70	845.00	844.40	"	"	1.96	"	"	843.98	842.02
	13	12	"	"	58	844.40	842.80	"	"	1.62	"	"	842.02	840.40
	12	11	"	"	57	842.80	841.10	"	0.0383	2.18	8.4	10.30	840.40	837.22
	11	10	"	"	60	841.10	839.30	"	"	2.30	"	"	837.22	834.92
	10	9	2.68	6.70	68	839.30	836.50	"	"	2.60	8.7	10.63	834.92	832.32
	9	8	"	"	70	836.50	832.00	"	"	2.68	"	"	832.32	829.64
	8	7	"	"	"	832.00	830.00	"	0.0283	1.98	"	"	829.64	827.66
	7	6	"	"	"	830.00	827.80	"	"	"	"	"	827.66	825.68
	6	5	"	"	"	827.80	825.00	"	"	"	"	"	825.68	823.70
	5	4	"	"	65	825.00	824.50	"	"	1.84	"	"	823.70	821.86

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
N	4	3	2.68	6.70	65	824.50	821.90	15	0.0283	1.84	8.7	10.63	821.86	820.02
	3	2	"	"	88	821.90	822.00	18	0.003	0.26	3.5	6.25	820.02	819.76
	2	1	"	"	85	822.00	821.70	"	"	"	"	"	819.76	819.50
	1	MN9	"	"	25	821.70	816.10	"	0.0916	2.29	11.5	20.40	819.50	817.21
MN	9	8	5.06	12.63	39	816.10	820.30	18	0.0173	0.68	7.4	13.15	817.00	816.32
	8	7	"	"	55	820.30	818.00	"	"	0.95	"	"	816.32	815.37
	7	6	"	"	60	818.00	816.50	"	"	1.04	"	"	815.37	814.33
	6	5	"	"	"	816.50	815.80	"	"	"	"	"	814.33	813.29
	5	4	"	"	54	815.80	815.30	"	"	0.94	"	"	813.29	812.35
	4	3	"	"	60	815.30	813.80	"	"	1.04	"	"	812.35	811.31
	3	2	"	"	82	813.80	811.40	"	0.0356	2.92	9.7	17.2	811.31	808.39
	2	1	"	"	70	811.40	809.50	"	"	2.50	"	"	808.39	805.89
	1	T42	"	"	"	809.50	805.00	"	"	"	"	"	805.89	803.49

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
R	8	7	0.155	0.620	55	849.30	846.50	8	0.0055	0.30	2.3	0.80	846.80	846.50
	7	6	"	"	92	846.50	848.00	"	"	0.50	"	"	846.50	846.00
	6	5	"	"	78	848.00	844.40	"	0.0425	3.30	5.0	1.75	846.00	842.70
	5	4	"	"	73	844.40	842.80	"	0.0274	2.00	4.3	1.50	842.70	840.70
	4	3	"	"	50	842.80	839.50	"	0.052	2.60	5.5	1.90	840.70	838.10
	3	2	"	"	50	839.50	838.70	"	0.005	0.30	2.3	0.80	838.10	837.80
	2	1	"	"	"	838.70	838.00	"	"	"	"	"	837.80	837.50
	1	N10	"	"	"	838.00	839.00	"	"	"	"	"	837.50	837.20

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
S	13	12	0.453	1.130	60	839.10	839.50	8	0.0068	0.40	3.2	1.12	837.20	836.80
	12	11	"	"	58	839.50	838.20	"	"	"	"	"	836.80	836.40
	11	10	"	"	67	838.20	834.50	"	0.0583	3.90	6.8	2.42	836.40	832.50
	10	9	"	"	50	834.50	831.80	"	"	2.90	"	"	832.50	829.60
	9	8	"	"	70	831.80	828.90	"	0.0415	"	6.0	2.10	829.60	826.70
	8	7	"	"	75	828.90	825.50	"	"	3.10	"	"	826.70	823.60
	7	6	"	"	"	825.50	822.10	"	"	3.00	"	"	823.60	820.60
	6	5	"	"	"	822.10	819.60	"	"	"	"	"	820.60	817.60
	5	4	"	"	"	819.60	816.70	"	"	"	"	"	817.60	814.60
	4	3	"	"	66	816.70	813.50	"	0.044	2.90	6.2	2.17	814.60	811.70
	3	2	"	"	60	813.50	810.70	"	"	2.64	"	"	811.70	809.06

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation		Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation	
						Upper manhole	Lower manhole						Upper manhole	Lower manhole
S	2	1	0.453	1.130	66	810.70	809.00	8	0.0293	1.93	5.2	1.82	809.06	807.13
	1	-	"	"	60	809.00	806.90	"	"	1.86	"	"	807.13	805.27
Q	16	15	0.273	1.09	50	835.00	834.80	8	0.03	1.50	4.8	1.68	832.00	830.50
	15	14	"	"	"	834.80	831.40	"	0.03	"	"	"	830.50	829.00
	14	13	"	"	60	831.40	830.00	"	"	1.70	"	"	829.00	827.30
	13	12	"	"	"	830.00	828.00	"	"	"	"	"	827.30	825.50
	12	11	"	"	53	828.00	826.40	"	"	1.59	"	"	825.50	823.91
	11	10	"	"	70	826.40	824.30	"	"	2.10	"	"	823.91	821.81
	10	9	"	"	"	824.30	822.40	"	"	"	"	"	821.81	819.71
	9	8	"	"	"	822.40	820.20	"	"	2.10	"	"	819.71	817.61
	8	7	"	"	50	820.20	819.60	"	0.0042	0.21	2.6	0.91	817.61	817.40

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
Q	7	6	0.273	1.09	55	819.60	819.80	8	0.0042	0.23	2.6	0.91	817.40	817.17
	6	5	"	"	63	819.80	819.90	"	"	0.27	"	"	817.17	816.90
	5	4	"	"	65	819.90	818.80	"	"	0.28	"	"	816.90	816.62
	4	3	"	"	56	818.80	818.00	"	0.0210	1.18	4.2	1.47	816.62	815.44
	3	2	"	"	55	818.00	815.90	"	"	1.15	"	"	815.44	814.29
	2	1	"	"	"	815.90	812.70	"	0.0782	4.30	7.0	2.45	814.29	809.99
	1	T46	"	"	60	812.70	807.00	"	"	4.79	"	"	809.99	805.29
T	46	45	0.725	1.810	50	807.00	806.40	8	0.0110	0.55	4.1	1.44	805.20	804.65
	45	44	"	"	"	806.40	805.70	"	"	"	"	"	804.65	804.10
	44	43	"	"	75	805.70	803.70	12	0.0033	0.25	2.6	2.04	804.10	803.85

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
T	43	42	0.725	1.810	75	803.70	805.10	12	0.0033	0.25	2.6	2.04	803.85	803.60
	42	41	5.785	14.50	67	805.10	806.10	18	0.0158	1.06	7.8	13.80	803.60	802.54
	41	40	"	"	65	806.10	804.00	"	"	1.03	"	"	802.54	801.51
	40	39	"	"	68	804.00	802.20	"	0.0285	1.94	9.6	17.00	801.51	799.57
	39	38	"	"	65	802.20	800.00	"	"	1.86	13.0	23.00	799.57	797.71
	38	37	"	"	63	800.00	793.30	"	0.0813	5.13	5.8	10.50	797.71	792.58
	37	36	9.315	23.3	78	793.30	796.00	27	0.0052	0.42	"	23.20	792.58	792.16
	36	35	"	"	70	796.00	794.10	"	"	0.37	"	"	792.16	791.79
	35	34	"	"	"	794.10	793.70	"	"	"	"	"	791.79	891.42
	34	33	"	"	72	793.70	792.10	"	"	0.38	"	"	791.42	791.04
	33	32	"	"	70	792.10	793.00	"	"	"	"	"	791.04	790.66

LINE	From man-hole	To man-hole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flow-ing full ft/sec	Cap. flow-ing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
T	32	31	9.315	23.3	58	793.00	792.60	27	0.0052	0.30	5.8	23.2	790.60	790.30
	31	30	"	"	65	792.60	792.10	"	"	0.34	"	"	790.30	789.96
	30	29	"	"	"	792.10	792.00	"	"	"	"	"	789.96	789.62
	29	28	"	"	82	792.00	792.30	"	"	0.43	"	"	789.62	789.19
	28	27	"	"	85	792.30	791.80	"	"	0.44	"	"	789.19	788.75
	27	26	"	"	65	791.80	791.90	"	"	0.34	"	"	788.75	788.41
	26	25	"	"	75	791.90	791.70	"	"	0.39	"	"	788.41	788.02
	25	24	"	"	"	791.70	791.60	"	"	0.39	"	"	788.02	787.63
	24	23	"	"	"	791.60	791.30	"	"	"	"	"	787.63	787.24
	23	22	"	"	"	791.30	791.10	"	"	"	"	"	787.24	786.85
	22	21	"	"	"	791.10	789.40	"	"	"	"	"	786.85	786.46

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
T	21	20	9.375	23.4	72	789.40	789.70	27	0.0052	0.37	5.8	23.2	786.46	786.09
	20	19	"	"	70	789.70	787.90	"	"	0.36	"	"	786.09	785.73
	19	18	"	"	76	787.90	783.90	"	0.0538	4.10	11.0	44.0	785.73	781.63
	18	17	"	"	27	783.90	782.90	"	0.0080	0.22	6.7	26.8	781.63	781.41
	17	16	10.33	25.8	85	782.90	782.00	"	0.0085	0.72	7.0	28.0	781.41	780.69
	16	15	"	"	80	782.00	782.00	"	"	0.68	"	"	780.69	780.01
	15	14	"	"	55	782.00	779.70	"	0.0627	3.45	11.5	46.0	780.01	776.56
	14	13	"	"	"	779.70	775.10	"	"	"	"	"	776.56	773.11
	13	12	"	"	80	775.10	772.90	"	0.0284	2.27	10.5	42.0	773.11	770.84
	12	11	"	"	75	772.90	770.60	"	"	2.13	"	"	770.84	768.71
	11	10	"	"	85	770.60	769.80	30	0.0035	0.23	5.0	31.2	768.71	768.48

LINE	From manhole	To manhole	Q ave.	Q max.	Length m	Ground Elevation Upper manhole	Ground Elevation Lower manhole	Dia. inch	Grade of Sewer	Fall of Sewer	Vel. flowing full ft/sec	Cap. flowing full cfs	Invert Elevation Upper manhole	Invert Elevation Lower manhole
T	10	9	10.33	25.8	85	769.80	769.00	30	0.0035	0.23	5.0	31.2	768.48	768.25
	9	8	"	"	79	769.00	770.00	"	"	"	"	"	768.25	768.02
	8	7	"	"	75	770.00	770.00	"	"	"	"	"	768.02	767.79
	7	6	10.56	26.4	93	770.00	769.00	"	"	0.32	5.1	31.8	767.79	767.47
	6	5	"	"	"	769.00	768.40	"	"	0.36	"	"	767.47	767.11
	5	4	"	"	93	768.40	768.90	"	"	"	"	"	767.11	766.75
	4	3	"	"	90	768.90	767.00	"	"	0.35	"	"	766.75	766.40
	3	2	"	"	"	767.00	767.40	"	"	"	"	"	766.40	766.05
	2	1	"	"	"	767.40	768.10	"	"	"	"	"	766.05	765.70
	1	treatment Plant	14.5	36.48	66	768.10	765.60	42	0.0015	0.10	"	"	765.70	765.60

6 TREATMENT OF SEWAGE

6.1. Recipient Conditions

The watercourses that are conceivable as receiving bodies of water for the waste water drainage of Sulaimaniya are (a) Wadi Karaizah Washak and (b) Wadi Walobah.

The available watercourses are equally suitable for purposes of discharge of waste water. Most of the year they are not water bearing or only slightly so. It is only during a short period that they are water-bearing to any significant degree.

The natural receiving body of water for the present city is Wadi Walobah. Only a small part of city is drained naturally to Wadi Karaizah Washak. The drainage from the entire existing settlement can be arranged to flow by gravity to Wadi Walobah. Eventually it may be advisable to resort to pumping of water precipitating in a very small part of the city in order to avoid deep excavations. Considering the extent of the present city, Wadi Walobah would thus be chosen as the receiving body of water and the sewage treatment plant could be located somewhere in the region. With regard to the development of the city in the foreseeable future and the accompanying geographic expansion presumably in a westerly direction, it is more advantageous part of the city as shown in Drawing No. 5.

This location has the following advantages:

a) All waste water flowing by gravity to the treatment plant.

b) The land is cheap and a large area can be purchased not only for present requirements but for the maximum estimated future extension works, plus a safety margin to allow for unexpected future possibilities.

c) The area of the treatment plant has a favorable slope which will help in reducing pumping within the treatment plant.

d) The selected land for the treatment works is not subject to floods.

e) The possibility of using the effluent for the irrigation of the downstream agricultural lands.

f) The site of the treatment plant would be so located that the prevailing winds would drive any possible odors away from the city.

6.3. Available Financial Resources

Sulaimaniya has a capable municipality to look after all essential services in the city. However, the municipality is not in a position to construct a project such as the one in question. Sewerage schemes, being recognized as the indispensable basis both for the individual's hygiene and for public health, imply a common work in which the various political and administrative organizations must take an active share. The Iraqi Government has not neglected this principle. The Ministry of Municipalities provides free technical help and long term loans free of interest (usually from 5 to 10 years) which enable the municipality to construct large projects. However, the Ministry is not capable of supporting all the projects simultaneously. But since the city has well organized systems for water supply and electricity it is expected that in the near future a sewerage scheme will be constructed.

6.4. Extent of Treatment

Sewage treatment works are designed to reduce the strength of sewage to a value which may be expected to insure avoidance of nuisance under the conditions in which sewage is discharged. The extent of treatment depends upon the required quality of the effluent which depends on the following factors[14]:

a) Regulatory Agencies

The design of most sewage treatment plants in developed countries requires the approval of a governmental agency. In the United Kingdom, for instance, the Royal Commission on Sewage Disposal laid down as a general standard for sewage effluents that an effluent should be considered satisfactory if it contains not more than 30 ppm. of suspended solids and has a strength of not more than 20 ppm of 5-day BOD[13]. Illinois and New York standards, on the other hand, require that the effluent from treatment plants should have not more than 15 ppm of BOD and should contain not more than 30 ppm of suspended solids.

In Iraq there is no regulatory agency concerned with the abatement of water pollution and the setting of quality standards and, hence; the designer has to estimate the required quality of the effluent at his own discretion.

b) Receiving Body of Water

The extent of treatment required is based upon the ability of the receiving body of water to assimilate the wastes and upon the uses of the water.

The proposed receiving body of water is not water-bearing during most of the year. This will necessitate that the final effluent be of high quality.

c) Effluent Utilization

The effluent from treatment plant may be used for irrigation, recharge of ground-water reservoirs and industrial purposes. In the case of the proposed project there is a possibility that the effluent will be used for irrigation of low growing plants such as vegetables due to the following reasons:

(1) The need for irrigation of agricultural land during the dry season since the land is usually planted with wheat only during winter.

(2) The water table is low and hence it will be cheaper to utilize the final effluent rather than ground water.

(3) There is a great need for irrigation.

In the light of the above discussion it is reasonable to anticipate a reduction in the strength of the effluent down to 15 mg/L of BOD with a properly designed and operated sewage treatment plant. The final effluent will also be chlorinated inorder to destroy the pathogenic bacteria.

6.5. Choice of Treatment Method

The type of sewage treatment works to be adopted in any particular instance depends upon the following factors:

a) Construction and operation costs.

b) Availability of funds and qualified personnel.

c) Characteristics of the sewage to be treated.

d) Method of final disposal.

e) Availability of suitable land for each of the methods of treatment.

f) Drop from incoming sewer to the point of outfall.

g) Efficiency and characteristic of the treatment plant.

h) Quantity and quality of the sludge to be handled.

i) Availability of equipment and material in the local market.

j) Possibility of future expansion of the treatment plant.

These factors collectively form an aconomic problem to which there should be only one correct answer in any particular instance.

Some of the important methods of treatment of domestic sewage which appear to fit the local conditions in Sulaimaniya are discussed briefly hereunder:

a) Imhoff Tank

An Imhoff tank, which is still widely used, consists of two chambers: an upper chamber through which the sewage passes at a very low velocity, and a lower chamber in which anaerobic decomposition takes place. The Imhoff tank represents a unit in which both sedimentation and digestion occur. It has been claimed that the initial cost of such treatment is cheap. However, this method is not suitable for the city in question because:

(1) The quality of the effluent from Imhoff tanks is less than the required one. Hence, further treatment will be necessary which will add to the cost.

(2) Nuisance is sometimes caused at the gas vents by a black foam which may reach such proportions as to spill over into the upper chamber. The foam is sometimes accompanied by very offensive odors.

(3) The operation and maintenance of these tanks is more difficult than many other methods and necessitates continuous supervision.

(4) The cost and the results of separate digestion tanks are more attractive[12].

b) Oxidation Ponds or Lagoons

Oxidation ponds or lagoons are artificial ponds of sewage which provide a water surface at which oxygen is dissolved from the atmosphere to provide aerobic conditions. Aerobic bacteria fungi and algae will then oxidize the organic matter and change it to inoffensive inorganic substances. Construction and operation is simple and cheap. Under favorable conditions they are capable of reducing the BOD of the sewage by more than 85 percent[14].

However, stabilization lagoons are not suitable for application in this project on account of the following:

(1) Ponds require a very large area of land which is not available at low cost. Moreover, the land is hilly which will increase the initial construction cost of the oxidation pond.

(2) Greater potential for disease transmitted by such vectors as mosquitoes[20] which may bried in improperly maintained lagoons.

(3) Greater difficulty in the effective desinfection of oxidation pond content for control of disease transmission (Cl_2 kills algae which are useful for maintaining aerobic conditions and the BOD of dead algae will thus increase the organic load on the pond.).

(4) Fencing of the pond area is necessary but this would increase the initial cost. In cases where the pond is not fenced cattle and even ignorant people will use it for drinking or swimming.

c) Oxidation Ditches

An oxidation ditch is a modified form of the activated sludge process and may be classified in the extended aeration group. Sewage flows into a small circular channel where it is made to circulate by means of a rotary brush which also aerates the sewage. Among the advantages claimed for it are: low initial and operational costs, flexibility, simplicity and ease of operation and maintenance. This method of treatment is not suitable for Sulaimaniya because:

(1) It is not applicable for populations of more than 10,000 persons.

(2) As the area is hilly it will be expensive to construct an oxidation ditch.

(3) The operational cost is relatively high[21].

d) Activated Sludge Process

The activated sludge is similar in character, composition and action to the biological film in trickling filters. When sewage is mixed with a sufficient amount of biologically active sludge rapid oxidation takes place. The bacterial population is supplied with oxygen artificially. The method is highly effective in the removal of suspended solids, oxygen - requiring substances and bacteria. Removals of suspended solids and BOD vary from 85 to 95 percent and bacteria from 90 to 98 percent[22]. Moreover, the process is free from odors, requires relatively small areas, and the produced sludge has greater fertilizer value. However, activated sludge is not recommended for Sulaimaniya because it requires a constant skilled attendance which is difficult to secure not only in this city but in the whole of Iraq. Moreover, this process has the following additional disadvantages:

(1) Lack of adaptability to variation in strength and composition of sewage.

(2) The sludge produced by this process has a high water content and, hence, its dewatering and disposal will be difficult.

(3) Operational cost is high.

e) Conventional Process (Trickling Filters)

The conventional process consists of primary and secondary treatment. The heart of the secondary treatment is the trickling filter. Among the advantages of a trickling filter the following may be included:

(1) Relatively low operating cost.

(2) Resistance to shock loads.

(3) Ability to work under variable climatic conditions.

(4) Relative high efficiency in the removal of BOD and suspended solids.

(5) Good performance with a minimum of skilled technical supervision.

The main disadvantage of the trickling filter is the head loss through it, which varies between 5 to 11 feet, in addition to the depth of the filter which is usually between 6 and 10 feet[12]. Other disadvantages over the activated sludge process are:

(1) Construction cost is relatively higher.

(2) It requires a slightly larger area.

(3) Production of odors and filter flies.

The area required for trickling filters is only slightly larger than that required by activated sludge units. Since the price of land in Sulaimaniya is relatively cheap, there will be only little additional cost. The cost of construction, operation and maintenance given by Schroepfer[12] are reproduced in Tables 6.1 and 6.2.

TABLE 6.1

CONSTRUCTION COST OF TRICKLING FILTER AND ACTIVATED SLUDGE
(Costs in Thousands of Dollars)

Plant Capacity of 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 hrs.	Average for Aeration 6 hrs.	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	10,850	3,700	4,750	5,200	6,200
100	7,900	10,000	13,200	6,300	6,300	6,900	8,200

TABLE 6.2

OPERATION AND MAINTENANCE COST FOR TRICKLING FILTER AND ACTIVATED SLUDGE PROCESSES
(In Thousands of Dollars)

Plant Capacity m.g.d.	Trickling Filter Plant			Activated Sludge Process			
	Usual Lower Limit	Estimated in this analysis	Usual Higher Limit	Usual Lower Limit	Average for Air 0.6 cft per gal.	Average for Air 1.0 cft per gal.	Usual Higher Limit
10	22	30	38	75	90	110	125
25	47	62	78	150	185	220	255
50	83	110	138	250	310	370	430
75	115	155	195	340	420	495	570
100	145	195	245	430	520	615	710

From these two tables, it is evident that the construction cost for trickling filters will be 60 percent higher than that for activated sludge units. The additional cost, however, will be off set by the savings resulting from operation and maintenance of the trickling filter which is 33 percent of that required for the activated sludge process.

Nuisance resulting from odor and filter flies will be minimized to a negligable effect by the use of rotary distributors and proper and adequate operation.

In the light of the foregoing discussion a conventional treatment plant will be adopted for the treatment of domestic sewage for Sulaimaniya.

7 DESIGN FOR TREATMENT PLANT

7.1 Treatment: Purpose and Design Criteria

7.1.1 Primary Treatment

Primary treatment consists solely in separating a portion of the suspended solids from the sewage. This is usually accomplished by screens, grit chambers and primary sedimentation tanks.

a) Screens

Screening is the first step in the treatment of sewage in order to remove comparatively large suspended and floating material liable to cause the following problems:

(1) Choke small pipes within the treatment plant.

(2) Interfere with the performance of pumps.

(3) Increase the volume of putrifying sludge.

(4) Form unsightly masses on weirs or scum boards.

The following design criteria will be adopted for the design:

(1) Screens should consist of galvanized steel racks having a cross section of 2" x 3/8" with 2" side parallel to sewage flow.

(2) The screens should be designed to provide a velocity of 1.0 ft./sec. at average rate of flow. Maximum velocities should not exceed 2.5 ft./sec.[12].

(3) Screen bars should be placed at 45° with the horizontal.

(4) Clear openings between bars should be one inch.

Since cheap unskilled labour is ~~are~~ available the cleaning of the screens will be performed manually at convenient intervals. The screenings will be disposed ~~off~~ along with the general refuse of the city or buried, for which sufficient and suitable land is available.

The screen channel invert will be located 6 inches below the invert of the incoming sewer to allow space for collection of large particles, which otherwise may cause stagnation and clogging at screens.

b) Grit Channel

A grit channel is an enlarged channel or tank through which the sewage flows at a velocity so controlled as to allow only the heavier in~~non~~organic solids, such as grit and sand, to be deposited while the lighter organic matter is kept suspended in the sewage.

The removal of grit has the following advantages:

(1) Prevention of clogging of pipes.

(2) Protection of pumps and other moving mechanisms and appurtenances.

(3) Protection of treatment plant equipment; grit may cause serious trouble in digestion tanks, clogs sludge lines, or accumulate in sedimentation tanks.

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

(1) The horizontal velocity of flow through the channel should be 1 fps.

(2) The settling velocity of particles 0.2 mm in size is given as 0.1 foot per second. This is the average size of grit usually found in sewage[14].

(3) Cleaning of channel is to be performed manually in view of the cheap labour.

(4) Two channels will be provided to facilitate cleaning. This operation will be performed during low flow periods when all the flow is passed through only one channel, thus relieving the second for cleaning. The construction of two channels will also facilitate proper control of velocities through the channel.

(5) Parshall flume will be used to control the velocity in the channel.

(6) One chamber for the screening and grit removing device will be constructed. This provides savings in construction and operation of the chamber.

(7) The grit will be washed in a separate chamber to remove the organic matter from it.

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

(9) The grit may be buried or used as a fill for nearby low areas.

c) Primary Sedimentation Tank

Primary sedimentation is a treatment method intended to reduce the content of settleable solids in order to prevent the formation of sludge banks, to reduce the load on the following treatment units, and to prepare the sewage for further treatment.

Sedimentation basins may be operated on the fill and draw principal, or they may be of the continuous flow type. The efficiency of the latter is much higher than the first, its cost lower, and the head losses in it are smaller; hence, it has been chosen for use in this project. Actually, sedimentation bains of the fill and draw type are seldom used nowadays.

The degree of clarification in the continuous flow sedimentation tanks is very much related to the detention period. As the detention period increases, the BOD removal increases also, but when it exceeds 2 hours the increased efficiency does not justify the extra cost. Hence, a detention period of 2 hours will be used.

The following design criteria will be adopted in the design of the tank.

(1) The tank shall be circular in shape and of the continuous flow system. The circular shape has been adopted because the maintenance cost is generally lower than that for any other shape[14].

(2) The floor of the tank should slope towards the centre at a rate of one inch per foot to facilitate removal of sludge.

(3) Cleanings will be achieved through mechanically operated scrapers. This will facilitate continuous removal of sludge without need to put the unit out of service for cleaning purposes. Furthermore, no allowance for sludge accumulation will be required in the design.

(4) The surface overflow rate should not exceed 800 gallons per square foot of surface area per 24 hours[14].

(5) The flow rate over the weir should not exceed 15,000 gallons per lineal foot per 24 hours[14].

(6) The depth of the tank will be as shallow as practical, but not less than 7 feet[18].

(7) Multiple units will be arranged for the present and ultimate flow.

(8) The units will be designed for average rate of flow.

(9) A concentric influent baffle will be provided, with a diameter 15 percent of the tank diameter and will extend 4 feet below the surface(14).

(10) V-notched type outlet weir will be provided which will extend around the entire periphery of the tank(14).

(11) Baffles extending 10 inches below the surface will be provided ahead of the overflow weirs to retain floating scum(14).

(12) Sludge removal from the hopper shall be performed by direct connection to the pump.

(13) Scum will be removed by skimmer floats on the surface and rides with the revolving collector arm.

(14) Sludge hopper will be located near the center and will have a volume sufficient to seal the outlet pipe and avoid entrance of overlying liquid(14).

(15) The details and dimensions of the structure, mechanical equipment, driving motors, etc., will be provided by manufacturers..

7.1.2 Secondary Treatment

The secondary treatment involves further treatment (or oxidation) of the effluent from the primary treatment process. This treatment consists of the trickling filters and final sedimentation tanks.

a) Trickling Filter

The action of the trickling filter is biological. Soon after a filter is placed in operation the surface of the media becomes coated with slime which consists mainly of aerobic bacteria and other micro-organisms such as worms, protozoa, etc.. Under favorable environmental conditions the slime layer absorbs and decomposes the suspended and dissolved solids. In this way all, or most, of the offensive organic matter is changed to an inoffensive matter.

Trickling filters are classified as low-rate filters and high rate-filters. The principal differences between the two are the hydraulic and organic loadings. Although there is no well defined practice for the loadings, the ranges recommended by FSIWA Subcommittee on Units of Expression are given in Table 7.1 and will be adopted in the design of the filter.

TABLE 7.1

HYDRAULIC AND ORGANIC LOADINGS FOR TRICKLING FILTERS(14)

Type of Filter	Hydraulic Loadings		Organic Loadings	
	Units	Ranges	Units	Ranges
Low rate	Gal. per day per $ft.^2$	25 - 100	Pounds per 100 cu.ft. per day	5 to 25
	Million gal. per acre per day.	1.1-4.4	Pounds per acre-ft. per day	220 to 1,100
High rate	Gal. per day per $ft.^2$	200 to 1000	Pounds per 1000 cu.ft. per day	25 to 300
	Million gal. per acre per day.	8.7 to 44	Pounds per acre-ft. per day.	1,100 to 13,000

High rate trickling filters have been recommended for the following reasons:

(1) The initial cost of the high-rate filters is less than that of low-rate filters.

(2) High-rate filters require less space than the low-rate filters.

(3) Recirculation in the high rate filter takes care of the hourly, daily and seasonal variation in both volume and ~~the~~ strength of the sewage received at the treatment plant.

(4) The effect of sewage temperature on the efficiency of high-rate filters is less than that of the low-rate filters.

(5) The head required to operate high-rate filters is less than that required by low-rate filters(14).

Single stage filters will be adopted because of the following reasons:

(1) The construction and operation cost of single-stage filters is less than that of two-stage filters.

(2) The required strength of effluent can be obtained by single-stage filters.

Although recirculation will increase the initial and operational costs of the treatment plant, it will be adopted in our project because of the following advantages:

(1) Organic matter in the sewage is brought in contact with active biological material on the filter more than once. This increases contact time and seeds the filter with organisms.

(2) It will decrease the opportunity for fly breeding by removing worn-out film.

(3) It will freshen the influent and will thus reduce odors.

(4) It will equalize and reduce the loading and will thus improve efficiency.

(5) It prevents ponding[14].

(6) It will keep the filter working almost continuously. It will also reduce the strength of sewage applied to the filter. This helps to maintain the filter in good condition during periods of fluctuations in loading.

Out of several recirculation systems, the flow diagram presented in Figure 7.1 will be adopted in the design of the filter. This includes recirculation through the primary sedimentation tank which has the following advantages:

(1) It will freshen the sewage and reduce the scum formation in the primary tank.

(2) It will dampen variations in loadings applied to the filter over 24-hours period[14].

(3) The recirculation from the sludge from the hopper of the secondary sedimentation tank to the primary sedimentation tank will remove sludge and reduce depletion of oxygen in the plant effluent.

The following design criteria will also be adopted in the design of the filter.

(1) The filter will be circular in shape.

(2) Material for filter medium will be round hard pieces of crushed stone with all three dimensions as nearly the same as possible, clean and free from dust, screenings and

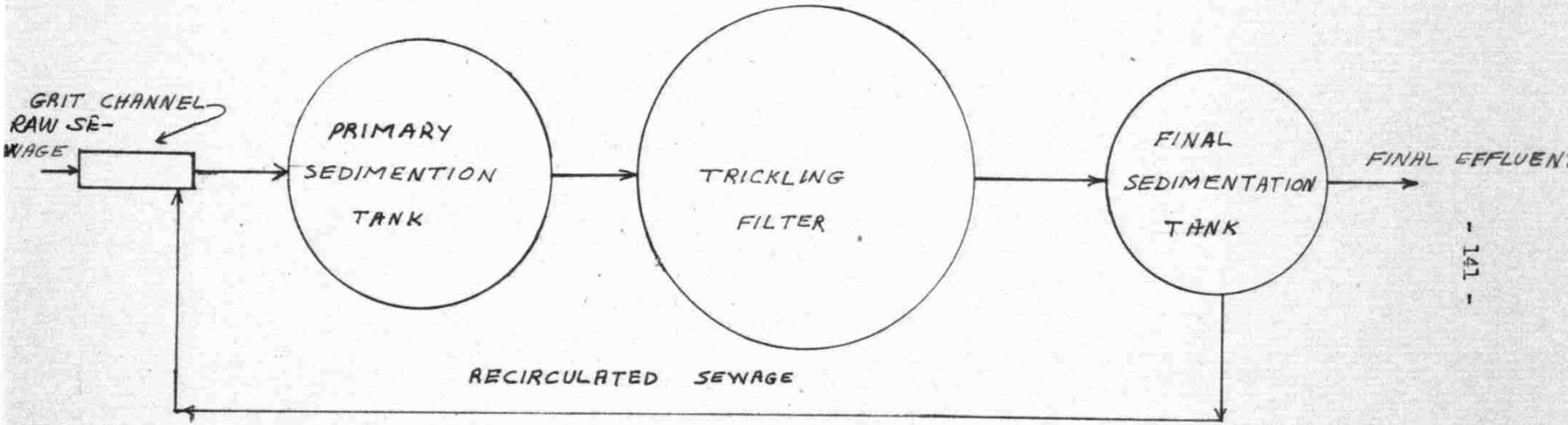

FIG. 7.1

FLOW DIAGRAM OF SEWAGE

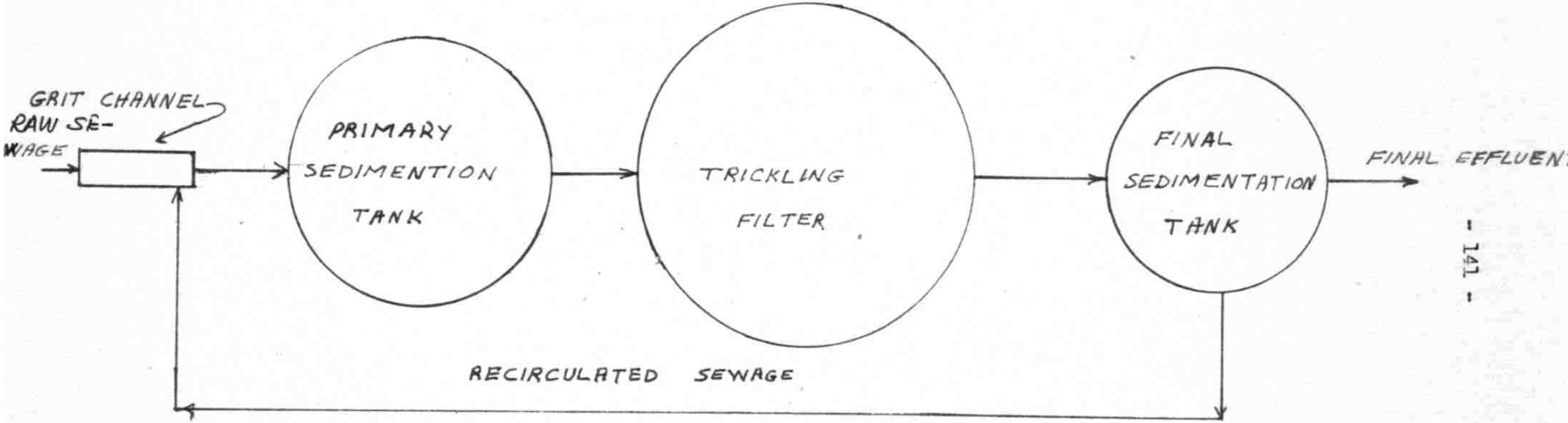

FIG. 7.1

FLOW DIAGRAM OF SEWAGE

other fine material. The size of the stone will be such that 95% or more of the media pass 4-in. mesh screens and retained on $2\frac{1}{2}$-in. mesh screens[14]. It should be of as nearly uniform size as possible and the material should not disintegrate under service conditions, either by breaking into smaller pieces or by crumbling into fine material.

(3) The depth of the ~~filtering~~ filter medium will be from 6 to 8 feet.

(4) Distribution of sewage will be achieved through the use of reaction-jet revolving arms.

(5) The operation of the filter will be continuous by means of multiple pumping arrangement.

(6) Ventilation will be provided by making the area of the openings at the top of underdrains equal to about 20 percent of the floor area, and by installing vents or risers around the inside of the filter wall[14].

(7) The floor of the filter will be of reinforced concrete 6 inch thick; and a reinforcement of half-inch bars spaced 12-inches on centers in both directions is recommended. The floor will slope at a rate of 1 foot per 100 feet, but the total drop will be not less than 2 inches[24].

(8) The walls will be of reinforced cement concrete of 12-inches thickness. They must be water-tight and the drainage

channel must be provided with gates in order to control filter flies and ponding by flooding the filter.

(9) The filter underdrain blocks will be of vitrified clay or concrete and will be set in a thin bedding layer so as to cover the floor completely, and to form continuous channels perpendicular to the drainage channel.

(10) Drainage channels will be provided to carry the filter effluent from the underdrains and to allow air to the underdrains for ventilation. The slope and dimensions of the channel will be designed to produce a velocity of about 3 feet per second[10].

b) Final Sedimentation Tank

It has been pointed out that the trickling filter medium is covered by micro organisms that constitute a zoogleal film. Fine suspended solids are removed and held by this film and acted upon by its bacterial population. The film increases continuously, and after some time becomes heavy with dead organic matter and this begins to "slough" off, and appears in the effluent in the form of a humus-like suspended matter. At times the discharge becomes so great that the filter is said to be unloading.

This humus still exerts some BOD and has to be removed from the effluent prior to final disposal. The principal aim of the final sedimentation tank is to remove such suspended matter from the trickling filter effluent.

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

(1) The floor of the tank will slope towards the center at a rate of 1:2 to facilitate removal of sludge.

(2) The tank will be circular in shape and of the continuous flow type.

(3) Cleaning will be done mechanically by means of a trailing chain.

(4) The surface overflow rate will not exceed 800 gallons per square foot of surface area per day.

(5) The flow rate over the weir will not exceed 50,000 gallons per lineal foot per day.

(6) Four units will be provided for ultimate condition two of which will be constructed for present.

(7) Other design criteria will be similar to that of primary sedimentation tank.

7.1.3 Sludge Treatment

a) Digestion Tank

The most common method of treatment involves the digestion of sludge in specially designed sludge digestion tanks. This process is accomplished by biological action in which the suspended organic matter deposited by sedimentation is liquified and gassified, and the condition of the sludge is so changed that its volume is reduced. The subsequent drying of sludge becomes an easy process producing no nuisances and odors. Gas (methane and carbon dioxide) and fertilizing matter are by-products of the process, and both could be used beneficially. The advantages of sludge digestion include:

(1) Compaction of the sludge and reduction of its water content.

(2) Digested sludge gives up its water more readily and drying on beds can be accomplished more readily and economically.

(3) Bacteria, especially the pathogenic ones, will be reduced substantially.

The optimum conditions for sludge digestion are as follows:

(1) Favourable Temperatures

Since the bacterial population responsible for sludge digestion is mainly of the mesophilic type, it is necessary to maintain the temperature within the mosephilic range; this lies between $90^{o}F$ and $100^{o}F$.

(2) pH

Bacteria are known to thrive best in neutral environments. The most favorable range is between 7.2 and 7.4.

(3) Maintenance of a proper relationship between the acid-forming and methane-forming bacteria through proper control of sludge additions and withdrawals.

The major features to be considered in the design of sludge digestion tank are discussed briefly hereunder:

(1) Amount and Characteristics of Sludge

Sludge contains suspended solids which can be measured but, for purposes of this project, this is not feasible because of the distance. The amount of suspended solids and the volume of sludge will be estimated, therefore, by assuming the figures for normal domestic sewage. Table 7.2 gives the average per capita solids in domestic sewage[6].

TABLE 7.2

AVERAGE PER CAPITA SOLIDS IN DOMESTIC SEWAGE
(grams per capita per day)

State of Solids	Mineral	Organic	Total
1. Suspended	25	65	90
a. Settleable	15	39	54
b. Non-settleable	10	26	36
2. Dissolved	80	80	160
Total	105	145	250

(2) Period of Digestion

Sludge digestion proceeds in almost any range of temperature likely to be encountered, but the time required for satisfactory digestion varies greatly with the temperature. An increase in temperature would substantially reduce the time required for digestion, thereby reducing the capacity of the digestion tank. For most effective results, the temperature should be maintained as uniformly as possible without sudden changes of more than a few degrees. Time and temperature relationships are given in Table 7.3[14].

TABLE 7.3

TIME REQUIRED FOR SLUDGE DIGESTION[14]

Description	Temperature Versus Digestion Period									
	Mesophilic						Thermophilic			
Temperature (^{o}F)	50	60	70	80	90	100	110	120	130	140
Digestion Period (days)	75	56	42	30	25	24	26	16	14	18

(3) Heating of Digester

Since the living organisms accomplish the digestion process, it is obvious that this process will proceed most rapidly at the temperature best suited to the organisms. The mesophilic organisms thrive best between 85^{o} and $95^{o}F$, while the thermophilic type thrives best in the range of 95 to $128^{o}F$. The high temperature are most difficult to maintain and require considerable heat transfer, especially in cold climates. The daily mean temperatures in Sulaimaniya range from $30^{o}C$ ($86^{o}F$) in summer to $8^{o}C$ ($46.4^{o}F$) in winter on the average. In summer the temperature is suitable to obtain optimum digestion within the mesophilic range, while in winter heating is required. However in this project heating will not be adopted for the following reasons:

(i) The cost of providing extra capacity for digestion tanks at $46.4^{o}F$ will probably be less than constructing heating equipment. This is due to the fact that the capacity of the sludge digestion tank is not directly proportional to the digestion time[10].

(ii) The duration of low winter temperatures are short in Sulaimaniya (about three months).

(iii) Operation of the heating system will need skilled labour which is difficult to find in Iraq.

(iv) The operational cost will be low without a heating system.

(4) Gas Collection

Sludge gas produced by sludge digestion tanks is either collected for utilizing or for burning to avoid nuisance. The latter procedure will be adopted in this design for the following reasons:

(i) Heating of sludge will not be practiced in the sludge digestion tanks.

(ii) Fuel cost in Iraq is very low. Hence, there will be no advantage in collecting the sludge gas and utilizing it.

The sludge gas will be drawn off by a pipe to gas burners where it will be burnt to avoid nuisances and possible hazards.

(5) Method of Stirring

Effective mixing of incoming sludge with the contents of the digester is necessary to provide all working organisms with their essential food supply and to maintain a uniform temperature throughout the digester. Moreover, it is essential to keep the entire contents of the digester well mixed for similar reasons. Mixing can be effectively provided by[14]:

(i) Releasing compressed sewage gas through diffusers near the bottom of the digester or in conjunction with a gas lift device.

(ii) Recirculating sludge through an exterior heat exchanger.

(iii) Mechanical mixing or pumping the sludge within the digester.

Mixing by recirculation of sludge gas is not possible since the gas is to be burnt. Recirculation of sludge requires a heat exchanger and a pump. Moreover, the operational cost of such stirring method is high and it needs constant supervision. Mechanical stirring is the simplest method and will be adopted. Details of the mechanical stirrer and its specifications can be furnished by the manufacturer upon request.

(6) Type of Digestion Tank

Two-stage digestion systems employing two tanks operated in series will be adopted. The use of two-stage digestion tanks will provide the following advantages:

(i) The initial cost will be less since only the primary tanks need be equipped with stirrers and covers. The cover will be eliminated from the secondary tank because the amount of gas produced is small.

(ii) Secondary tanks will serve as balancing tanks for the sludge-drying works.

(iii) Separation of supernatant liquor and sludge can be achieved better by using secondary digestion tanks[24].

(iv) The arrangement of having two-stage tanks will reduce the tendency for sludge to short-circut and pass through untreated[25].

(7) Type of Cover

Open digestion tanks are seldom used because of their inability to confine the odors produced by the process. The cover will also reduce the heat losses from the sludge. Two types of covers are usually used-fixed and floating.

Floating covers have been used especially where sludge gas is collected. Since the sludge gas in this project will not be utilized, therefore, fixed covers will be adopted for the primary digestion tanks. Other reasons for selecting such a type of cover are:

(i) They are low in cost and have been found to be satisfactory in practice.

(ii) They do not need to be maintained or operated.

(iii) To overcome the danger of drawing air into the sludge digestion tank, the amount of fresh sludge added to the tank should be equal to the volume of the digested sludge drawn from it. This operation is not difficult to be performed.

Other criteria for the design of sludge digestion tanks are:

(8) Side depth will be at least 20 ft.[14].

(9) A 24-inches freeboard will be provided.

(10) To minimize power driven devices no mechanical sludge-removal equipment will be used.

(11) To facilitate sludge removal the bottom slope of the tank will be 1:2 .

(12) In the digestion tank roofs, two manholes with 30-inch openings will be constructed.

b) Digested Sludge Disposal

Digested sludge normally has a slight musty odor. The moisture content of it varies, but usually is less than that of the raw sludge (about 93%). Moreover, digested sludge is of a granular texture and yields water quite readily. Hoever, its removal is still a problem and needs to be accomplished properly and economically. Digested sludge will either be used as fertilizer or can be disposed as a sanitary land fill. Before doing so it needs to be dried to a suitable degree of moisture. Among the important methods used for drying the sludge are:

(1) Drying on sludge beds.

(2) Filter pressing.

(3) Heat drying.

(4) Vacuum filtration and flash-drying.

(5) Centrifuge drying.

Other methods, which are less expensive and applicable to small works only and do not require any construction, are: disposal to permanent lagoons, drying on earth plots, and pumping in wet form to farmers(13). These methods are liable to cause nuisance and, hence, will not be adopted. Table 7.3 presents an approximate relative costs of the various means of sludge disposal as given by Escritt in terms of percentage of the cost of drying on beds(13).

TABLE 7.3

APPROXIMATE RELATIVE COST IN TERMS OF PERCENTAGE OF DRYING BEDS (13)

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

Filter pressing, heat drying and vacuum filtration are not only more expensive than drying beds but they are also more difficult to maintain and operate. Drying beds will, therefore, be proposed.

The area required for drying beds depends upon the duality of the digested sludge, climatic conditions - such as temperature, winds and relative humidity - and upon the requirements of the local authority. The area required for sludge drying beds for high rate filters ranges between 1.0 to 1.5 sq. ft. per capita[12]. For Sulaimaniya one square foot per capita will be quite sufficient because the required time for sludge drying expected to be short on account of the higher solar intensity and low relative humidity.

The following design criteria will be adopted for the design of sludge drying beds:

(1) The ground floor of the drying beds will be graded to the underdrains for efficient collection of sludge water.

(2) Embankments will be of concrete watertight walls which will extend 12-inches above the sand and at least 6 inches below the surface.

(3) Underdrains will be precast blocks of concrete. The main underdrains will be 6 inches in diameter and will be laid down diagonally under the drying beds. The lateral

underdrains will be 4 inches in diameter and will join the main at acute angles.

(4) The bottom of the drying beds will be constructed of crushed stones. The latter will be graded in three sizes: $1\frac{1}{2}$", 3/4", $\frac{1}{4}$" and laid 12 inches deep with the coarses at the bottom and the finest on top. The depth of each layer will be 4 inches.

(5) The upper layer of the sludge drying bed will consist of 12 inches of sand. The sand will be of good quality and meet the following specification(14):

(i) Washed (less than 1% dirt by volume).

(ii) Uniformity coefficient not over 4.0 and preferably 3.5 .

(iii) Effective size of grains between 0.3 and 0.75 mm.

(6) Shear gates will be used to provide an economical and satisfactory control of sludge flow to the bed(14).

(7) Gravity pipelines will be laid on grade to give a velocity of at least 2.5 fps, for proper distribution of sludge(14).

(8) Impervious spalsh plates of concrete will be constructed to distribute the flow of wet sludge uniformaly over the drying beds without causing sand scouring(14).

(9) Multiple beds will be used to provide operating flexibility.

(10) The dimensions of drying beds will be 30 x 60 feet.

(11) Cake removal will be performed by dump cars which will move on a narrow-gage track. The cars will be pushed by hand to a point of disposal.

7.1.4 Chlorination

Among the uses of chlorine in sewage treatment is disinfection. Disinfection is practiced for the following purposes: (a) where water supplies are endangered, (b) if there are shellfish beds in the sewage polluted water, (c) to protect recreation waters such as bathing beaches, and (d) when the effluent is used for crop irrigation[14].

Since the treated effluent at Sulaimaniya may be used for irrigation low grown plants, it would be necessary to chlorinate the treated sewage before disposal into Wadi Karaizah Washak. To be effective, chlorine requires a contact time of not less than 15 min. and its residual should not be less than 0.2 to 1.0 mg/L[14]. Under such conditions it has been found that the chlorination of the effluent from secondary treatment will generally result in more than a 99.9 percent reduction in the coliform content of the effluent[14]. The chlorine dosage required for disinfection of the effluent from high rate trickling filters is about 15 mg/L[9].

Using this rate of chlorine the present amount required

$$= \frac{2.16 \times 10^6 \times 15 \times 8.34}{10^6}$$

$$= 270 \text{ lbs. per day.}$$

Ultimate requirement of chlorine

$$= \frac{9.4 \times 10^6 \times 15 \times 8.34}{10^6}$$

$$= 1075 \text{ lbs. per day.}$$

It is recommended that chlorination should be practiced only during the dry season when the sewage effluent will be used for irrigation. This period usually ranges between 3 to 4 months per year.

Chlorine is available in 100 or 150 lbs cylinders which should be made of seamless with a cylinder valve containing a fusible metal safety plug. Cylinders should be stored in a cool place, which should be well - ventilated and away from heat, sources, walkways, elevators, stairway and ventilating system intakes. For storage, a 20 m^2 room will be provided.

The chlorine feeding equipment will be of the vacuum feed type. The details of such a feeder will be supplied by the manufacturers.

7.2. Design of Treatment Plant Units

7.2.1 Design of Primary Treatment

a) Grit Channel

For a compact unit, screens and grit channel, it will be necessary to design the grit channel first and the screens next.

The design of the grit channel in accordance with the method outlined by Babbitt[12] the following data is essential:

Present population	=	57.000
Ultimate population	=	112,500
Present sewage flow	=	36 g.p.c.p.d.
Ultimate sewage flow	=	80 g.p.c.p.d.
Present industrial waste	=	58,010 g.p.d.
Ultimate industrial waste	=	400,000 g.p.d.

$$Q \text{ min.} = \frac{57,000 \times 36 + 58,010}{7.480 \times 24 \times 60 \times 60} \times 0.5$$

$$= 1.633 \text{ cfs.}$$

$$Q \text{ max.} = \frac{80 \times 112,500 + 400,000}{7.480 \times 24 \times 60 \times 60} \times 2.5$$

$$= 36.4 \text{ cfs.}$$

For velocity control by Parshall flume two units, each consisting of two channel, must be provided. To control the velocity in each unit the following formula will be used[12]:

$$\frac{Q\ min.}{Q\ max.} = \frac{1.1\ (Q\ min/4.1\ W)^{2/3} - Z}{1.1\ (Q\ max/4.1\ W)^{2/3} - Z}$$

Where Q min. = minimum rate of sewage flow.

Q max. = maximum rate of sewage flow.

W = flume throat.

Z = distance as shown in Drawing No. 10.

Q min. for each unit $= \frac{1.633}{2} = 0.8165$ cfs.

Q max. for each unit. $= \frac{36.4}{2} = 18.2$ cfs.

Assume the flume with 9 inches feet throat.

$$\frac{0.8165}{18.2000} = \frac{1.1\ \left(\frac{0.8165}{4.1 \times 0.75}\right)^{0.67} - Z}{1.1\ \left(\frac{18.2}{4.1 \times 0.75}\right)^{0.67} - Z}$$

$$0.0448 = \frac{1.1 \times 0.412 - Z}{1.1 \times 3.30 - Z}$$

$$0.163 - 0.0448Z = 0.454 - Z$$

$$Z = \frac{0.291}{0.9552}$$

$$= 0.305 \text{ ft. } (9.6 \text{ cm}).$$

To determine the depth the following expression is used[12]:

$$d = 1.1\left(\frac{Q}{4.1\ W}\right)^{2/3} - Z$$

Where d $=$ depth of flow at a rate of Q

d max. $= 1.1 \left(\frac{Q \text{ max.}}{4.1 \times 0.75}\right)^{0.67} - 0.305$

$= 1.1 \left(\frac{18.2}{4.1 \times 0.75}\right)^{0.67} - 0.305$

$= 3.63 - 0.305$

$= 3.325$ ft. (1 m).

d min. $= 1.1 \left(\frac{Q \text{ min.}}{4.1 \times 0.75}\right)^{0.67} - 0.305$

$= 0.453 - 0.305$

$= 0.148$ ft.

The width of the channel is obtained from the following expression(12):

b $= \frac{Q \text{ max.}}{d \text{ max. } V} = \frac{Q \text{ min.}}{d \text{ min. } V}$

Where b $=$ width of the grit channel.

V $=$ the velocity in the channel $= 1$ ft./sec.

b $= \frac{18.2}{3.325} = \frac{0.8165}{0.1480}$

$= 5.5$ ft.

The width of each channel will, therefore be

$= 2$ ft. 9 in (84 cm).

Flow should be greater than Q max. in this formula[12].

$$Q\ max. = 130\ WN^{3/2}$$

N is found from tables, and

$$= 0.375\ (11\ cm)\ \text{for 9-inch throat.}$$

$$Q\ max. = 130 \times 0.75 \times (0.375)^{1.5}$$

$$= 22.5\ cfs.$$

Since this rate of flow is larger than the maximum rate of 18.2 cfs., free-flow conditions will exist[12].

$$B = 1.5\ (Q\ max.)^{1/3}$$

Where B as shown in Drawing No. 8.

$$B = 1.5\ (18.2)^{0.33}$$

$$= 3.94\ ft.\ (1.2\ m).$$

Since the value of B is more than 2 ft. it is satisfactory[12].

From tables[12]:

$$D = 1'\ 10\ s/8'' = 1.87\ ft.$$

$$B = 2'\ 10'' = 2.83\ ft.$$

$$\text{Ratio}\quad \frac{D}{B} = \frac{1.87}{2.83} = 0.662$$

In the design the ratio D/B should be 0.662.

Therefore,

$$D = 3.94 \times 0.662 = 2.6\ ft. (80\ cm)$$

$$F = 1\ ft.\ (30\ cm).$$

$$G = 1.5\ ft.\ (46\ cm).$$

$$C = 1.25\ ft.\ (38\ cm).$$

$K \quad = 0.25$ ft. (7.6 cm).

$Q \quad = 4.1\ WHA^{3/2}$

Therefore

$$HA = \left(\frac{Q}{4.1\ W}\right)^{2/3}$$

$$HA = \left(\frac{18.2}{4.1 \times 0.75}\right)^{0.67}$$

$$= 3.3 \text{ ft. (1 m).}$$

To prevent submergence $dc + K > dc'$ (12)

$$dc = \sqrt[3]{\frac{Q^2}{gb^2}}$$

where

$b \quad$ = breadth at flume

$\quad = 0.75$ ft.

$g \quad$ = acceleration due to gravity

$\quad = 32.2$ ft./sec^2

$Q \quad$ = descharge in cfs.

$$dc = \sqrt[3]{\frac{18.2^2}{32.2 \times 0.75^2}}$$

$$= 2.54 \text{ ft.}$$

$$dc' = \sqrt[3]{\frac{18.2^2}{32.2 \times 2.6^2}}$$

$$= 1.14 \text{ ft.}$$

$$dc + K = 2.54 + 0.25 = 2.79 \text{ ft.}$$

K = 0.25 ft. (7.6 cm).

Q = $4.1\ WHA^{3/2}$

Therefore

HA = $\left(\frac{Q}{4.1\ W}\right)^{2/3}$

HA = $\left(\frac{18.2}{4.1 \times 0.75}\right)^{0.67}$

= 3.3 ft. (1 m).

To prevent submergence dc + K $\rangle$ dc' (12)

dc = $\sqrt[3]{\frac{Q^2}{gb^2}}$

where

b = breadth at flume

= 0.75 ft.

g = acceleration due to gravity

= 32.2 ft./sec^2

Q = descharge in cfs.

dc = $\sqrt[3]{\frac{18.2^2}{32.2 \times 0.75^2}}$

= 2.54 ft.

dc' = $\sqrt[3]{\frac{18.2^2}{32.2 \times 2.6^2}}$

= 1.14 ft.

dc + K = 2.54 + 0.25 = 2.79 ft.

Since dc + K (i.e. 2.79) is greater than dc' (1.14) there will be no submergence.

The settling velocity = 0.1 ft./sec.

Maximum depth = 3.325 ft.

Therefore, the time taken by each particle to settle

$$= \frac{3.325}{0.1} = 33.25 \text{ sec.}$$

The velocity of flow within the channel

= 1 ft./sec.

Therefore, the length of the channel

$$= \frac{33.25}{1}$$

= 33.25 ft. (10.1 m).

The volume of grit is 4 cu. ft. per m.g. of sewage.

Maximum flow per channel $= \frac{9.2}{4} = 2.3$ mgd.

Volume of grit = 4 x 2.3 = 9.2 cu.ft.

For a clearance interval of 7 days

Volume of grit = 9.2 x 7 = 64.4 cu.ft.

Therefore, depth of grit collection

$$= \frac{64.4}{2.75 \times 33.25}$$

= 0.70 ft.

Total depth = 0.70 + 3.320 = 4.02 ft.

say 4 ft. (1.23 m).

Providing freeboard of 1 foot the total depth, therefore,

$$= 4.0 + 1 \qquad = 5 \text{ ft. } (1.53 \text{ m})$$

b) Design of Screens

Clear opening $= 1.0$ inch

Width of each bar $= 3/8'' \qquad = 0.0314$ ft.

Spacing of bars $= \dfrac{1.0}{12} + \dfrac{0.0314}{2}$

$$= 0.0834 + 0.0157$$

$$= 0.09491 \text{ ft.}$$

Width of channel $= 2.75$ ft.

Number of bars $= \dfrac{2.75}{0.0949} - 1 \qquad = 28$ bars.

Space taken by the bars $= 28 \times 0.0314$

$$= 0.88 \text{ ft.}$$

Effective width for screening channel

$$= 2.75 - 0.88$$

$$= 1.87 \text{ ft.}$$

Velocity at minimum flow $= \dfrac{Q \text{ min.}}{\text{Effective width} \times d \text{ min}}$

$$= \frac{\frac{0.8165}{2}}{1.87 \times 0.148}$$

$= 1.46$ ft./sec., which is satisfactory.

Velocity of ultimate flow = $\frac{Q\ max.}{\text{Effective width} \times d\ max.}$

$= \frac{9.1}{1.87 \times 3.325}$

= 1.46 ft./sec. which is satisfactory.

c) Design of Primary Sedimentation Tank

(1) Design for Ultimate Conditions

Recirculation of the sewage = 3 : 1

Ultimate waste water = 9.4 m.g.d.

Volume of waste water to be treated by primary sedimentation tank = 9.4 + 3 x 9.4

= 37.4 m.g.d.

Detention period in the tank

= 2 hours

Provide four tanks each will have a volume

$= \frac{37.4 \times 10^6 \times 2}{4 \times 24 \times 7.48}$

= 10,700 cu. ft.

Adopting 10 ft. (3.04 m) for the side depth, the area of the tank = $\frac{10,700}{10}$

= 10,470 sq. ft.

The bottom of the tank slopes at 1 inch per foot of slope

Diameter of the tank $= \sqrt{\dfrac{10,470 \times 4}{\pi}}$

$= 115$ ft. (35 m).

Overflow rate $= \dfrac{9.4}{10,470}$

$= 895$ gal. per sq. ft. per day

This value is satisfactory.

Weir Overflow $= \dfrac{9.4}{115\pi} \doteq \dfrac{9.4}{360}$

$= 26,000$ gal. per lineal foot per day.

This value is satisfactory.

(2) Design for Present Conditions

Recirculation ratio of the sewage

$= 6 : 1$

Present sewage flow $= 2.11$ g.m.d.

Present sewage to be treated by primary sedimentation tank $= 2.11 + 6 \times 2.11$

$= 14.77$ m.g.d.

Provide two tanks, the volume of each being 104,700 cu.ft.;

Diameter of each tank $= 115$ ft. (35 m).

and depth $= 10$ ft. (3.04 m)

Detention time $= \dfrac{104,700 \times 7.48 \times 24}{7.385 \times 106}$

$= 3.54$ hours.

Overflow rate $= \dfrac{7.385 \times 10^6}{10,470}$

$= 707$ gal. per sq. ft. per day.

This is satisfactory.

Weir overflow rate $= \dfrac{7.385 \times 10^6}{115\,\pi}$

$= 20,400$ gal. per lineal ft. per day.

(3) Design of the Inlet Baffle

Ratio of inlet baffle to tank diameter

$= 0.15$

Therefore , the diameter of inlet baffle

$= 0.15 \times 115 \quad = 1.73$ ft.

Depth of baffle $= 4$ ft.

7.2.2 Design of Secondary Treatment

a) Trickling Filters

(1) Design for Present Conditions

Present domestic sewage = 2.052 m.g.d.

Present industrial waste = 0.058 m.g.d.

Total present waste water = 2.11 m.g.d.

Present domestic loading = 9,700 lb. of BOD

Present industrial loading = 500 lb. of BOD

Overflow rate $= \frac{7.385 \times 10^6}{10,470}$

$= 707$ gal. per sq. ft. per day.

This is satisfactory.

Weir overflow rate $= \frac{7.385 \times 10^6}{115 \pi}$

$= 20,400$ gal. per lineal ft. per day.

(3) Design of the Inlet Baffle

Ratio of inlet baffle to tank diameter

$= 0.15$

Therefore, the diameter of inlet baffle

$= 0.15 \times 115 = 1.73$ ft.

Depth of baffle $= 4$ ft.

7.2.2 Design of Secondary Treatment

a) Trickling Filters

(i) Design for Present Conditions

Present domestic sewage = 2.052 m.g.d.

Present industrial waste = 0.058 m.g.d.

Total present waste water = 2.11 m.g.d.

Present domestic loading = 9,700 lb. of BOD

Present industrial loading = 500 lb. of BOD

Total Loading $= 10,200$ lb. of BOD

Present strength of sewage $= \dfrac{10,200 \times 10^6}{2.11 \times 10^6 \times 8.34}$

$= 580$ ppm. of 5 day BOD

The detention time of the secondary sedimentation tank is 1.5 hours. The percentage removal of suspended matter and 5-day BOD by settling will be = 24 percent(9). BOD applied to the secondary sedimentation tank is, therefore, $= 15 \times \dfrac{100}{76} = 19.7$ ppm. Assuming a recirculation ratio of 6 : 1

The recirculated effluent will be taken from the bottom of the secondary sedimentation tank.

The effluent discharged by the secondary sedimentation tank is 15 ppm, the BOD of the recirculated effluent is, therefore,

$$= 19.7 + \frac{19.7 - 15}{6}$$

$$= 19.7 + \frac{4.7}{6}$$

$$= 20.50 \text{ ppm.}$$

Average BOD applied to the primary sedimentation tank

$$= \frac{580 + 20.50 \times 6}{7}$$

$$= \frac{580 + 123}{7}$$

$$= \frac{703}{7} \quad = 100.4 \text{ ppm.}$$

The detention time of primary sedimentation tank is 2 hours; it is, therefore, the percentage removal of BOD = 36 percent[9]

BOD applied to the filter = 100.4 x .64

= 64 ppm.

Q (average) = 2.11 + 6 x 2.11

= 2.11 + 12.66

= 14.77 m.g.d.

For the present provide two filters each with a capacity of

$$= \frac{14.77}{2} = 7.385 \text{ m.g.d.}$$

Total applied BOD to each filter

= 64 x 8.34 x 7.385

= 3930 lbs.

Adopt diameter of filter as 150 ft. (45.7 m) and depth of 6 ft. (1.83 m).

Volume of filter $= \frac{\pi\, 150^2}{4} \times 6$

= 106,000 cu. ft.

Organic loading on the filter

$$= \frac{3930}{106{,}000} \times 1000$$

= 37 lb. of BOD/1000 cu.ft. of filter medium. This value is satisfactory.

Area of the filter $= \frac{150^2 \pi}{4}$

$= 17,700$ sq. ft.

Hydraulic laoding on the filter

$= \frac{7.385 \times 43,560}{17,700}$

$= 18.2$ m.g.a.d.

This value is satisfactory.

The reduction of BOD by the trickling filter and secondary sedimentation tank at 37 lbs. BOD/1000 cu.ft. of filter medium and with R as 0[9] = 74.5 percent. Therefore, the strength of the final effluent

$= 64 \times 0.255$

$= 16$ ppm.

This value is satisfactory.

(2) Design for Ultimate Conditions

Ultimate domestic sewage = 9 m.g.d.

Ultimate industrial waste water

= 0.4 m.g.d.

Total waste water = 9 + 0.4 = 9.4 m.g.d.

Ultimate domestic loading = 19,100 lbs of 5 days BOD

Ultimate industrial loading = 850 lbs of 5 days BOD

Total ultimate loading = 19,950 lb. of 5 day BOD

Ultimate strength of sewage $= \dfrac{19950 \times 10^6}{9.4 \times 10^6 \times 8.34}$

$= 255$ ppm.

BOD applied to the secondary sedimentation tank

$= 19.7$ ppm.

Assuming a recirculation ratio of 3 : 1

then, BOD of the recirculated effluent

$= 19.7 + \dfrac{19.7 - 15}{3}$

$= 19.7 + 1.5$

$= 21.2$ ppm.

Average BOD applied to the primary sedimentation tank

$= \dfrac{255 + 21.2 \times 3}{4}$

$= \dfrac{255 + 63.6}{4}$

$= 79.5$ ppm.

BOD removal by primary sedimentation tank(9)

$= 27$ percent

BOD applied to the filter $= 79.5 \times 0.73$

$= 58$ ppm.

Q average $= 9.4 + 9.4 \times 3$

$= 9.4 + 28.2$

$= 37.6$ m.g.d.

For ultimate conditions provide four filters each with a

a capacity of $\dfrac{37.6}{4}$ $= 9.4$ m.g.d.

Total applied BOD to each filter

$= 58 \times 8.34 \times 9.4$

$= 4550$ lbs.

Adopt diameter of filter as 150 ft. (45.7 m) and depth of 6 ft. (1.83 m).

Volume of filter $= 106{,}000$ cu. ft.

Organic loading on the filter

$$= \frac{4550}{106{,}000} \times 1000$$

= 43 lbs. of BOD/1000 cu.ft. of filter medium. This value is satisfactory.

The reduction of BOD by the trickling filter and secondary sedimentation tank at 43 lbs. of BOD/1,000 cu.ft. of filter medium loading and with R as 0[9] = 73 percent. Therefore, the strength of the final effluent

$= 58 \times 0.27 = 15.6$ ppm.

This value is satisfactory.

Area of the filter $= 17{,}700$ sq. ft.

Hydraulic loading $$= \frac{9.4 \times 43{,}560}{17{,}700}$$

= 23.2 m.g.a.d.

This value is satisfactory.

(3) Design of Pipes

(i) Inlet Pipe for the Trickling Filter

Present flow per filter = 7.385 m.g.d.

= 11.57 cfs.

Ultimate flow per filter = 9.4 m.g.d.

= 14.53 cfs.

Adopt 30 inches pipe, the area of it

$$= \frac{\pi\, 2.5^2}{4} = 4.92 \text{ ft.}^2$$

Present velocity $= \frac{11.57}{4.92} = 2.36$ ft/sec.

Ultimate velocity $= \frac{14.53}{4.92} = 2.96$ ft/sec.

The allowable maximum velocity is 4 ft,/sec.[14].

The size of the adopted pipe is, therefore, satisfactory.

(ii) Distribution Pipe System for the Filter

Adopting a rotary reaction type distributor with 4 arms at right angles, then:

Maximum flow per filter = 14.53 cfs.

Flow per pipe $= \frac{14.53}{4} = 3.632$ cfs.

Adopting 15 inches size pipe, then area

$$= \frac{\pi\, 1.25^2}{4}$$

$$= 1.232 \text{ ft.}^2$$

Maximum velocity $= \frac{3.632}{1.232}$

= 2.95 ft./sec.

Since the maximum velocity is less than 4 ft./sec. it is satisfactory[14].

(iii) Size and Number of Nozzles

The size of nozzles will be = $\frac{7}{8}$ inch with $\frac{3}{8}$ inch spindles[23].

The discharge through spray nozzles can be represented by the standard orifice formula[23]:

$$Q = Cn\ An \sqrt{2ghn}$$

where Q = discharge from the orifice in cu. ft. per sec.

Cn = coefficient of the discharge = 0.67[3]

An = area of nozzle in sq. ft.

g = acceleration due to gravity.

hn = head on centre line of orifice = 6 ft.

$$Q = 0.67 \times 0.785 \times \frac{1}{144}\left[\left(\frac{7}{8}\right)^2 - \left(\frac{3}{8}\right)^2\right]\sqrt{32.2 \times 2 \times 6}$$

$$= 0.0448 \text{ cfs.}$$

$$\text{Number of nozzles per arm} = \frac{\text{Flow per arm}}{\text{Flow per nozzle}}$$

$$= \frac{3.632}{0.0448}$$

$$= 81 \text{ nozzles.}$$

$$\text{Length of arm} = \frac{150}{81} = 75 \text{ ft.}$$

$$\text{Spacing between nozzles} = \frac{75}{81} = 0.93 \text{ ft.}$$

(28 cm).

(iv) Depth of Underdrain

Radius of filter = 75 ft.

Slope of floor of the filter

= 1%

Depth of the filter at the middle

= 6 ft.

Depth of the filter at its wall

= 6 + 0.75

= 6.75 ft. (2.06 m).

(v) Effluent Pipe

Q max. = 14.53 cfs/filter

Adopting 30 inches pipe, then the area

= 4.92 ft.^{2}

Velocity $= \frac{14.53}{4.92}$

= 2.96 ft./sec. which is satisfactory.

(vi) Periphery Channel

Area of section = 4.92 ft.^{2}

Adopt section = 2.0' x 2.5'

b) Secondary Sedimentation Tank

(1) Design for Ultimate Conditions

Discharge from the filters = 37.6 m.g.d.

Volume of recirculated sewage $= 37.6 \times \frac{3}{4}$

$= 28.2$ m.g.d.

Volume of sewage to be treated by secondary sedimentation tank $= 9.4$ m.g.d.

The detention time $= 1.5$ hours.

The volume of the secondary sedimentation tank

$$= \frac{9.4 \times 10^6 \times 1.5}{7.48 \times 24}$$

$= 78{,}700$ cu. ft.

Provide four tanks the volume of each being

$$= \frac{78{,}700}{4} = 19{,}675 \text{ cu.ft.}$$

Adopt a circular section with similar dimensions to that of Middlesex Council, U.K., and Templewood Hawsley Ltd., U.K. The tank will also have a trailing chain to disturb sludge[13].

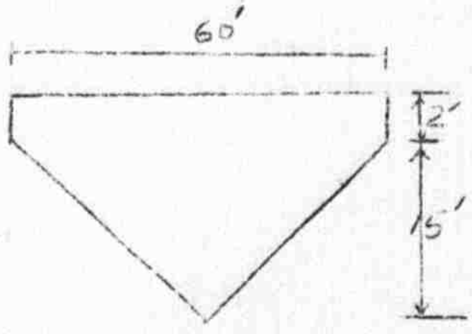

Figure 7.2

Final Sedimentation Tank

Volume of tank $= \frac{\pi D^2}{4} \times 2 + \frac{\pi D^2}{4} \times \frac{D}{4} \times \frac{1}{3}$

Diameter = 60 ft. (18.25 m)

Overflow rate $= \frac{2.35 \times 10^6}{2830}$

= 830 gal. per sq. ft. per day, which is satisfactory.

Weir overflow rate $= \frac{2.35 \times 10^6}{60 \pi}$

= 12,500 gal. per lineal foot per day.

This value is satisfactory.

(2) Design for Present Conditions

Discharge from filters = 14.77 m.g.d.

Volume of recirculated sewage $= 14.77 \times \frac{6}{7}$

= 12.65 m.g.d.

Volume of sewage to be treated by secondary sedimentation tank = 2.12 m.g.d.

Provide two tanks the detention time of each

$= \frac{19,675 \times 7.48 \times 24}{1.06 \times 106}$

= 3.32 hours.

Overflow $= \frac{1.06 \times 10^6}{2830}$

= 375 gal. per sq. ft. per day., which is satisfactory.

Weir overflow rate $= \frac{1.06 \times 10^{6}}{60 \times 3.14}$

= 5,650 gal. per lineal ft. per day which is satisfactory.

7.2.3 Sludge Treatment Plant

a) Design of Sludge Digestion Tanks

From Table 7.2 the total settleable solids per capita per day is given hereunder:

Volatile Solids	0.086 lbs.	or	72.2%
Fixed Solids	0.033 lbs.	or	27.8%
Total	0.119 lbs.		100 %

The settleable solids are removed by the primary sedimentation tank.

From the above mentioned table the amount of non-settleable suspended solids per capita per day is:

Volatile Solids	0.0574 lbs.
Fixed Solids	0.0223 lbs.
Total	0.0797 lbs.

Assuming that 7.5% of the weight of volatile solids is destroyed during filtration and that 87.5% of the remaining weight of solids is captured in the sludge of the secondary sedimentation tank, then

The remaining weight of volatile solids

$= 0.875 \times 0.925 \times 0.0574$

$= 0.0463$ lb. 70%

Fixed Solids $= 0.875 \times 0.0233$

$= 0.0199$ lbs. 30%

Total sludge of secondary sedimentation tank $= 0.0662$ lb.

Adding the sludge from primary and secondary sedimentation tanks, the position of total dry solids in the combined sludge will be as under:

	Primary sedimentation tank	Secondary sedimentation tank	Total	Percentage
Volatile	0.086	0.0463	0.1323	71.4%
Fixed	0.033	0.0199	0.0529	28.6%
Total	0.119	0.0662	0.1852	100 %

The specific gravity of solids is obtained from this expression[6]:

$$Ss = \frac{250}{100 + 1.5\ Pv}$$

Where Ss = the specific gravity of solids.

Pv = the percentage volatile solids in the sludge.

$$Ss = \frac{250}{100 + 1.5 \times 71.4}$$

$$= 1.206$$

The volume of the wet sludge $V = \frac{Ws}{w} \cdot \frac{100\ Sw + P(Ss + Sw)}{(100 - P)\ Ss \cdot Sw}$

Where Ws = weight of dry solids.

P = percentage moisture of the mixed sludge from primary and secondary sedimentation tank is given by Fair and Geyer 97 - 94%. Adopting average figure 95.5%[6].

w = unit weight of water.

Ss = specific gravity of solids

Sw = specific gravity of water taken as 1

$$V \doteq \frac{0.1852}{62.4} \times \frac{100 \times 1 + 95.5\ (1.206 - 1)}{4.5 \times 1.206 \times 1}$$

$$= 0.00297 \times \frac{100 + 19.7}{5.44}$$

$$= 0.0654 \text{ cu. ft. per capita per day.}$$

Assuming that 67% of the volatile matter ~~is~~ is destroyed during digestion, and 25% of the remaining ~~being~~ are converted into fixed solids,[6] then:

remaining volatile solids = (1 - 0.67) x 0.1323

= 0.0432 lb. .... (37.0%)

remaining fixed solids = 0.0529 + 0.25(0.1323-0.432)

= 0.07480 lb. .... (63 %)

Total solids = 0.11900 lb.

Specific gravity of the dry solids ~~is~~ now

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

$$= 1.607$$

Assuming 7.0% solids in the wet sludge, then:

the volume of the wet sludge $V = \frac{0.119}{62.4} \times \frac{100 \times 1 + 93(1.607-1)}{7.0 \times 1.607 \times 1}$

$= 0.0265$ cu.ft. per capita

The volume of digestion tank per capita is given by the following formula(6):

$$C = \left[Vf - \frac{2}{3}(Vf - Vd) \right] t$$

Where C = the basic tank capacity in cubic feet per capita;

Vf = the daily per capita volume of fresh sludge in cubic feet;

Vd = the daily per capita volume of digested sludge in cubic feet;

t = the time in days required for digestion.

Average temperature in summer = 85°F

Average temperature in winter = 46.4°F

Designing for the winter temperature for which the digestion period(6) = 82 days

$$C = \left[0.0654 - 0.67\,(0.0654 - 0.0265) \right] \times 82$$

$$= \left[0.0654 - 0.0260 \right] 82$$

$$= 0.0394 \times 82$$

$$= 3.23 \text{ cu. ft. per capita.}$$

It is recommended that the design capacity should be larger than the calculated one(6). Hence an additional capacity of

24% will be provided, as a safety factor. The capacity of the digestion tank is, therefore, 4 cubic feet per capita.

The present capacity of digestion tanks for domestic waste

$$= 4 \times 57{,}000$$

$$= 228{,}000 \text{ cu. ft.}$$

The present capacity of digestion tanks for industrial waste

$$= 2600 \times 4$$

$$= 10400 \text{ cu. ft.}$$

Total present capacity $= 238{,}400$ cu. ft.

Ultimate capacity of digestion tank for domestic waste

$$= 112{,}500 \times 4$$

$$= 450{,}000 \text{ cu. ft.}$$

Ultimate capacity of digestion tank for industrial waste

$$= 5{,}000 \times 4$$

$$= 20{,}000 \text{ cu. ft.}$$

Total ultimate capacity $= 470{,}000$ cu. ft.

The capacity required for primary digestion tanks are usually more than the capacity of the secondary digestion tanks.

For present conditions adopt three tanks of equal volume out of which two will be used as primary digestion tanks and one will be used as a secondary digestion tank.

The volume of each tank $= \dfrac{238{,}400}{3}$

$$= 79{,}467 \text{ cu. ft.}$$

The dimensions of the tank are as in Fig. 8.3.

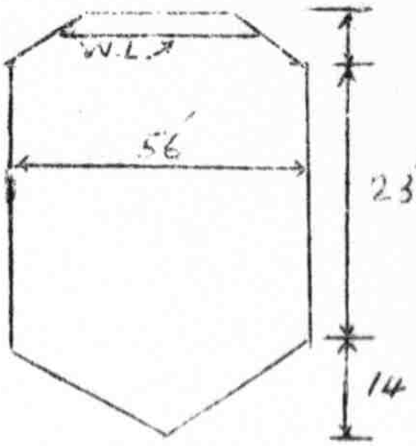

Fig. 8.3

For ultimate conditions six tanks will be provided four of which will be used as primary digestion tanks and two will be used as secondary digestion tanks. The volume of each tank will be the same as that of the first stage.

b) Design of Sludge Drying Beds

Present population = 57,000

Industrial population equivalent

= 2600

Present required area for drying beds

= 59,600 ft.^{2}

Adopt a size of 30 ft. x 60 ft.

No. of drying beds $= \dfrac{59,600}{30 \times 60} =$ 33 say 34

Ultimate population = 112,500

Industrial population equivalent = 5,000

Ultimate required area for drying beds

$$= 117{,}500 \text{ sq. ft.}$$

Adopt the same dimensions as for present requirements

The number of drying beds $= \dfrac{117{,}500}{60 \times 30}$

$$= 65 \text{ beds.}$$

Provide 68 beds.

8 HYDRAULICS OF THE TREATMENT PLANT, PUMPING STATION AND SUPPLEMENTARY WORKS

8.1 Hydraulics of the Treatment Plant

8.1.1 Losses of Head Head in the Treatment Plan Units

The hourly fluctuation of the sewage, as received at the treatment plant, during the day is not expected to be significant. For design purposes the average flow will be considered.

Present Q average	= 2.11 mgd.	= 3.26 cfs.
Ultimate Q average	= 9.4 mgd.	= 14.55 cfs.
Present recirculation ratio of sewage	= 6	
Future recirculation ratio of sewage	= 3	
Present total flow to the treatment units	= 3.26 x 7	= 22.82 cfs.
Ultimate total flow through the treatment plant units	= 14.55 x 4	= 58.20 cfs.

The loss of head through each of the treatment plant units is calculated as follows:

a) From Grit Channel to Sedimentation Tank Distribution Box

Q max.	= 58.20 cfs.
Q min.	= 22.82 cfs.

For cast iron pipes, the tables provide the following data[26]:

For maximum discharge, size of pipe	= 36 inches
Area of pipe	= 7.07 sq. ft.
Max. velocity	= 8.23 ft./sec.
	Which is satisfactory.
Loss of head	= 0.01 ft. per ft. length
For minimum discharge, size of pipe	= 36 inches.
Minimum velocity	= 3.24 ft./sec.
	Which is satisfactory.
Loss of head	= 0.002 ft. per foot length.

Loss of head will be as follows:

(1) Length of pipe = 132 ft. (40 m)

(2) Loss of head at entrance in equivalent length of straight line from nomogram[27] = 52 ft.

(3) Loss of head due to enlargement from 36 pipe to the diameter of distribution box (assuming for ratio of pipe diameter to diameter of distribution box as $\frac{1}{4}$) in equivalent length of straight pipe[27] = 92 ft.

(4) Loss of head in one 90° bend for medium sweep elbow from nomogram in equivalent length of straight pipe[27]

= 80 ft.

Total losses of head in equivalent length of pipe $= 132 + 52 + 92 + 80 = 356$ ft.

hf (max.) $= 356 \times 0.01$

$= 3.56$ ft. (1.09 m).

hf (min.) $= 356 \times 0.002$

$= 0.712$ ft. (22 cm)

b) From Sedimentation Tank Distribution Box to Centre of Sedimentation Tank

Q max. in each pipe $= \frac{58.2}{4} = 14.55$ cfs.

Q min. in each pipe $= \frac{22.82}{4} = 5.705$ cfs.

The entering velocity into the primary sedimentation tank should be low inorder to prevent currents towards the outlet and to distribute the sewage over the cross-section of the tank. It is, therefore, a 36 inches pipe will be adopted. From tables[26] the following data is obtained:

For maximum discharge (Q max.):

Velocity = 2.05 ft./sec.

Loss of head = 0.0007 ft./ft..

For minimum discharge (Q min.):

Velocity = 0.81 ft./sec.

Loss of head = 0.0001 ft./ft.

Loss of head is calculated as follows:

(1) Length of pipe = 98.5 ft. (30 m).

(2) Loss of head at entrance in equivalent length of straight pipe from nomogram[27] = 52 ft.

(3) Loss of head in one 90^0 bend for medium sweep elbow in equivalent length of straight pipe from nomogram[27]

= 85 ft.

Total losses of head in equivalent length of straight pipe

= 98 + 52 + 85 = 235 ft.

hf (max.) = 235 x 0.0007

= 0.1645 ft. (5 cm).

hf (min.) = 235 x 0.0001

= 0.0235 ft. (1 cm.)

c) From Sedimentation Tank Tower to Trickling Filter Distribution Box.

Q max. = 58.20 cfs.

Q min. = 22.82 cfs.

Adopting 36 inches pipe from tables[26], therefore:

For maximum discharge:

Velocity = 8.23 ft./sec.

Loss of head = 0.01 ft./ft. length.

For minimum discharge:

Velocity = 3.24 ft./sec.

Loss of head = 0.002 ft. per ft. length.

Losses of head are calculated as follows:

(1) Length of pipe = 306 ft. (93 m.)

(2) Loss of head at entrance from nomogram[27] (equivalent length of straight pipe) = 52 ft.

(3) Loss of head at entrance (as in a) = 92 ft.

Total loss of head in equivalent length of straight pipe

= 306 + 52 + 92

= 450 ft.

hf (max.) = 450 x 0.01

= 4.5 ft. (1.37 m).

hf (min.) = 0.9 ft. (28 cm).

d) From Filter Distribution Box to Centre of Filter at Gravel Level

Q max./pipe $= \frac{58.2}{4} = 14.55$ cfs.

Q min./pipe $= \frac{22.82}{4} = 5.705$ cfs.

Adopting 24 inches pipe from tables[26], therefore:

For maximum discharge:

Velocity = 4.63 ft./sec.

Loss of head = 0.005 ft. per ft. length.

For minimum discharge:

Velocity = 1.82 ft./sec.

Loss of head = 0.001 ft. per ft. length

Losses of head are calculated:

(1) Length of pipe (including 6 ft. depth of the filter)

= 131 ft. (40 m.)

(2) Loss at entrance from nomogram (in equivalent length of straight pipe)[27] = 40 ft.

(3) Loss of head for one 90° bend from nomogram[27] (in equivalent length of straight pipe)

= 52 ft.

Total losses (in equivalent length of pipe)

= 131 + 40 + 52 = 223 ft.

hf (max.) = 223 x 0.005

= 1.115 ft. (34 cm).

hf (min.) = 223 x 0.001

= 0.223 (7 cm).

Add to this, loss of head in nozzles.

(4) Height of nozzles above gravel = 8 inches (20 cm).

(5) Loss in nozzles[9] = 4 ft.

Therefore:

hf (max.) = 1.115 + 4

= 5.115 ft. (1.56 m).

hf (min.) = 0.223 + 4

= 4.223 ft. (1.29 m).

e) From Centre of Trickling Filter to the Distribution Box of Final Sedimentation Tank

Length of channel	=	476 ft. (145 m).
Slope of the channel	=	0.001
Loss of head	=	0.2380 ft. (7 cm)
Add for depth of the channel	=	2 ft. (61 cm)
Add for thickness of slab	=	6 inches (15 cm)
Total losses of head	=	0.238 + 2.000 + 0.5
	=	2.738 (84 cm)

f) From Final Sedimentation Tank Distribution Box to Centre of Final Sedimentation Tank

$$Q_{max.} = \frac{58.2}{4} = 14.55 \text{ cfs.}$$

$$Q_{min.} = \frac{22.82}{4} = 5.705 \text{ cfs.}$$

Adopt 20 inches pipe from tables[26] the following data is obtained:

For maximum discharge:

Velocity	=	6.6 ft./sec.
Loss of head	=	0.01 ft. per ft.

For minimum discharge:

Velocity	=	2.6 ft./sec.
Loss of head	=	0.003 ft. per ft.

Losses of head are calculated as follows:

(1) Length of pipe = 59 ft. (18 m).

(2) Loss of head at entrance (in equivalent length of straight pipe) from nomogram[27] = 30 ft.

(3) Loss of head due to 90° bend (in equivalent length of straight pipe) from nomogram[27]

= 52 ft.

Total loss of head (in equivalent length of pipe)

= 59 + 30 + 52

= 141 ft.

hf (max.) = 141 x 0.01

= 1.41 ft. (43 cm)

hf (min.) = 141 x 0.003

= 0.423 ft. (13 cm)

g) From Centre of Final Sedimentation Tank to the Wet Well of Pumping Station

Maximum flow per pipe $= \frac{58.2}{2} \times \frac{3}{4}$

$= 29.1 \times \frac{3}{4} = 21.8$ cfs.

Minimum flow per pipe $= \frac{22.82}{2} \times \frac{6}{7}$

$= 11.41 \times \frac{6}{7} = 9.8$ cfs.

Adopt 30 inches pipe from tables[26]:

For maximum discharge:

Velocity = 4.45 ft./sec.

Loss of head = 0.003 ft. per ft.

For minimum discharge:

Velocity = 2.0 ft./sec.

Loss of head = 0.0004 ft. per ft.

Losses of head are calculated as follows:

(1) Length of pipe = 220 ft. (67 m)

(2) Loss of head at entrance (in equivalent length of straight pipe) from nomogram(27) = 52 ft.

(3) Loss of two 90° bends (in equivalent length of straight pipe) from nomogram(27) = 80 x 2 = 160 ft.

Total losses of head (in equivalent length of pipe)

= 220 + 52 + 160

= 432 ft.

hf (max.) = 432 x 0.003

= 1.296 ft. (38 cm).

hf (min.) = 432 x 0.0004

= 0.1728 ft. (5 cm).

h) From Primary Sedimentation Tank to Sludge Digestion Tank Control Room

Volume of combined primary and secondary sludge

= 0.0654 cu.ft. per capita per day.

Maximum volume of wet sludge = 0.0654 x (112,500 + 5,000)

= 7650 cu.ft. per day

Desludging will be performed every 6 hours and the pump will be operated for 20 minutes.

Flow of sludge $= \frac{7700}{4 \times 20 \times 60} = 1.45$ cfs.

Adopt 10 inches pipe. From tables[26]

Velocity = 3.3 ft./sec.

hf = 0.009 ft. per ft.

Losses of head are calculated as follows:

(1) Length of pipe = 400 ft. (122 m).

(2) Loss of head at entrance (in equivalent length of straight pipe) from nomogram[27] = 12 ft.

(3) Loss of head for two 90^{o} bend (in equivalent length of straight pipe) from nomogram[27]

= 18 x 2 = 36 ft.

Total Losses of head (in equivalent length of pipe)

= 400 + 12 + 36

= 448 ft.

hf = 448 x 0.009

= 4.032 ft. (1.23 m).

Therefore the level in the dry well of the control room

= 763.10 - 1.23

= 761.87

8.1.2 Level at the Treatment Plant Units

a) Level at Final Sedimentation Tank

Minimum level in the wet well = 756.12

Maximum level in the wet well = 756.95

Water level at final sedimentation tank = 757.00

Maximum water level at final sedimentation tank distribution box = 757.00 + 0.43

= 757.43

Minimum water level at final sedimentation tank distribution box = 757.00 + 0.13

= 757.13

b) Level at Trickling Filters

Depth of trickling filter = 6 ft. (1.83 m).

Losses from centre of trickling filter to the distribution box of final sedimentation tank

= 84 cm.

Maximum water level at the trickling filter gravel

= 757.43 + 1.83 + 0.84

= 760.10

Maximum water level at the distribution box of trickling filter = 760.10 + 1.56

= 761.66

Minimum water level at the distribution box of
trickling filter = 760.10 + 1.29
= 761.39

c) Level at Primary Sedimentation Tank

Maximum water level in the outgoing distribution box
of primary sedimentation tank
= 761.66 + 1.37
= 763.03

Minimum water level in the outgoing distribution box of
primary sedimentation tank = 761.39 + 0.28
= 761.67

Adopt water level at the primary sedimentation tank
= 763.10

Maximum incoming W.L. at the distribution box of the
primary sedimentation tank = 763.10 + 0.05
= 763.15

Minimum incoming W.L. at the distribution box of the
primary sedimentation tank = 763.10 + 1
= 763.11

d) Level at Grit Channel

Maximum water level in the grit channel
= 763.15 + 1.09
= 764.24

Minimum water level in the grit channel

$= 763.11 + 0.22$

$= 763.33$

8.2. Pumping Station

8.2.1 Design of Sludge Pump

Volume of combined primary and secondary sludge

= 0.0654 cu.ft. per capita per day.

Present volume of wet sludge = 0.0654 (57,000 + 2,600)

= 0.0654 x 59600

= 3900 cu.ft. per day.

Ultimate volume of wet sludge = 0.0654 (112,500 + 5,000)

= 0.0654 x 117,500

= 7,700 cu.ft. per day.

One pump will be provided for the present and ultimate conditions. The design will, therefore, be based on the ultimate condition.

The desludging will be performed every six hours.

Assume the pump will be operated for 20 minutes.

Capacity of pump (gpm) $= \frac{7,700 \times 7.48}{4 \times 20}$

$= 720$

Static head required for the pump (from hydraulic profile) = 772.45 - 761.87

= 10.58 meters. = 35 ft.

Adopting an 18" pipe, the tables[26] provide the following:

Velocity = 4.58 ft./sec.

head loss = 0.023 ft. per ft.

Losses of head are calculated as follows:

Length of pressure pipe = 95 ft. (29 m).

Entrance losses (in equivalent length of straight pipe) from nomogram[27] = 12 ft.

Two 90^{o} bends = 2 x 20 = 40 ft.

Losses of head inside the pumping station (in equivalent length of straight pipe) = 6 ft.

Total losses of head (in equivalent length of straight pipe) = 95 + 12 + 40 + 6

= 153 ft.

hf 153 x 0.0023 = 3.52 ft.

Total head of the pump = 35 + 3.52 = 38.52 ft.
say 40 ft.

From Peerless Pumps Catalogue the following is obtained:

a) Specification of the Pump

Curve No.	= R - 1512
Type of pump	= NCN
R.P.M.	= 1160
Impeller size	= 11" diameter.
Sphere size	= $2\frac{1}{2}$ inch.
Size of suction x discharge	= 5 x 5 x $2\frac{1}{2}$S
Impeller	= V 1418
Efficiency	= 72%
HP	= 10

$$Ns = \frac{N\sqrt{Q}}{h^{3/4}}$$

$$= \frac{1160 \times \sqrt{720}}{(40)^{3/4}}$$

$$= 2020 \text{ r.p.m.}$$

b) Characteristic Curve

Table 8.1 represents the characteristic curve of the pump, and Table 8.2. represents the values of the head capacity curve. These two tables are also represented graphically in Fig. 8.1.

TABLE 8.1

VALUES OF CHARACTERISTIC CURVE

Discharge Q (gpm)	Head (ft)
0	60.0
200	54.5
400	49.0
600	44.0
720	40.0
800	37.5
1000	30.0

TABLE 8.2

CAPACITY HEAD CURVE OF PRESSURE PIPE
OF SLUDGE PUMPING STATION

Discharge Q (gpm)	Head in (ft)
0	35.0
200	35.3
400	36.1
600	37.3
720	38.5
800	39.6
100	41.1

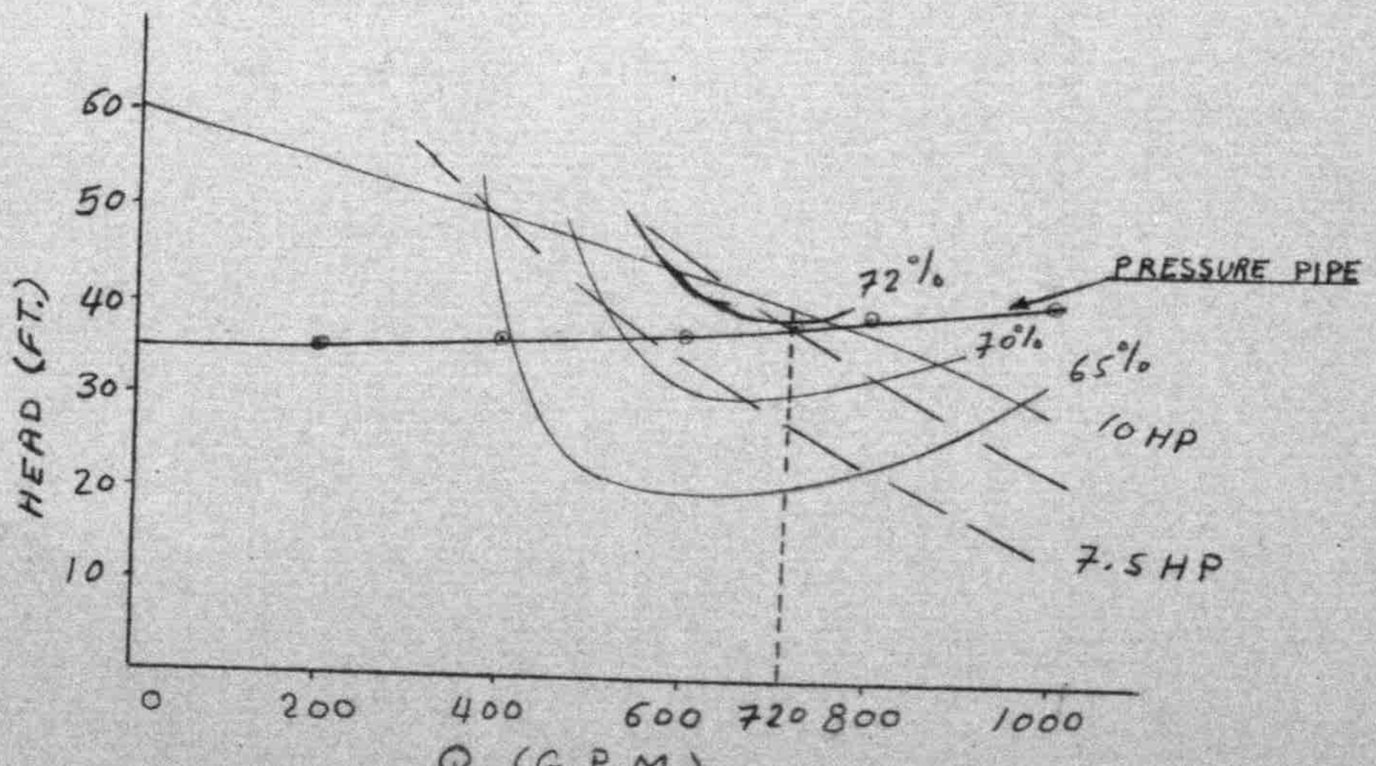

FIG. 8.1

CHARACTERISTIC CURVE OF SLUDGE PUMP

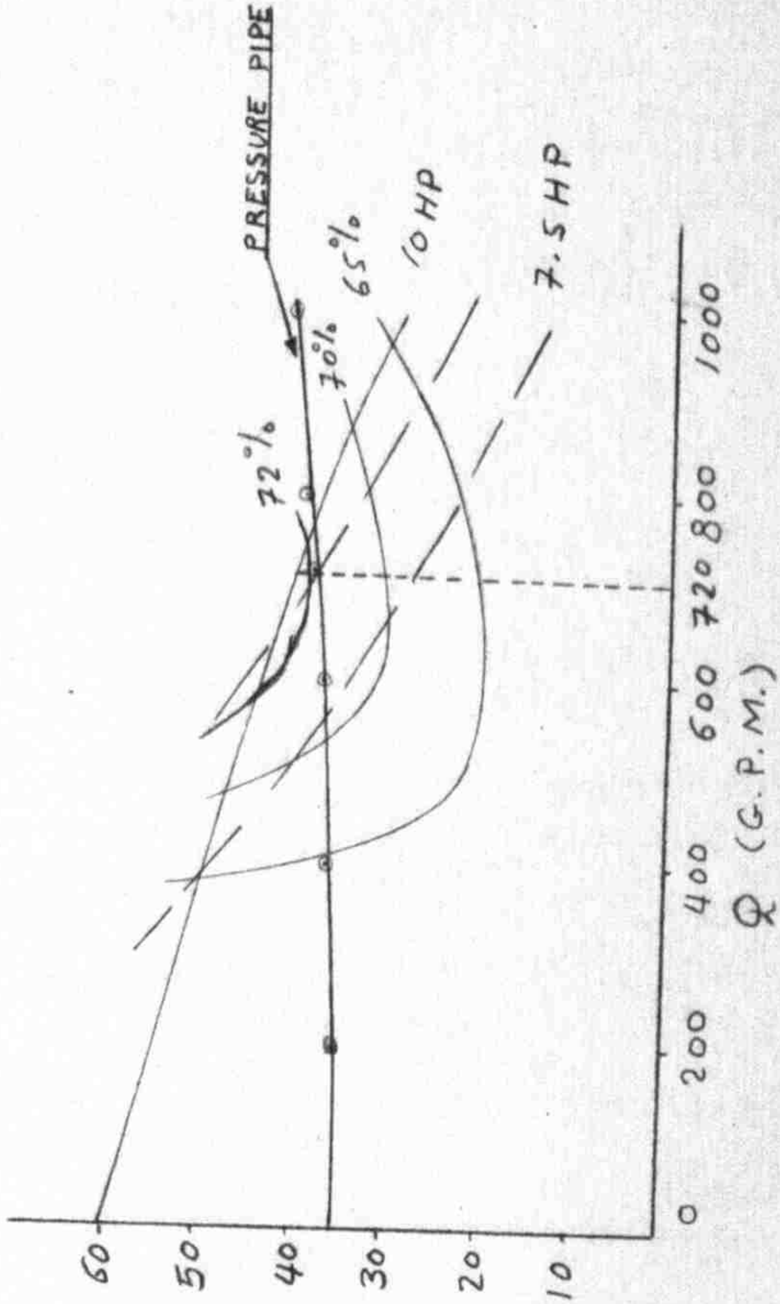

FIG. 8.1

CHARACTERISTIC CURVE OF SLUDGE PUMP

8.2.2 Recirculation Pumping Station

The flow of the sewage through the treatment plant will be by gravity. Pumping is, therefore, required only for the recirculation of treated effluent from the final sedimentation tank to the grit channel sump.

a) Pumping Station Structures

The following structures are necessary for the pumping stations (1) wet well (2) dry well (3) engine room (4) pumps and (5) pressure sewer.

(1) Wet Well

A wet well is required to minimize the fluctuations of load on the pump and to make the automatic operation possible with simple controls. The wet well will be constructed partly underground and partly above ground level.

To minimize depositions of solids the bottom of the wet well is sloped at 45°. Two inspection manholes will be provided in the wet well which will be connected with stair-cases and platforms to facilitate cleaning. In order to reach all the parts of the wet well, three small platforms will be constructed.

The spacing between suction inlets in the wet well will be chosen such that hydraulic interference and deposition of solids between the suction inlet are prevented as far as possible.

The turned down bellmouth inlets will be used, and the distance between the bell and the floor of the wet well will be half the bell diameter.

An emergency outfall to Wadi Karaizah Washak will be as small as possible to prevent septic action in sewage during low flow periods. At the same time the size of the wet well should be large enough to provide facility for cleaning and inspection. The width of the wet well will be 4 ft. (1.20 m) and its length will be equal to the length of the dry well.

(2) Dry Well

A dry well will be constructed under the motor room, and adjacent to the wet well. The pumps will be installed inside the well to maintain prime in the pumps. Moreover corrosion of pumps will be reduced, and service and maintenance will be easy.

The size of the dry well will be goverened by the size and number of pumps required to be installed for ultimate conditions.

(3) Engine Room

The size of the engine room will be the same as that of the dry well. An overhead crane will be installed to move heavy machinery along the room. A small cabin will be provided with a toilet room for the operator. The cabin will be enough to accommodate a table and a chair.

The control panel will be installed as shown on the drawing. A staircase will be provided to go down to the dry well. The slab of the engine room will have openings along the line of the motors for pumps to be lowered by the overhead crane and these openings will be covered by the steel gratings.

(4) Pressure Sewer

Pressure sewer is required to carry the flow from the wet well to the grit channel sump. The materials of the pressure sewer should be able to stand the relatively high pressures to which they will be subjected. Cast iron pipes will be installed to benefit from the following characteristics:

(i) Resistance to internal pressures and water hammer.

(ii) High water tightness.

(iii) More resistance to corrosion than many other pipes.

Centrifugal cast iron piping with a working pressure of 450 psi will, therefore, be provided.

(5) Pumps

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

To provide a constant flow through the treatment plant the sewage will be pumped at a rate that will be adjusted during the hours of minimum and maximum flow. This will affect the recirculation ratio, but the effect on the quality of the effluent is expected to be very small , since the periods of maximum and minimum flow are very short. The arrangement of the pumps will be in such a way that the required recirculated flow will be provided. A standby pump with an adequate capacity will be installed. The design will be made for the ultimate population and only the present requirement will be installed.

In order to provide constant flow through the treatment plant the recirculated flow is calculated to values given in Tables 8.3 and 8.4

TABLE 8.3

PRESENT RECIRCULATED FLOW

Type of flow	Raw Sewage flow (gpm)	Proposed flow to the treatment plant (gpm)	Required recirculated flow (gpm)
Average	1,475	10,325	8,850
Minimum	738	10,325	9,587
Maximum	3,680	10,325	6,645

TABLE 8.4

FUTURE RECIRCULATED FLOW

Type of flow	Raw Sewage flow (gpm)	Proposed flow to the treatment plant (gpm)	Required recirculated flow (gpm)
Average	6,520	26,080	19,560
Minimum	3,260	26,080	22,820
Maximum	16,300	26,080	9,780

To satisfy the present requirement, the following pumps are proposed:

2 pumps with a capacity of 4500 g.p.m. (P1).

2 pumps with a capacity of 3200 g.p.m. (P2).

For the average flow, the pair of (P1) pumps will be operated in parallel, and the pair of (P2) pumps will act as standby. During periods of low flow the pair of (P1) pumps will be operated in parallel to give a discharge at a rate of 9000 g.p.m. If this is not sufficient one of the pumps of (P2) set will be operated in parallel to complement the pair of (P1) pumps. For maximum flow, the pair of (P2) pumps will be operated in parallel.

For ultimate conditions the following set of pumps will be provided:

4 pumps of 4500 g.p.m. (P1).

4 pumps of 3200 g.p.m. (P2).

For average flow, three of the (P1) pumps and two of the (P2) pumps will be operated in parallel so that the total discharge will be at a rate of 19,700 g.p.m. For low flow, three of the (P1) pumps and three of the (P2) pumps will be operated in parallel so that the total discharge will be at a rate of 23,100 g.p.m. During periods of high flow, three of the (P2) pumps will be operated in parallel to provide a combined rate of 9,600 g.p.m. Alternatively, during periods of high flow it would be necessary to operate two of the (P1) pumps, and if is not adequate, a (P2) pump will be operated at the same time.

b) Size and Head Losses of Pressure Sewer

Water level in the wet well	= 756.00
Maximum W.L. in the grit channel	= 764.24
Static head in meters	= 764.24 - 756.00
	= 8.24 = 27 ft.
Length of pressure sewer	= 1,000 ft. (306 m)

Since the recirculated flow contains secondary sludge, the minimum velocity should be above 2.0 ft./sec.

Adopting 36 inches pipe as pressure sewer, then:

Q max.	= 51.0 cfs.
Q min.	= 14.8 cfs.
Velocity at maximum flow	= 7.2 ft./sec.
Velocity at minimum flow	= 2.1 ft./sec.

Calculations for water hammer pressure in cast iron pipes is based on the following formula[26]:

$$h = \frac{12\,V}{g\sqrt{\frac{w}{g}\left(\frac{1}{E'} + \frac{d}{Eb}\right)}}$$

Where h = velocity in feet per second = 6.2 ft./sec,

g = acceleration due to gravity is 32 feet per second per second.

w = density of water or sewage taken as 62.4 pounds per cu. ft.

E' = bulk modulus taken as 3×10^5 pounds per sq. inch.

E = modulus of elasticity of pipe walls taken as 15×10^6 pounds per sq. inch.

d = inside diameter of the pipe in inches = 36"

b = thickness of pipe walls in inches[28] = 1.54"

$$h = \frac{12 \times 6.2}{32\sqrt{\frac{62.4}{32}\left(\frac{1}{3 \times 10^5} + \frac{36}{15 \times 10^6 \times 1.54}\right)}}$$

h = 877 ft. = 383 psi.

Losses of head are calculated as follows:-

(i) Length of pipe = 1000 ft.

(ii) Entrance losses (in equivalent length of straight pipe from nomogram[27] = 52 ft.

(iii) Loss of head due to two 90° bends

= 2 x 80 = 160 ft.

(iv) One gate valve and one check valve

= 40 ft.

(v) Outlet losses from 36 inches pipe to the grit channel

= 55 ft.

(vi) Other losses of head inside the pumping station

= 15 ft.

Total losses of head = 1365 ft.

From tables [26] for maximum flow, losses due to friction

= 0.0077 ft. per ft.

hf (max.) $= 1365 \times 0.0077$

$= 10.5$ ft.

Total head to be sustained by the pressure line

$= 877 + 10.5 + 27$

$= 914.5$ ft. = 400 psi

which is satisfactory.

The capacity head curve of the pressure line is represented in Table 8.5[26].

TABLE 8.5

CAPACITY HEAD CURVE OF PRESSURE LINE

Q (gpm)	Q (cfs)	hf/ft	hf total
0	0	0	27.0
1,000	2.23	0.00005	27.07
2,000	4.46	0.00010	27.13
3,000	6.69	0.00015	27.20
4,000	8.92	0.00020	27.28
5,000	11.15	0.00040	27.55
6,000	13.38	0.00060	27.82
8,000	17.84	0.00090	28.23
10,000	22.30	0.00150	29.05
14,000	31.28	0.00260	30.50
18,000	40.20	0.00470	33.40
22,000	49.10	0.00750	37.20
26,000	58.00	0.01100	42.00

c) Characteristics of Pumps

The pump specifications given in details hereunder were obtained from Peerless Pumps Catalogue.

Specifications for (P1) Pumps:

Curve No.	= NCV
Type of pump	= 2801716
RPM	= 490
Impeller size	= 25" diameter.
Sphere size	= 6 - 3/4"
Size of suction x discharge	= 14 x 14 x 6 $\frac{3}{4}$ M
Impeller No.	= 2601015
Efficiency	= 79%
HP	= 50

The characteristic curve is presented in Table 8.6 and shown graphically in Fig. 8.2

$$N_s = \frac{N\sqrt{Q}}{H^{0.75}} = \frac{490\sqrt{4500}}{(37.5)^{0.75}} = 2170 \text{ r.p.m.}$$

TABLE 8.6[29]

CHARACTERISTIC CURVE FOR (P1) PUMP (4500 gpm)

Q (gpm)	Head ft.
0	52.0
1000	47.0
2000	43.5
3000	40.0
4000	36.0
5000	31.5
6000	27.0

Specifications for (P2) pumps:

Curve No.	= NCV
Type of Pump	= 2801716
RPM	= 490
Impeller size	= 24" diameter.
Sphere size	= 6 - 3/4"
Size suction x discharge	= 14 x 14 x 6 $\frac{3}{4}$ M
Impeller No.	= 2601015
Efficiency	= 76%
HP	= 30
Ns	$= \dfrac{490 \sqrt{3200}}{(37.5)^{0.75}}$
	= 1830 r.p.m.

The characteristic curve for (P2) pump is represented in Table 8.7 and shown on Fig. 8.3.

TABLE 8.7

CHARACTERISTIC CURVE FOR (P2) PUMP (3200 gpm)

Q (gpm)	Head (ft)
0	47.0
1000	41.5
2000	37.0
3000	34.5
4000	30.5
5000	26.0
6000	20.0

The performance curve for two (P1) pumps and for three (P1) pumps have been plotted in Fig. 8.2. Also, the performance curve of two (P2) pumps and three (P2) pumps have been plotted in Fig. 8.3.

8.3 Supplementary Works

8.3.1 Measurement Device

No reasonably accurate evaluation of plant performance can be made without measuring sewage flow. Since a Parshall flume will be constructed to control the velocity in the grit channel it will also be used for measuring the sewage flow. A small well will be provided at a 2/3 A from the flume throat. The well will be provided with a float which will be connected to a device to convert the depth of sewage in the well into the rate of flow.

8.3.2 Laboratory

For proper operation and maintenance of the treatment plant, a laboratory will be constructed. The laboratory is intended to provide the following:

a) Information on operating efficiencies of each unit, and the overall efficiency.

b) Intelligent control of unit operations such as the ratio of recirculation, sludge loading on digesters, digester heating, etc.

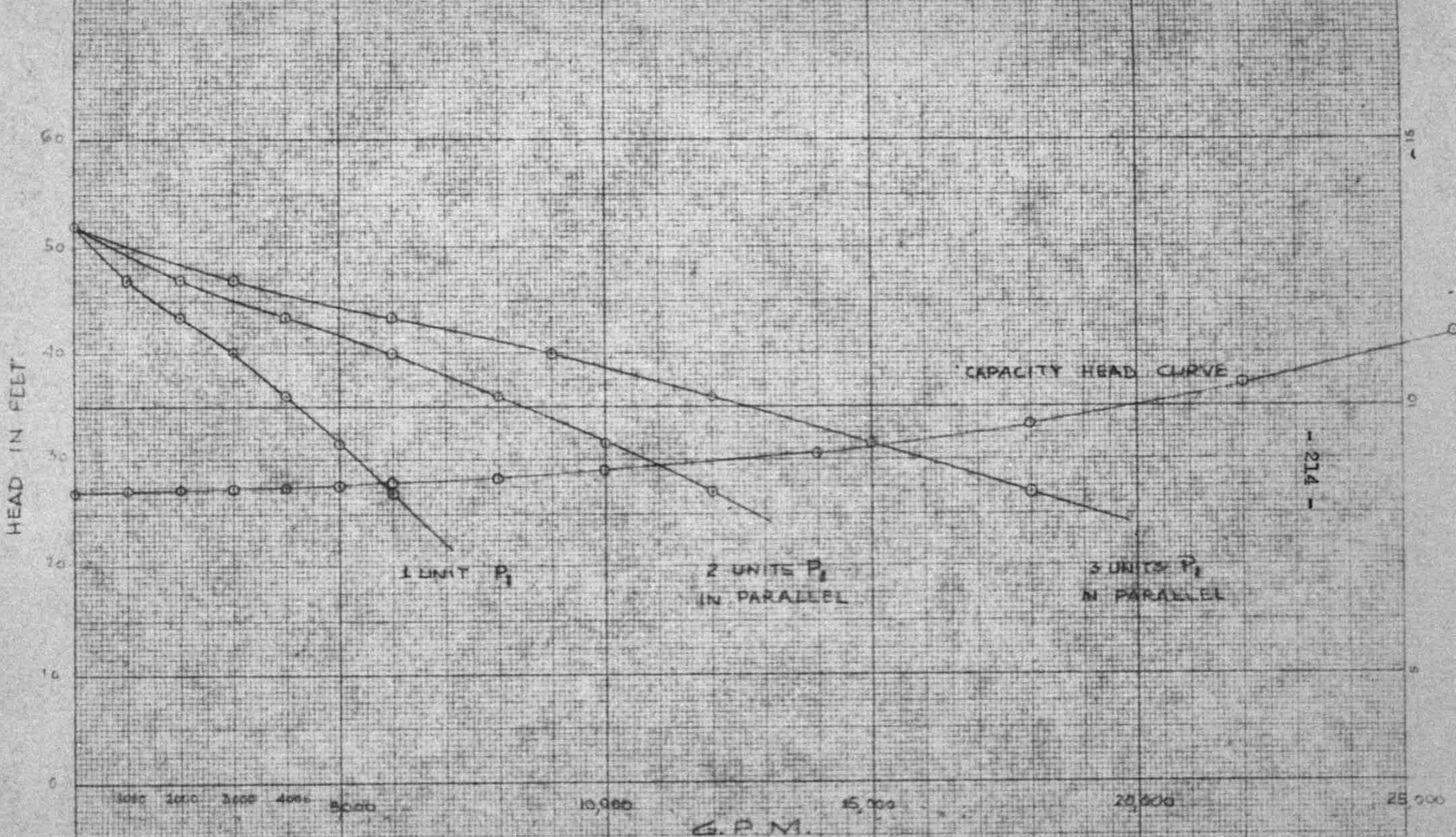

FIG. 8.2.
CHARACTERISTIC HEAD CURVE

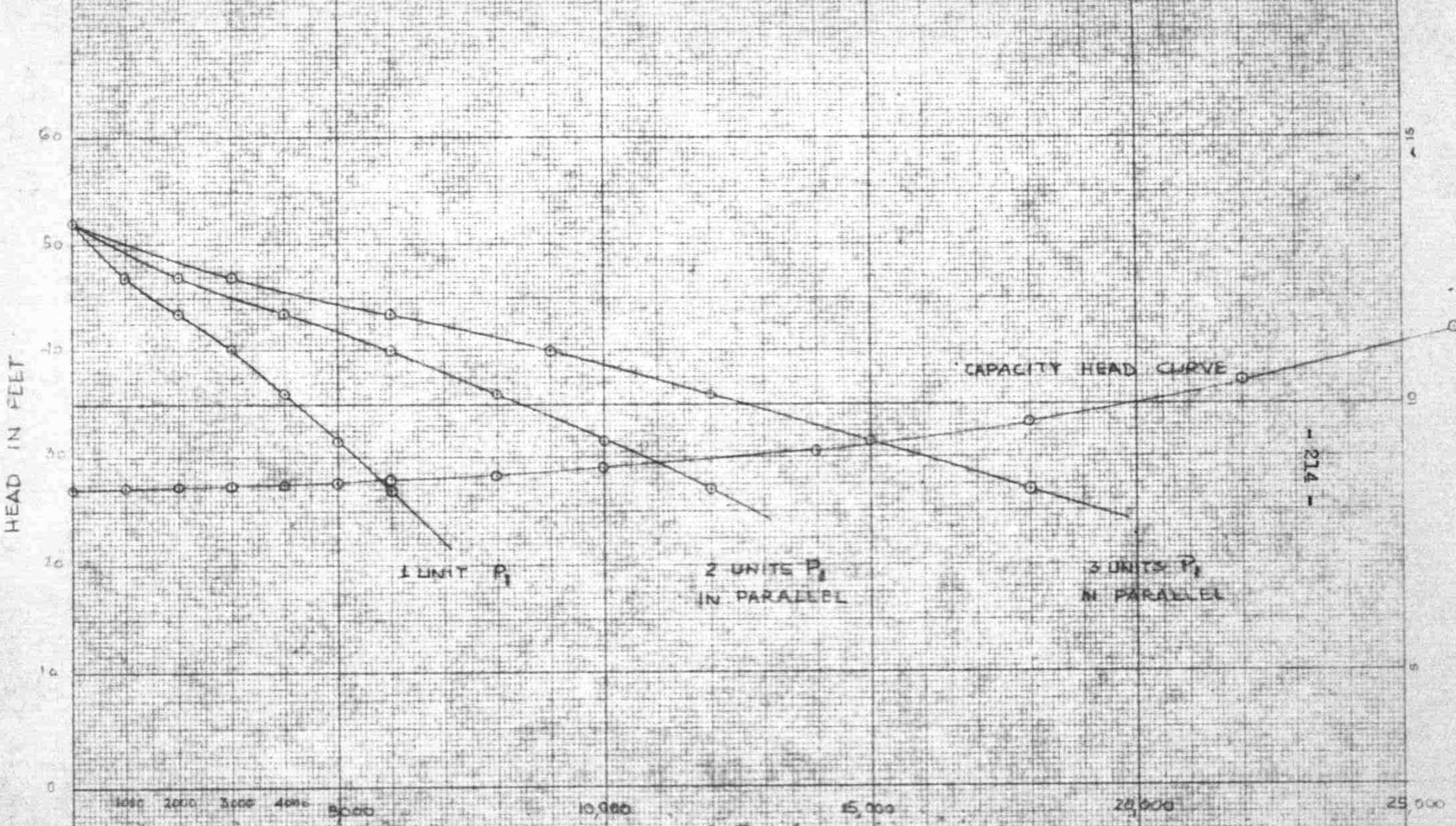

FIG. 8.2.

CHARACTERISTIC HEAD CURVE

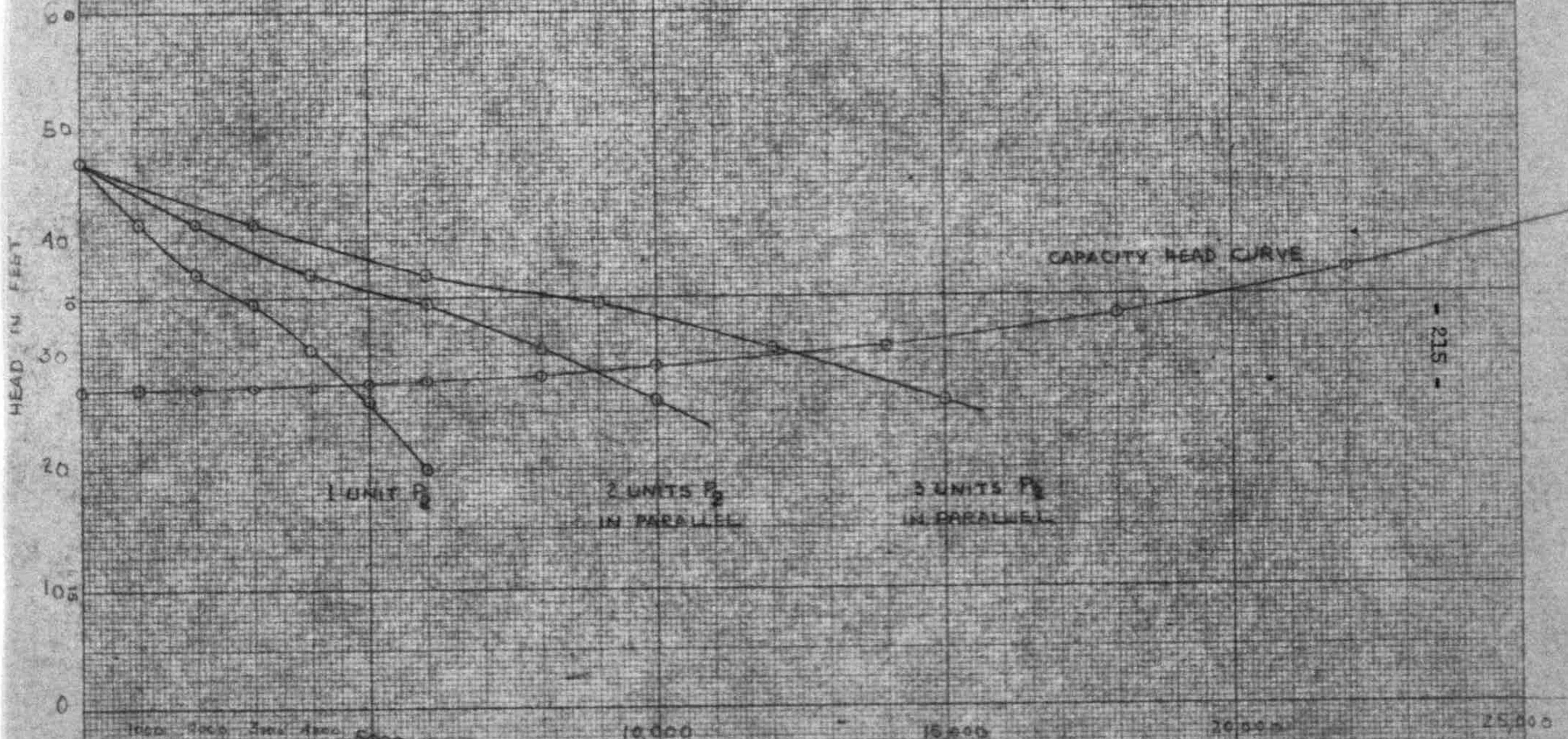

FIG. 8-3
CHARACTERISTIC HEAD CURVE

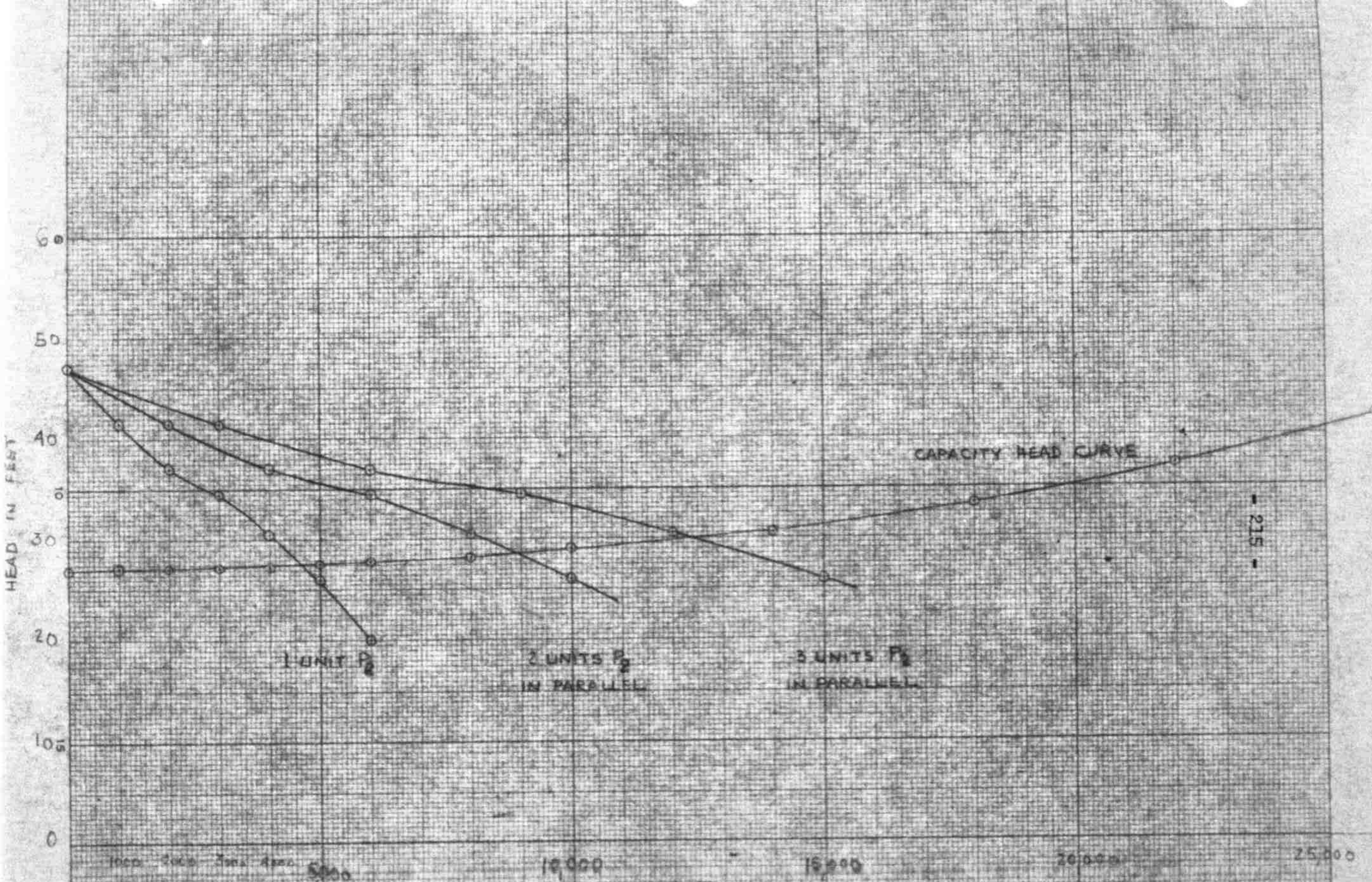

FIG. 8-3
CHARACTERISTIC HEAD CURVE

c) Data for the preparation of performance records.

To satisfy the above requirements, the laboratory should be equipped to run the following tests:

a) BOD.

b) Dissolved oxygen.

c) Suspended solids.

d) Total solids.

e) Settleable solids.

f) pH, temperature and odor.

g) Chlorine residual.

h) Coliform test.

The laboratory will consist of a chemist's office of about 4 x 4 m and a chemical laboratory of 8x5 m.

8.3.3 Superintendent's Office

The superintendent's office consists of a general office of 5 x 4 m for records, and a superintendent's room of about 4 x 4 m. A small reception room will also be constructed for visitors of the treatment plant.

8.3.4 Store and Workshop

Adequate space for storage and workshop is to be provided in the treatment plant for present and future requirements.

9 COST ESTIMATE

As stated in Chapter 1, the preparation of detailed specifications and cost estimates is beyond the scope of this project. However, it is of importance to have an idea of the cost of the whole scheme for allocation of funds and invitation of tenders.

The estimates of quantities are tabulated hereunder.

9.1. First Stage

9.1.1 Sanitary Sewers

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
1	Supply and construction of 42" diameter concrete pipe sewer at invert depth up to 2 meter complete job.	66	meters	9.0	594
2	(a) Supply and construction of 30" diameters concrete pipe sewer at invert depth up to 2 meter complete job.	1,315	meters	6.5	8,550
	(b) Supply and construction of 30" diameters concrete pipe sewer at invert depth upto 3.5 meter complete job.	835	meters	7.5	6,250

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
3	(a) Supply and construction of 27" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	805	meters	5.8	4,670
	(b) Supply and construction of 27" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	930	meters	6.4	5,950
4	(a) Supply and construction of 18" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	195	meters	4.5	880
	(b) Supply and construction of 18" diameters concrete pipe sewer at invert depth up to 3.5 meter complete job.	200	meters	5.2	1,040
5	(a) Supply and construction of 15" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	1,875	meters	3.8	7,120
	(b) Supply and construction of 15" diameters concrete pipe sewer at invert depth up to 3.5 meter complete job.	200	meters	4.2	840
6	(a) Supply and construction of 12" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	3,525	meters	3.2	11,300
	(b) Supply and construction of 12" diameters concrete pipe sewer at invert depth up to 3.5 meter complete job.	185	meters	3.5	650

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
7	(a) Supply and construction of 10" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job.	3,135	meters	2.7	8,450
	(b) Supply and construction of 10" diameters concrete pipe sewer at invert depth up to 3.5 meter complete job.	775	meters	3.0	2,320
8	(a) Supply and construction of 8" diameters concrete pipe sewer at invert depth up to 2.0 meter complete job. (for main and trunks)	6,060	meters	2.4	14,500
	(b) Supply and construction of 8" diameters concrete pipe sewer at invert depth up to 3.5 meter complete job. (for main and trunks)	445	meters	2.7	1,200
	(c) Supply and construction of 8" diameter pipe sewer up to an invert depth of 2.0 meters for laterols (qtry. taken as approximate) complete job.	30,000	meters	2.4	72,000
9	Construction of manholes as per drawing complete job.				
	(a) over 42" pipe sewer upto 2.0 meter depth.	1	Each	100.0	100
	(b) over 30" pipe sewer upto 2.0 meter depth.	18	Each	80.0	1,440
	(c) over 30" pipe sewer upto 3.5 meter depth.	14	Each	100.0	1,400

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
9	(d) over 27" pipe sewer upto 2.0 meter depth.	12	Each	70	840
	(e) over 27" pipe sewer upto 3.5 meter depth.	16	Each	90	1,440
	(f) over 18" pipe sewer upto 2.0 meter depth.	3	Each	55	165
	(g) over 18" pipe sewer upto 3.5 meter depth.	3	Each	70	210
	(h) over 15" pipe sewer upto 2.0 meter depth.	31	Each	45	1,400
	(i) over 15" pipe sewer upto 3.5 meter depth.	4	Each	55	220
	(k) over 12" pipe sewer upto 2.0 meter depth.	55	Each	35	1,925
	(l) over 12" pipe sewer upto 3.5 meter depth.	4	Each	45	180
	(m) over 10" pipe sewer upto 2.0 meter depth.	13	Each	30	390
	(n) over 10" pipe sewer upto 3.5 meter depth.	15	Each	40	600
	(o) over 8" pipe sewer upto 2.0 meter depth. (mains and trunks)	95	Each	25	2,375
	(p) over 8" pipe sewer upto 3.5 meter depth. (mains and trunks)	7	Each	35	245
	(q) over 8" pipe sewer upto 2.0 meter depth. (for laterols)	430	Each	25	10,750
					169,034

9.1.2 Treatment Plant

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
1	Construction of grit channel as per drawing including supplying and installing bar racks, constructing parshall flume, supply of measuring device and all accessories complete job.	2	Each	1,000	2,000
2	Construction of primary sedimentation tanks as per drawing, supply mechanical scrapers and all piping and accessories complete job.	2	Each	10,000	20,000
3	Construction of trickling filters as per drawing supplying rotary distributors, filter media, and all accessories complete job.	2	Each	15,000	30,000
4	Construction of final sedimentation tank as per drawing, supplying railing chain and all accessories complete job.	2	Each	8,000	16,000
5	(a) Construction of primary sludge digestion tanks as per drawing supplying, screw pump, mixing mechanisms and all accessories complete job.	2	Each	9,000	18,000
	(b) Construction of secondary sludge digestion tank with all the requirements.	1	Each	5,000	5,000

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
6	Construction of pumping station including cost of pumps and motors, dry well, wet well and control room, pressure sewer complete job.	1	Each	35,000	35,000
7	Construction of sludge drying beds including all piping work -- etc. complete job.	34	Each	1,500	51,000
8	Construction of primary tanks distribution box complete job.	1	Each	2,500	2,500
9	Construction of trickling filters distribution box complete job.	1	Each	2,000	2,000
10	Construction of control building for sludge digestion tanks complete job.	1	Each	5,500	5,500
11	Construction of chlorination plant complete job.	1	Each	3,000	3,000
12	Construction of laboratory with all the necessary equipment for tests complete job.	1	Each	5,000	5,000
13	Construction of store complete job.	1	Each	2,500	2,500
14	Construction of workshop.	1	Each	4,000	4,000
15	Construction of wall and gates complete job.	1	Job	8,000	8,000
16	Construction of roads inside the treatment plant.	1	Job	5,000	5,000

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
17	Cost of land.	50,000	sq. meter	0.5	25,000
18	Other expenses	-	-	-	30,000
					269,500

9.2. Second Stage

9.2.1 Sanitary Sewers

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
	Second stage (qty. of pipes are taken as approximate because there is no master plan for the city)				
1	(a) Supply and construction 15" diameters concrete pipe sewer at invert depth upto 2.0 meters complete job.	500	meter	3.8	1,900
	(b) Supply and construction 15" diameters concrete pipe sewer at invert depth upto 3.5 meters complete job.	200	meter	4.2	840
2	(a) Supply and construction 12" diameters concrete pipe sewer at invert depth upto 2.0 meters complete job.	600	meter	3.2	1,920

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
2	(b) Supply and construction 12" diameters concrete pipe sewer at invert depth upto 3.5 meters complete job.	100	meter	3.5	350
3	(a) Supply and construction 10" diameters concrete pipe sewer at invert depth upto 2.0 meters complete job.	3,800	meter	2.7	10,250
	(b) Supply and construction 10" diameters concrete pipe sewer at invert depth upto 3.5 meters complete job.	900	meter	3.0	2,700
4	(a) Supply and construction 8" diameters concrete pipe sewer at invert depth upto 2.0 meters complete job. (mains and trunks)	3,500	meter	2.4	8,400
	(b) Supply and construction 8" diameters concrete pipe sewer at invert depth upto 3.5 meters complete job (mains and trunks)	200	meter	2.7	540
	(c) Supply and construction 8" duameters concrete pipe sewer at invert depth upto 2.0 meters complete job.	10,000	meter	2.4	24,000
5	Constructing manholes as per drawing, complete job.				
	(a) over 15" pipe sewer upto 2.0 meters depth.	7	Each	45	315
	(b) over 15" pipe sewer upto 3.5 meters depth.	3	Each	55	165

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
5	(c) over 12" pipe sewer upto 2.0 meters depth.	8	Each	35	280
	(d) over 12" pipe sewer upto 3.5 meters depth.	2	Each	45	90
	(e) over 10" pipe sewer upto 2.0 meters depth.	47	Each	30	1,410
	(f) over 10" pipe sewer upto 3.5 meters depth.	16	Each	40	640
	(g) over 8" pipe sewer upto 2.0 meters depth. (mains and trunks)	48	Each	25	1,200
	(h) over 8" pipe sewer upto 2.0 meters depth. (laterals)	130	Each	25	3,250
					58,240

9.2.2 Treatment Plant

Item No.	Description	Estimated Quantities	Unit	Rate	Cost in I.D.
1	Construction of primary sedimentation tanks as per drawing, supplying mechanical scrapers and all piping and accessories complete job.	2	Each	10,000	20,000

Item No.	Description	Estimate Quantities	Unit	Rate	Cost in I.D.
2	Construction of filters as per drawing supplying rotary distributors, filter media, and all accessories complete job.	2	Each	15,000	30,000
3	Construction of final sedimentation tank as per drawing with trailing chain, complete job.	2	Each	8,000	16,000
4	(a) Construction of primary sludge digestion tank with all the requirements complete job.	2	Each	9,000	18,000
	(b) Construction of secondary sludge digestion tank complete job.	1	Each	5,000	5,000
5	Construction of sludge drying beds including all piping work etc. complete job.	34	Each	1,500	51,000
6	General expenses	-	-	-	10,000
					150,000

9.3. Abstract of Costs

Sanitary Sewer, 1st Stage	I.D.	169,034
Sanitary Sewer, 2nd Stage	I.D.	58,200
Treatment Plant, 1st Stage	I.D.	269,500
Treatment Plant, 2nd Stage	I.D.	150,000
Total	I.D.	646,734

This amount is equivalent to \$ 1,790,000 and is an investment intended to serve a population of 112,500.

$$\text{Cost per capita} \;\ldots\ldots\ldots\; \frac{1,790,000}{112,500}$$

$$= \$\,15.9 \text{ - say, } \$\,16.0$$

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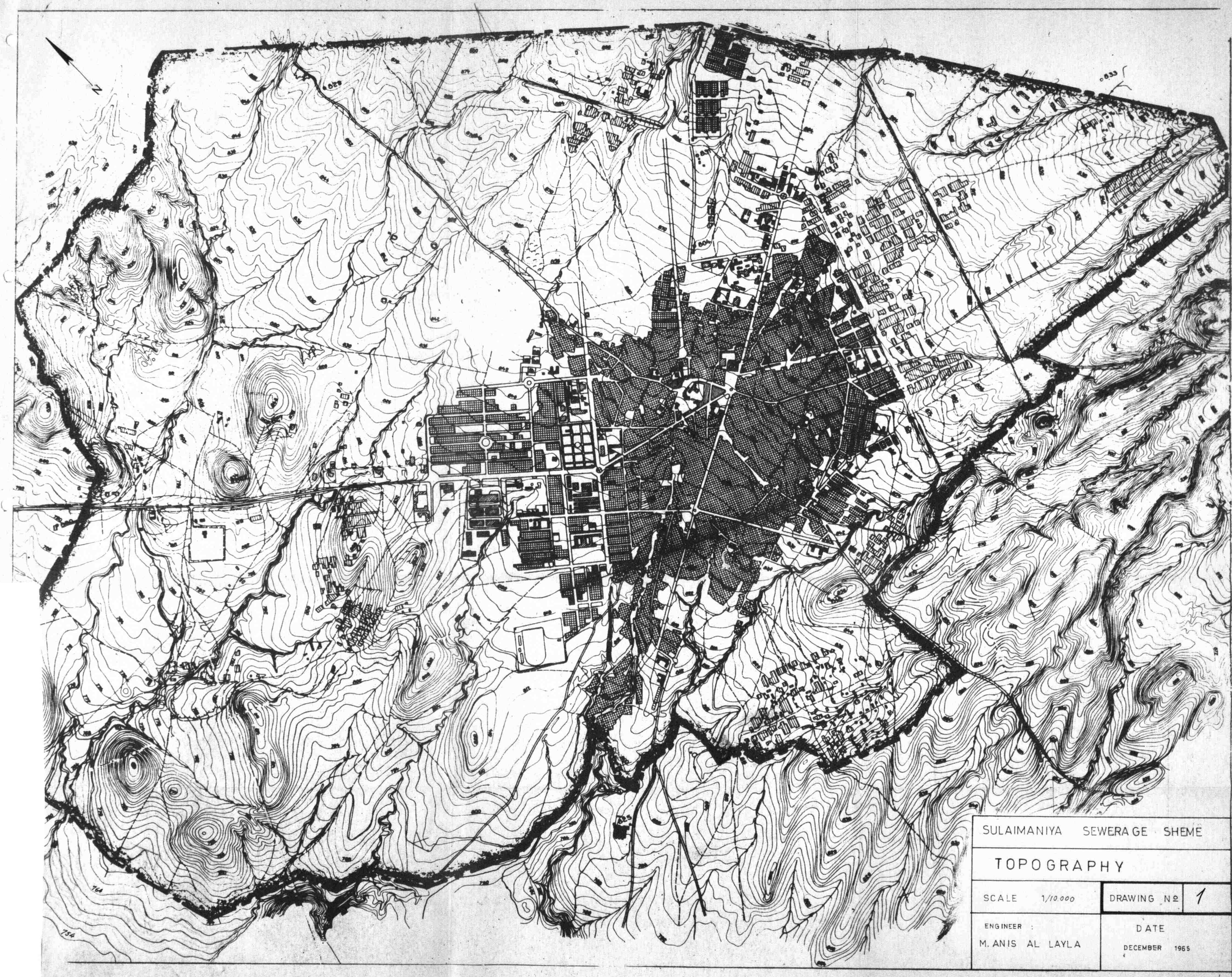
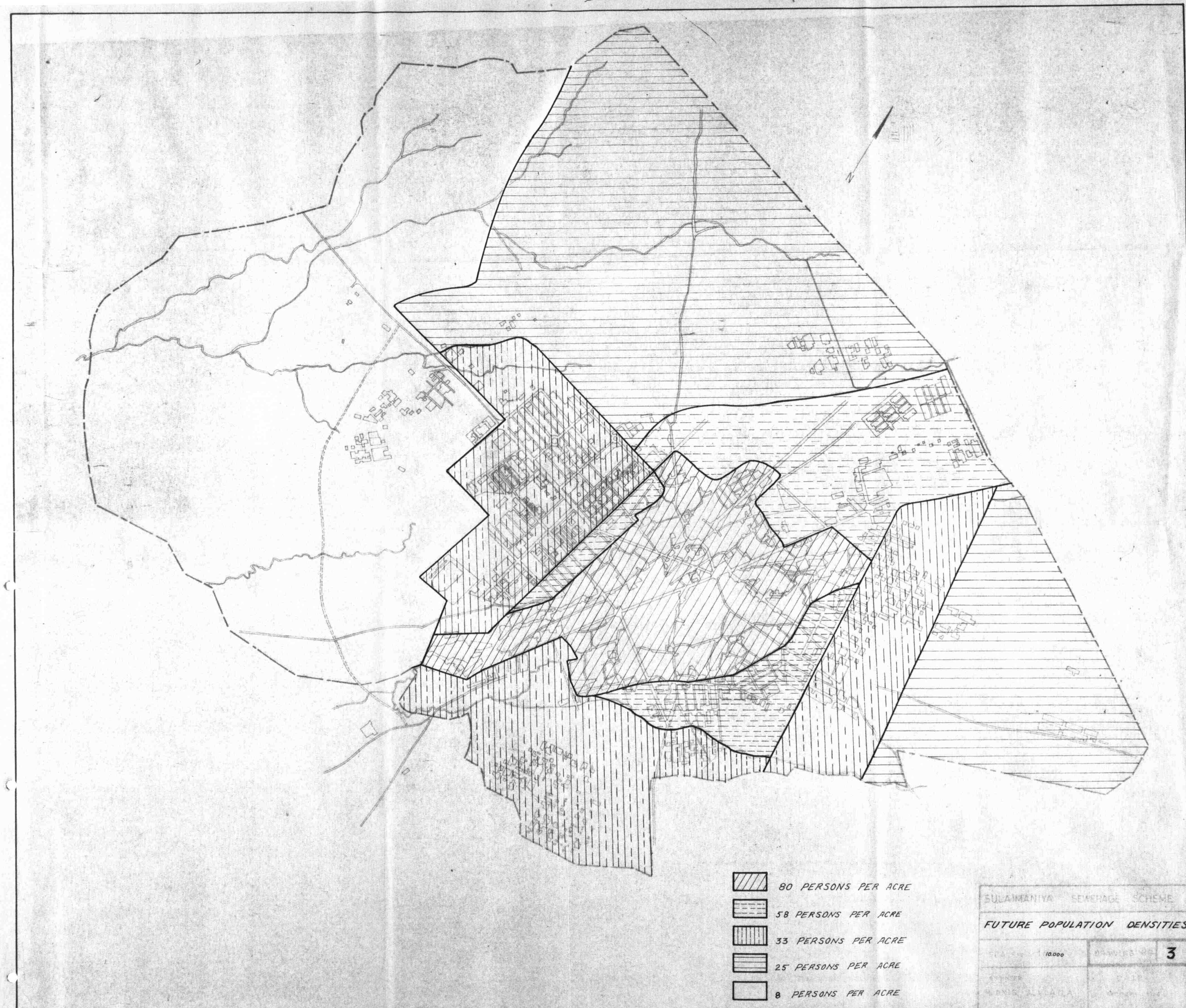

SULAIMANIYA SEWERAGE SHEME
TOPOGRAPHY
SCALE 1/10.000
DRAWING № 1
ENGINEER :
M. ANIS AL LAYLA
DATE
DECEMBER 1965

- 81 PERSONS PER ACRE
- 70 PERSONS PER ACRE
- 50 PERSONS PER ACRE
- 26 PERSONS PER ACRE
- 0 PERSONS PER ACRE

SULAIMANIYA SEWERAGE SCHEME	
PRESENT POPULATION DENSITIES	
SCALE 1/10.000	DRAWING No 2
ENGINEER : M. ANIS AL-LAYLA	DATE DECEMBER 196[illegible]


80 PERSONS PER ACRE
58 PERSONS PER ACRE
33 PERSONS PER ACRE
25 PERSONS PER ACRE
8 PERSONS PER ACRE
SULAIMANIYA SEWERAGE SCHEME
FUTURE POPULATION DENSITIES
10.000
3

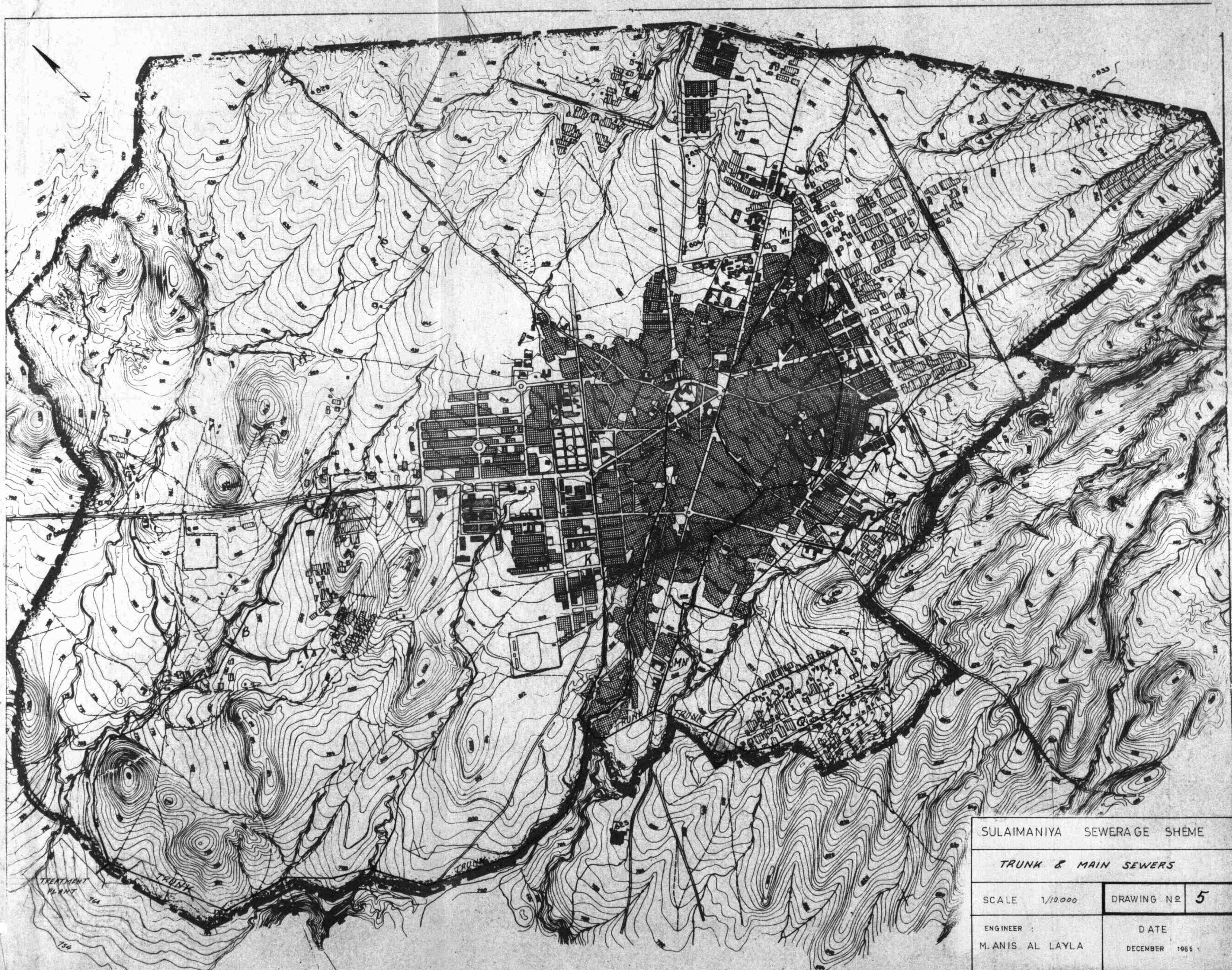
SULAIMANIYA SEWERAGE SCHEME

CATCHMENT AREAS & SEWER MAINS

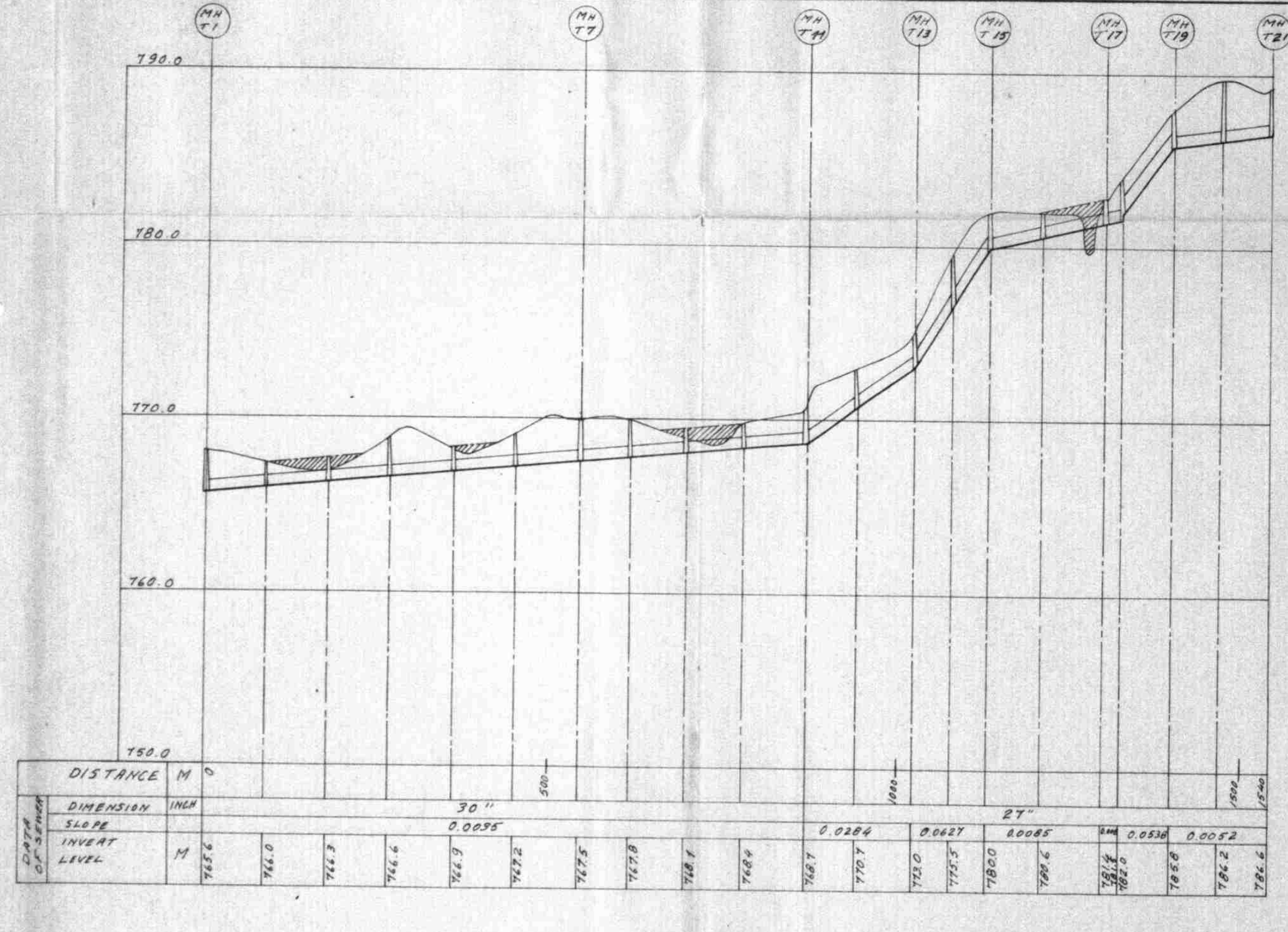
SCALE 1/10000 | DRAWING No. 4

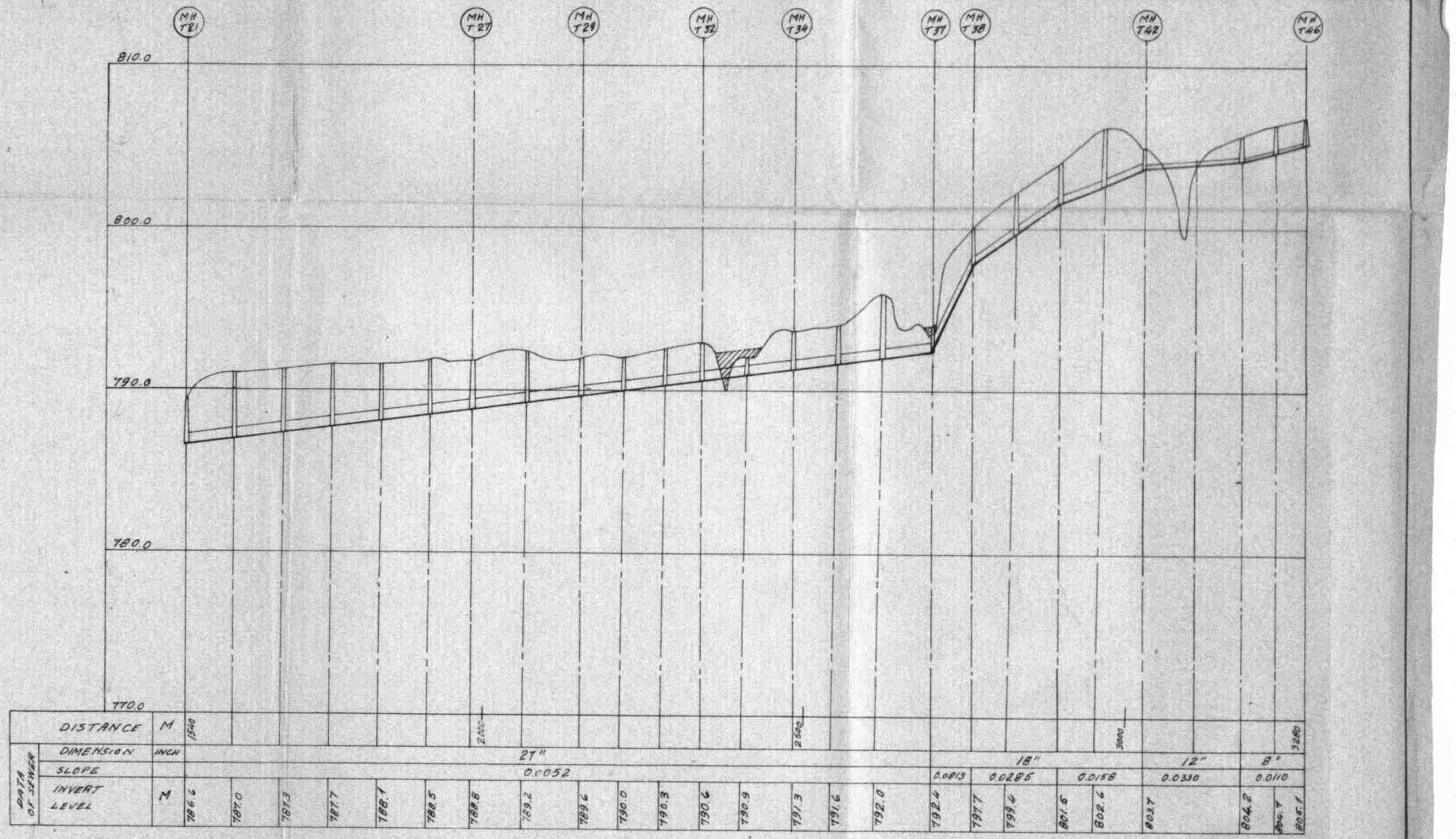
ENGINEER : M. ANIS AL-LAYLA

DATE DECEMBER 1965



SULAIMANIYA SEWERAGE SHEME
TRUNK & MAIN SEWERS
SCALE 1/10.000
DRAWING Nº 5
ENGINEER :
M. ANIS AL LAYLA
DATE
DECEMBER 1965
TREATMENT PLANT
TRUNK





SULAIMANIYA SEWERAGE SCHEME

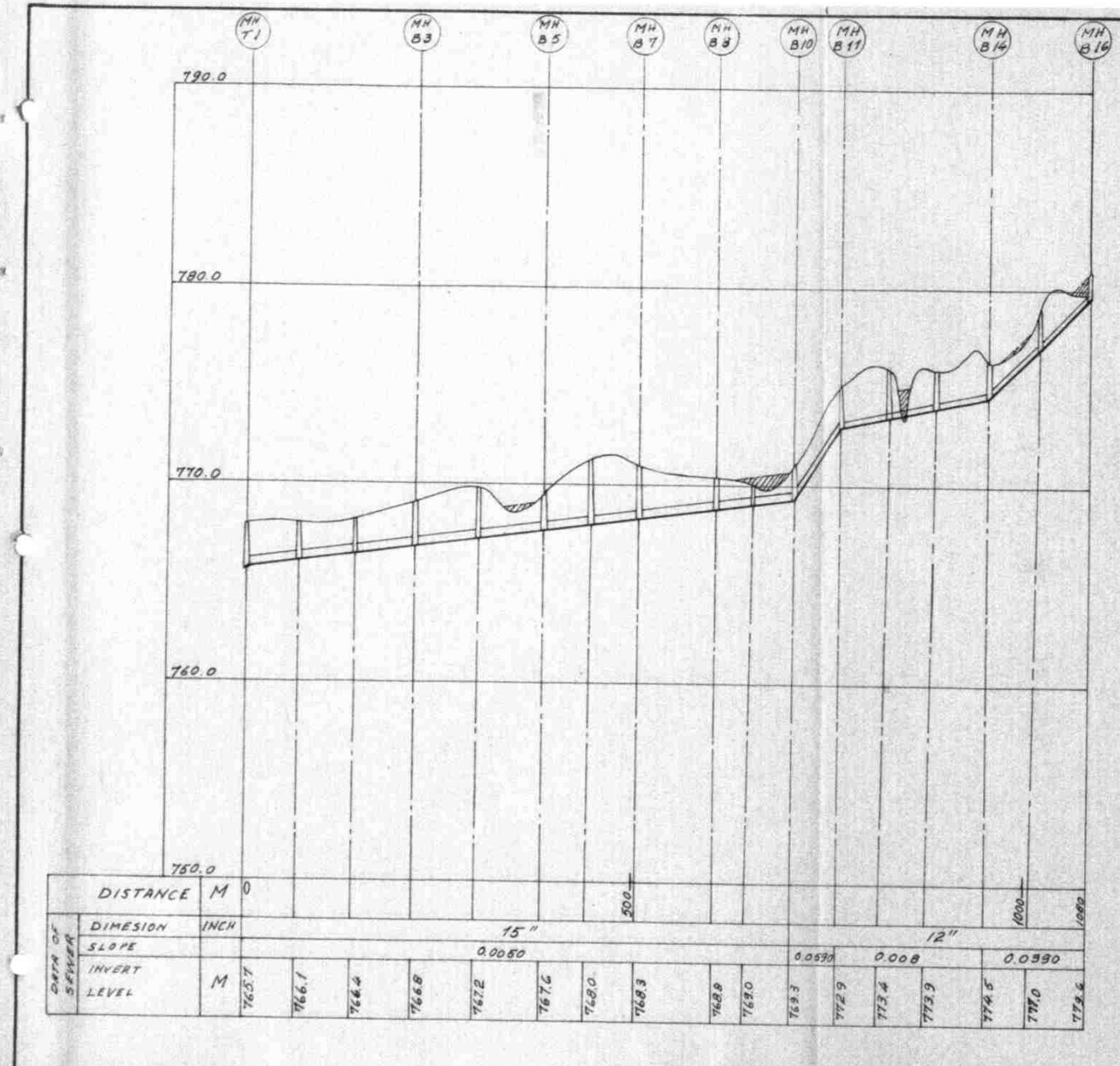
LONGITUDINAL SECTION FOR THE TRUNK

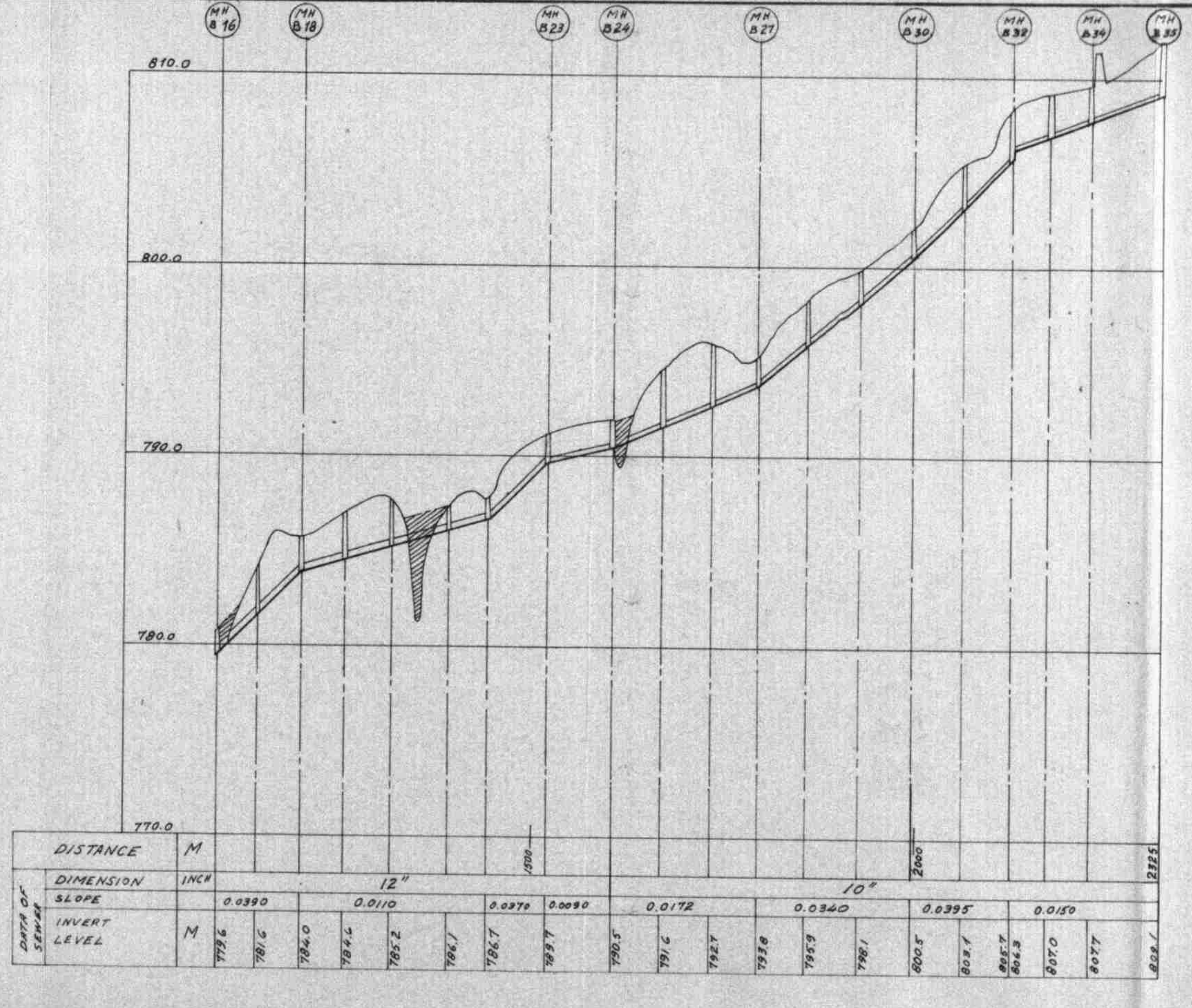
SCALE : HOR: 1/5000 VER: 1/200

DRAWING No. 6

ENGINEER: M. ANIS AL-LAYLA

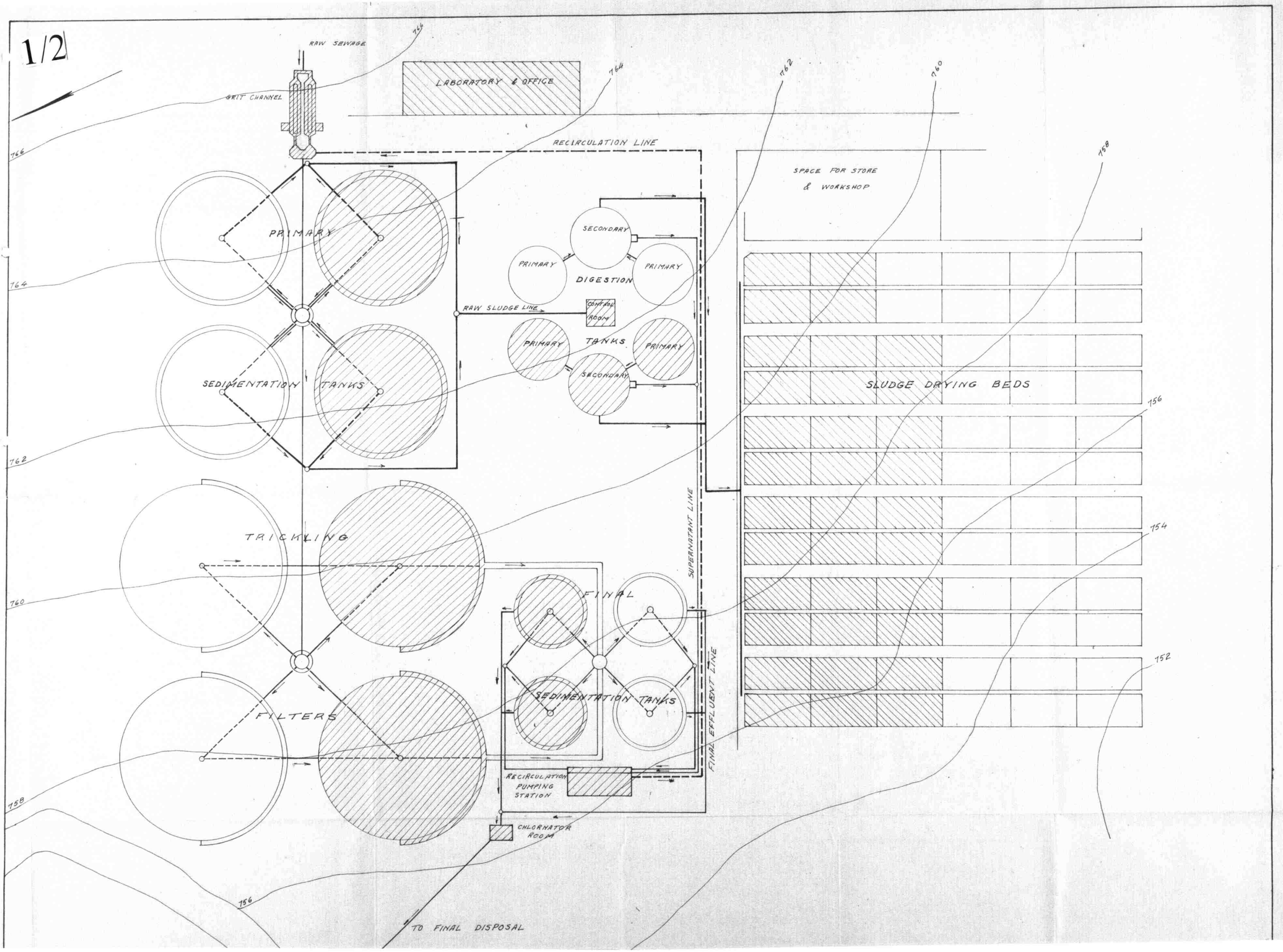
DATE DECEMBER 1965

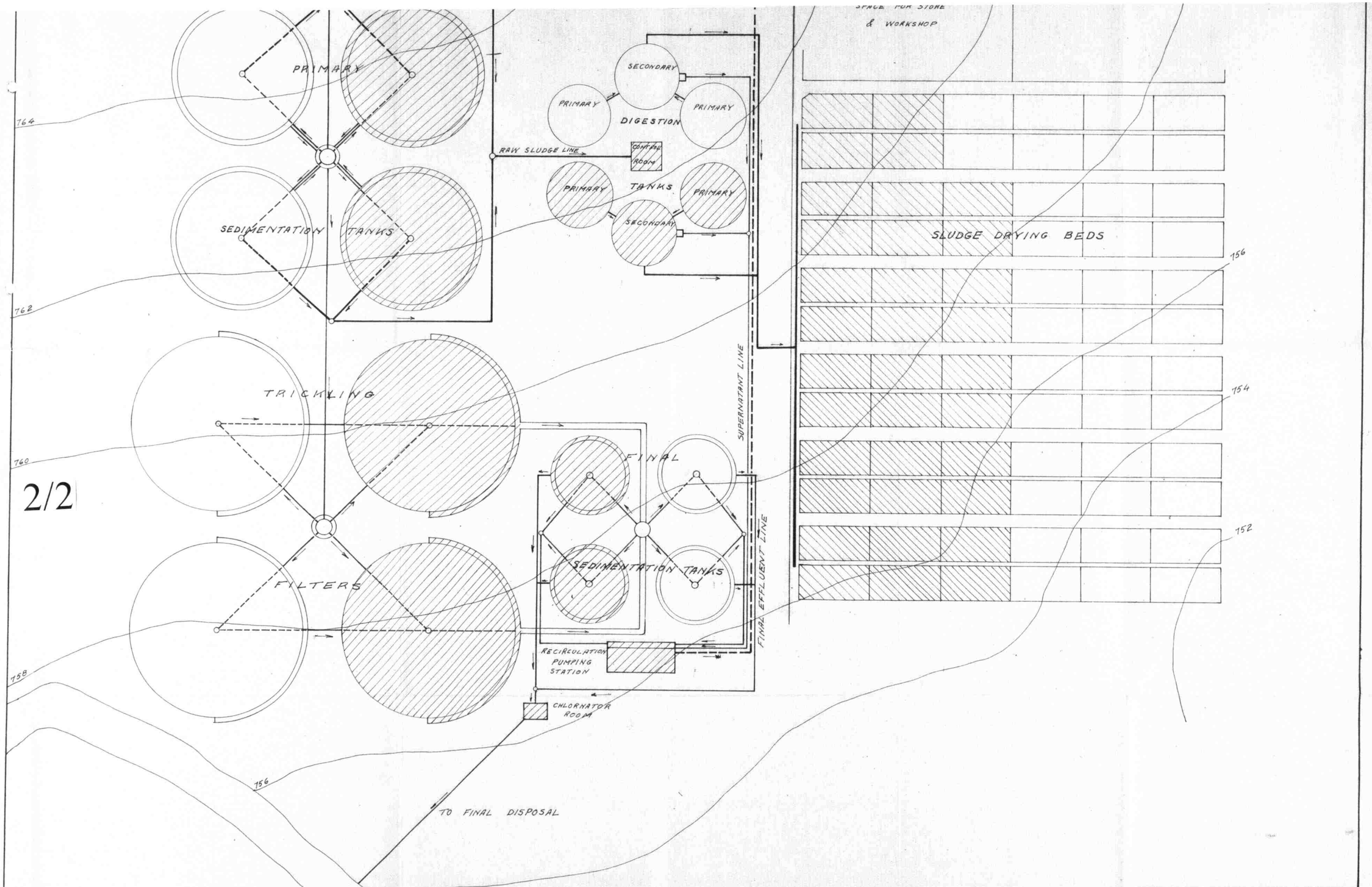




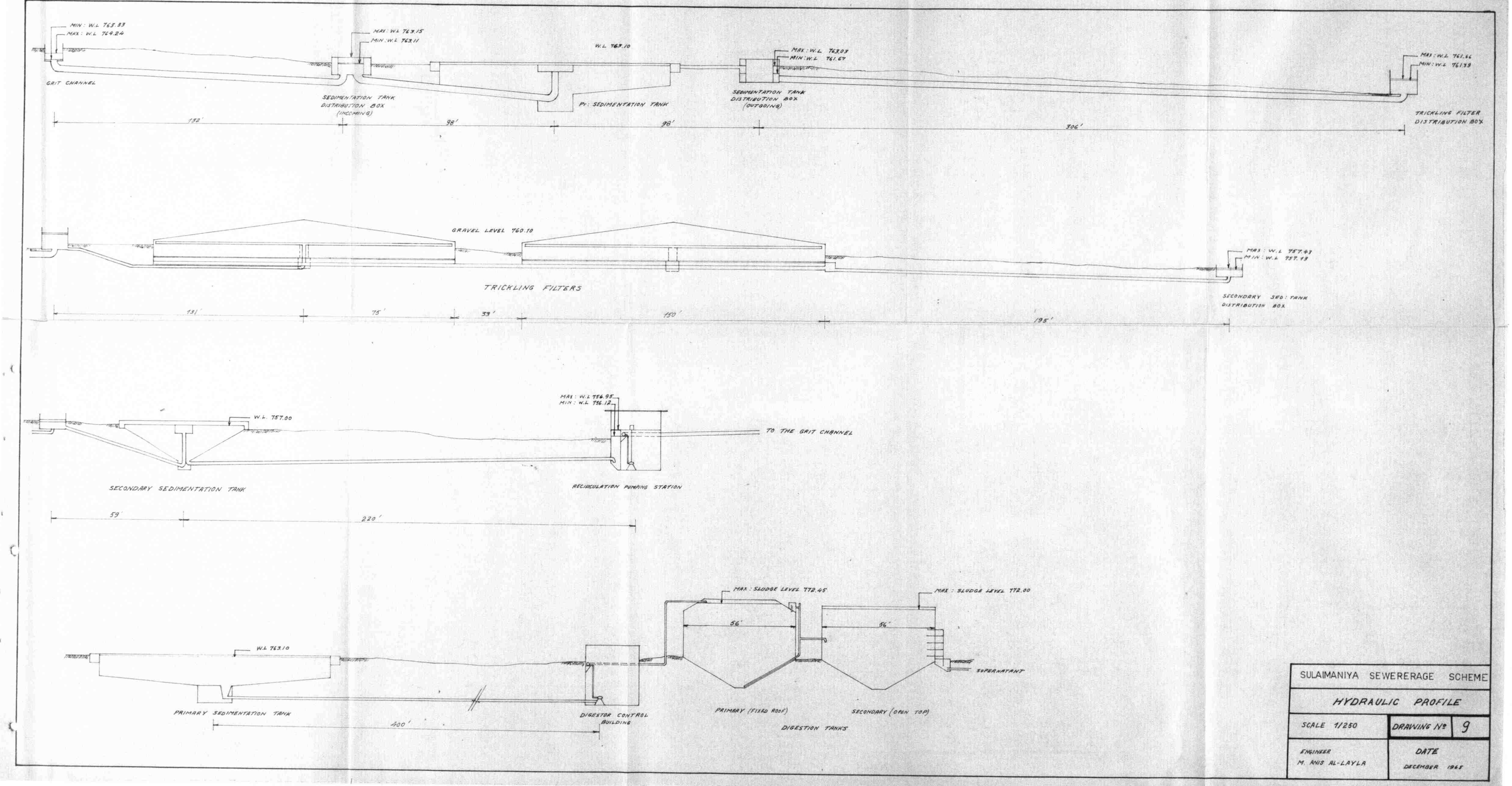
SULAIMANIYA SEWERAGE SCHEME	
LONGITUDINAL SECTION FOR LINE B	
SCALE : HOR. 1/5000 VER. 1/200	DRAWING Nº 7
ENGINEER: M ANIS AL-LAYLA	DATE DECEMBER 1965

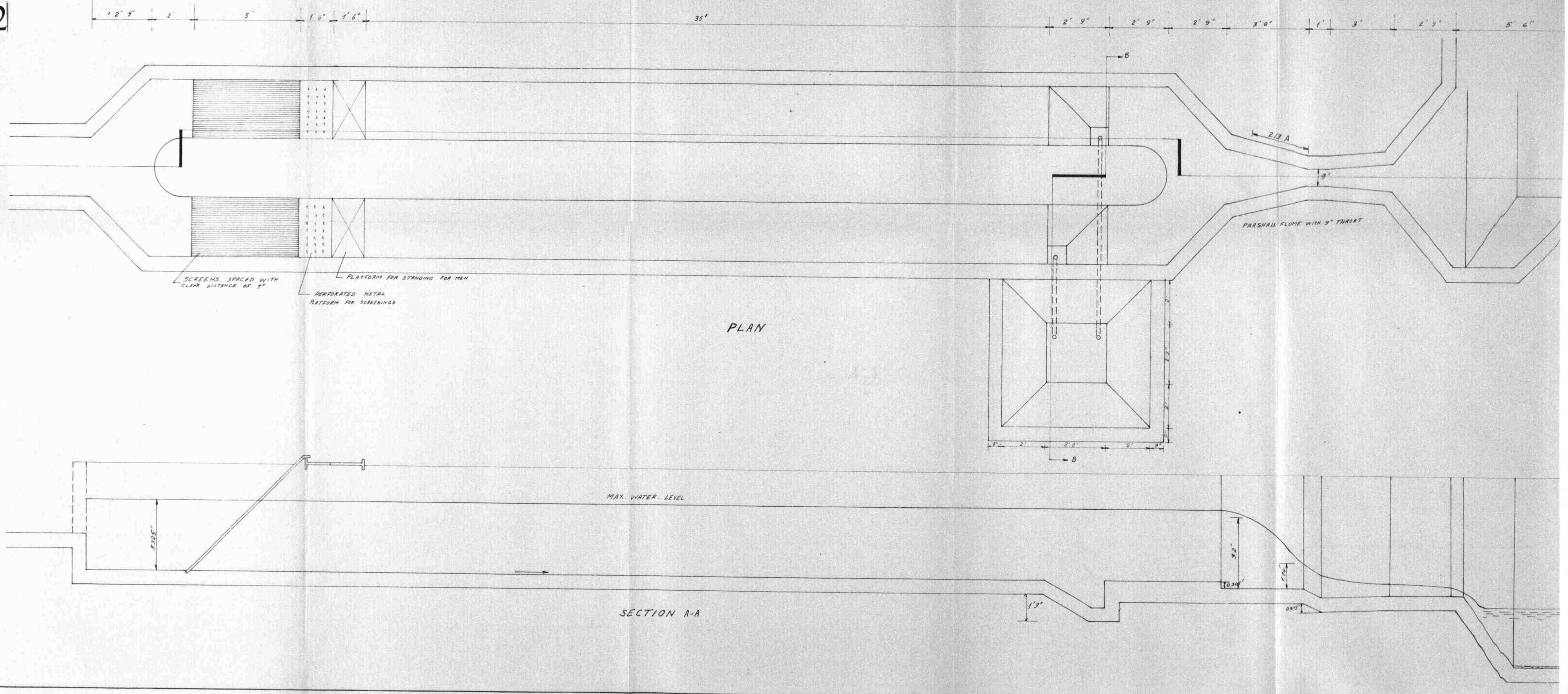
1/2


RAW SEWAGE
GRIT CHANNEL
LABORATORY & OFFICE
RECIRCULATION LINE
SPACE FOR STORE
& WORKSHOP
PRIMARY
SEDIMENTATION TANKS
SECONDARY
PRIMARY
DIGESTION
PRIMARY
RAW SLUDGE LINE
CONTROL ROOM
PRIMARY
TANKS
PRIMARY
SECONDARY
SLUDGE DRYING BEDS
SUPERNATANT LINE
TRICKLING
FILTERS
FINAL
SEDIMENTATION TANKS
FINAL EFFLUENT LINE
RECIRCULATION
PUMPING
STATION
CHLORNATOR
ROOM
TO FINAL DISPOSAL
766
764
762
760
758
756
754
752



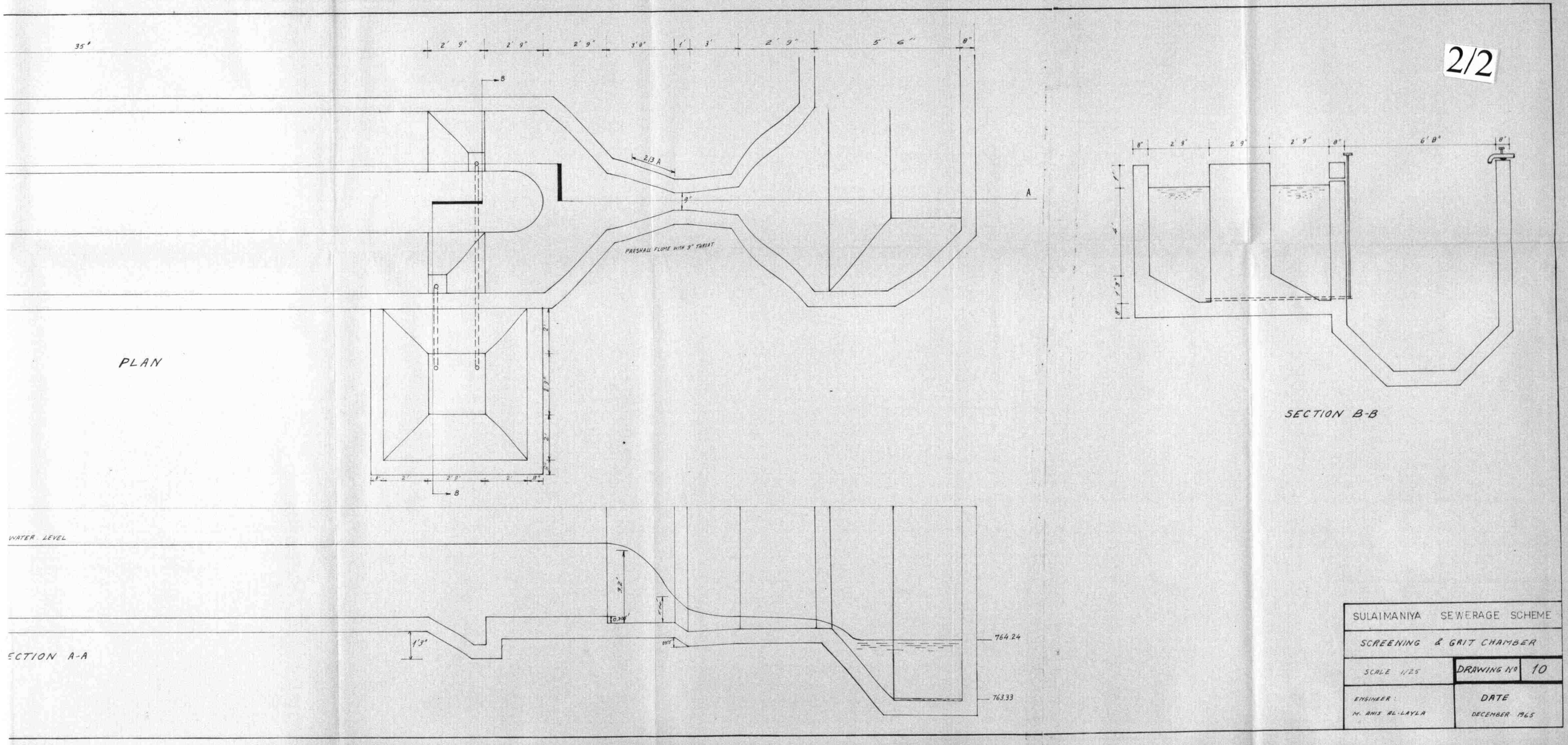
SULAIMANIYA SEWERAGE SCHEME	
TREATMENT PLANT LAYOUT	
SCALE 1/500	DRAWING Nº 8
ENGINEER M. ANIS AL-LAYLA	DATE DECEMBER 1965

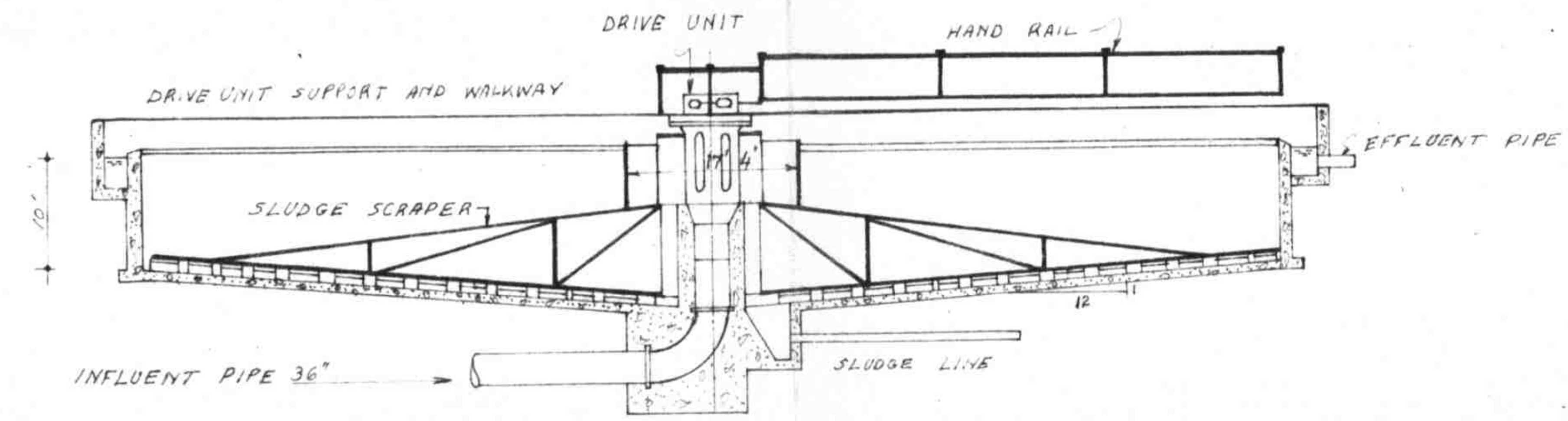

MIN : W.L 763.33
MAX : W.L 764.24
GRIT CHANNEL
MAX : W.L 763.15
MIN : W.L 763.11
SEDIMENTATION TANK
DISTRIBUTION BOX
(INCOMING)
W.L 763.10
Pr: SEDIMENTATION TANK
MAX : W.L 763.03
MIN : W.L 761.67
SEDIMENTATION TANK
DISTRIBUTION BOX
(OUTGOING)
MAX : W.L 761.66
MIN : W.L 761.39
TRICKLING FILTER
DISTRIBUTION BOX
132'
98'
98'
306'
GRAVEL LEVEL 760.10
MAX : W.L 757.43
MIN : W.L 757.13
TRICKLING FILTERS
SECONDARY SED: TANK
DISTRIBUTION BOX
131'
75'
33'
150'
195'
W.L. 757.00
MAX : W.L 756.95
MIN : W.L 756.12
TO THE GRIT CHANNEL
SECONDARY SEDIMENTATION TANK
RECIRCULATION PUMPING STATION
59'
220'
MAX : SLUDGE LEVEL 772.45
MAX : SLUDGE LEVEL 772.00
56'
56'
W.L 763.10
SUPERNATANT
PRIMARY SEDIMENTATION TANK
DIGESTOR CONTROL
BUILDING
PRIMARY (FIXED ROOF)
SECONDARY (OPEN TOP)
DIGESTION TANKS
400'
SULAIMANIYA SEWERERAGE SCHEME
HYDRAULIC PROFILE
SCALE 1/250
DRAWING № 9
ENGINEER
M. ANIS AL-LAYLA
DATE
DECEMBER 1965



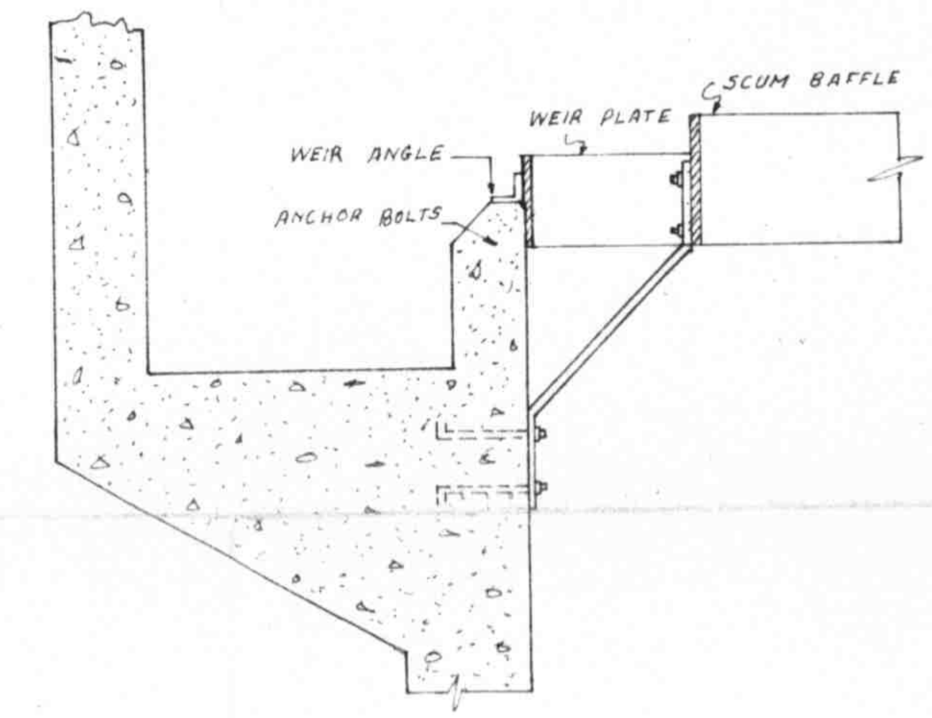
SCREENS SPACED WITH CLEAR DISTANCE OF 1"
PERFORATED METAL PLATFORM FOR SCREENINGS
PLATFORM FOR STANDING FOR MEN
PARSHALL FLUME WITH 9" THROAT
2/3 A
PLAN
MAX. WATER LEVEL
SECTION A-A

2/2

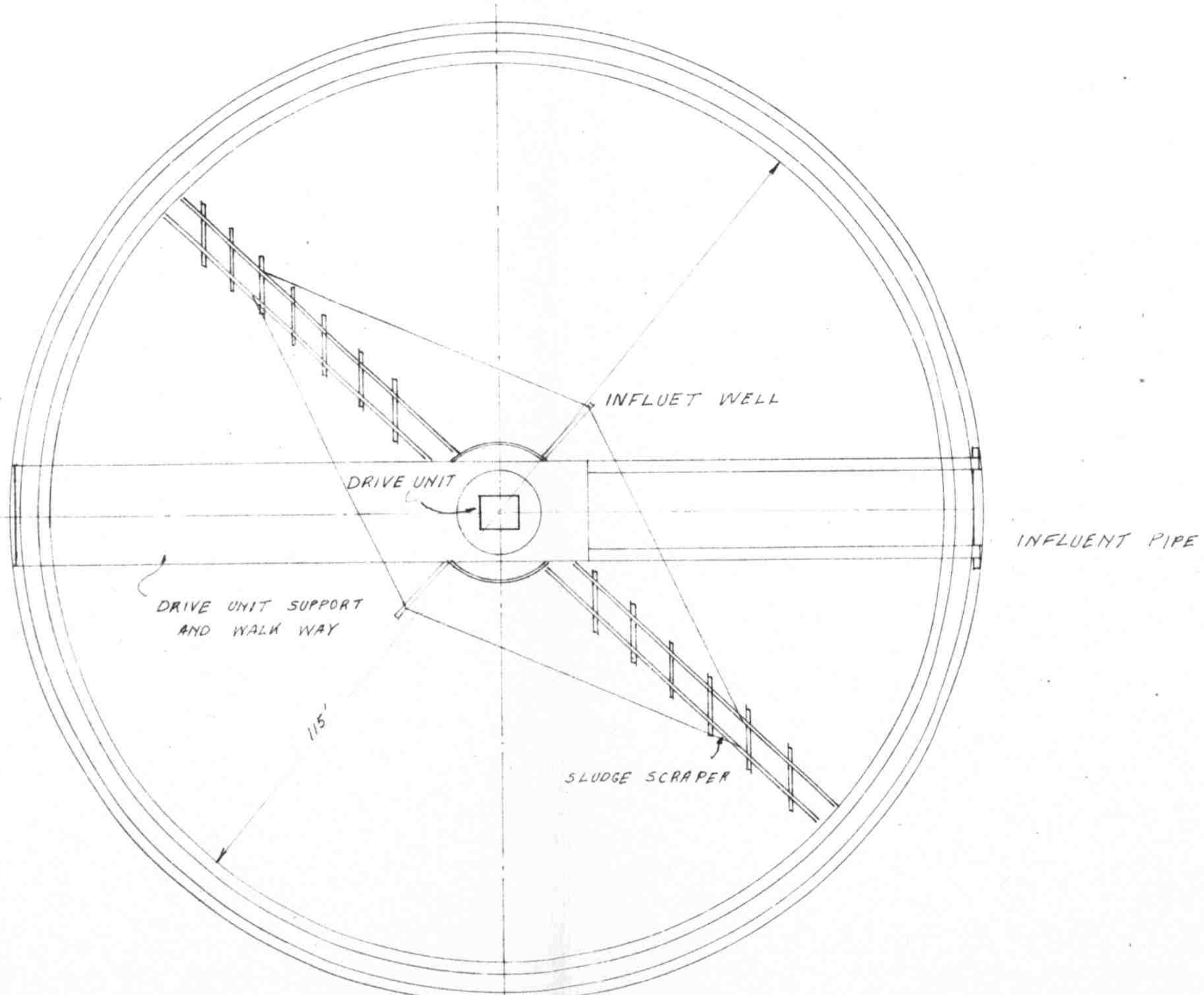

PLAN
PARSHALL FLUME WITH 9" THROAT
SECTION B-B
WATER LEVEL
SECTION A-A
764.24
763.33
SULAIMANIYA SEWERAGE SCHEME
SCREENING & GRIT CHAMBER
SCALE 1/25
DRAWING NO 10
ENGINEER :
M. ANIS AL-LAYLA
DATE
DECEMBER 1965



SECTIONAL ELEVATION

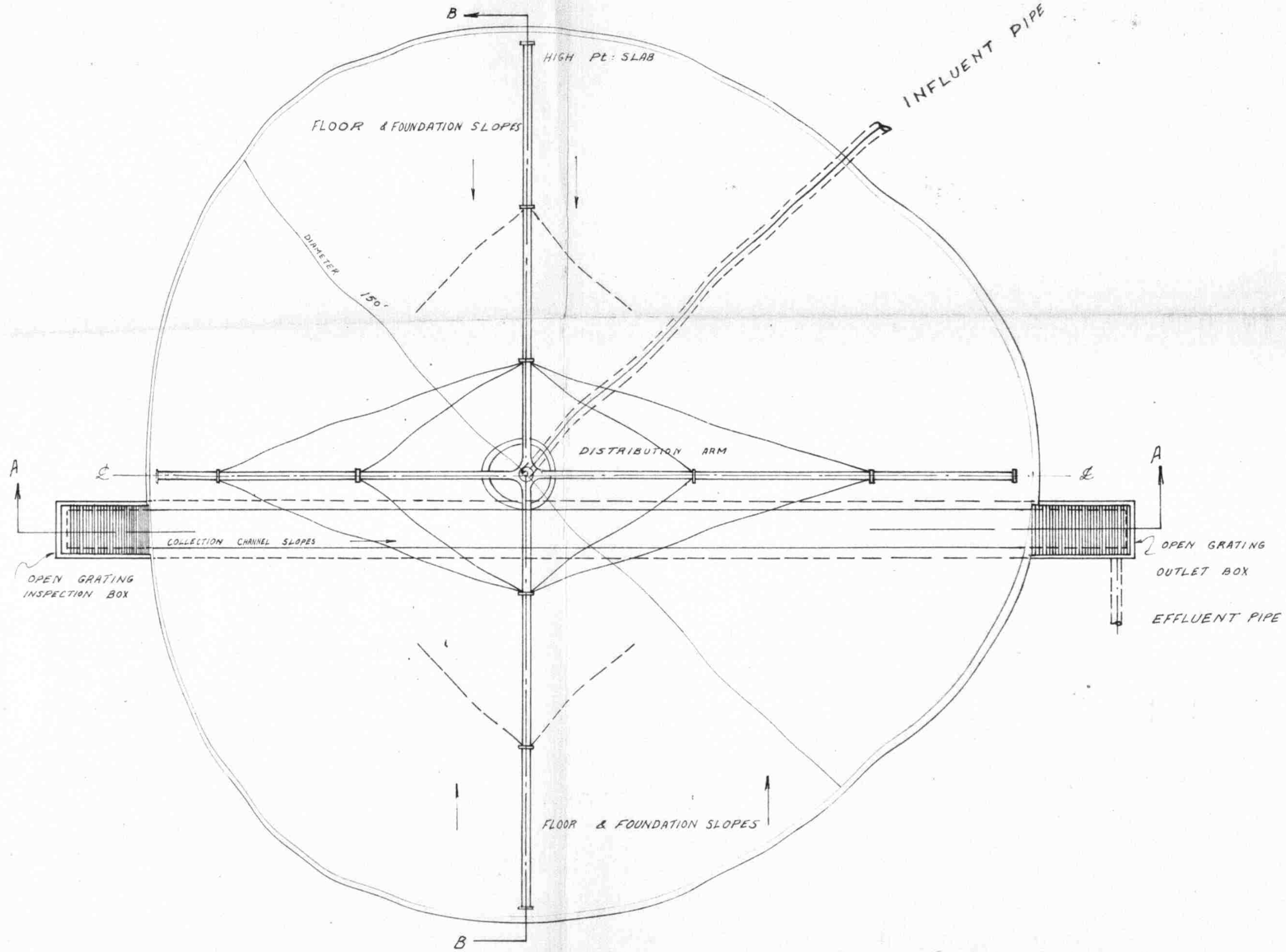


WEIR AND SCUM BAFFLE DETAIL

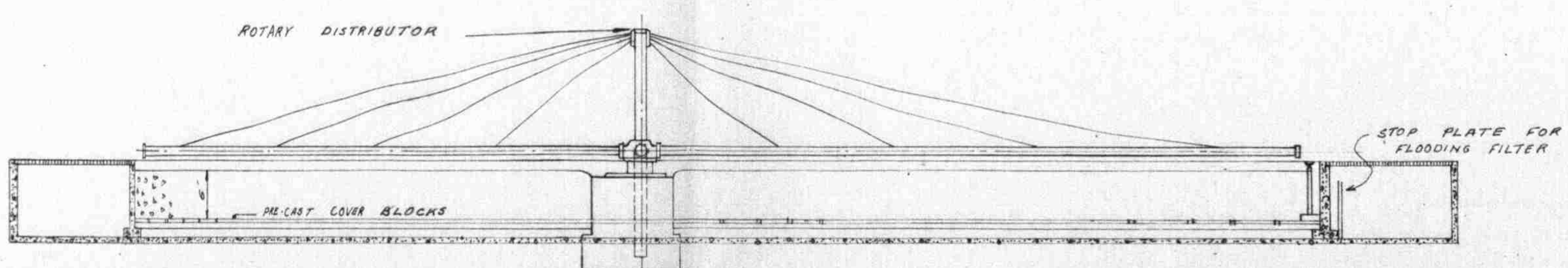


PLAN

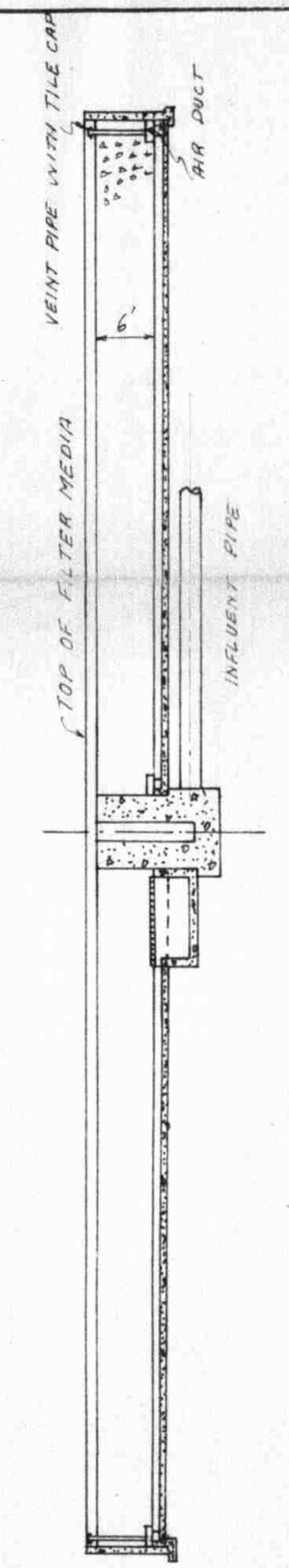
SULAIMANIYA SEWERAGE SCHEME	
PRIMARY SEDIMENTATION TANK	
SCALE 1/200	DRAWING No 11
ENGINEER M.ANIS AL LAYLA	DATE DECEMBER 1965



PLAN

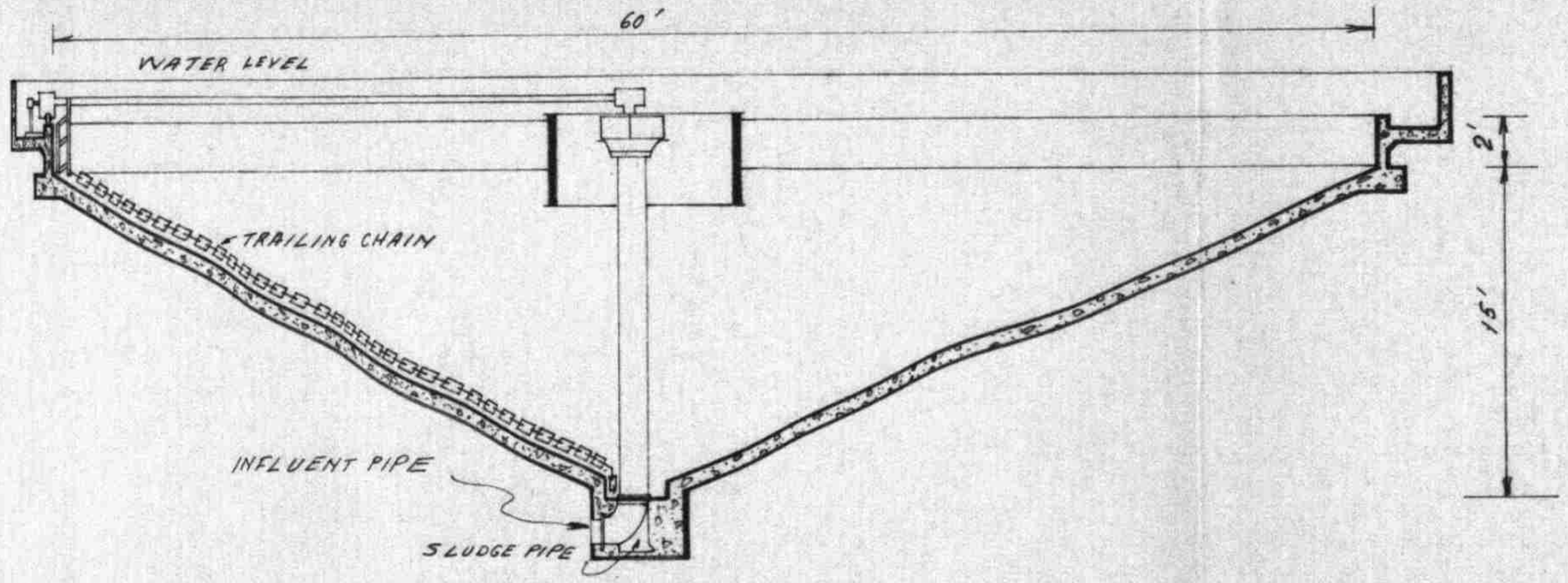


SECTION A-A

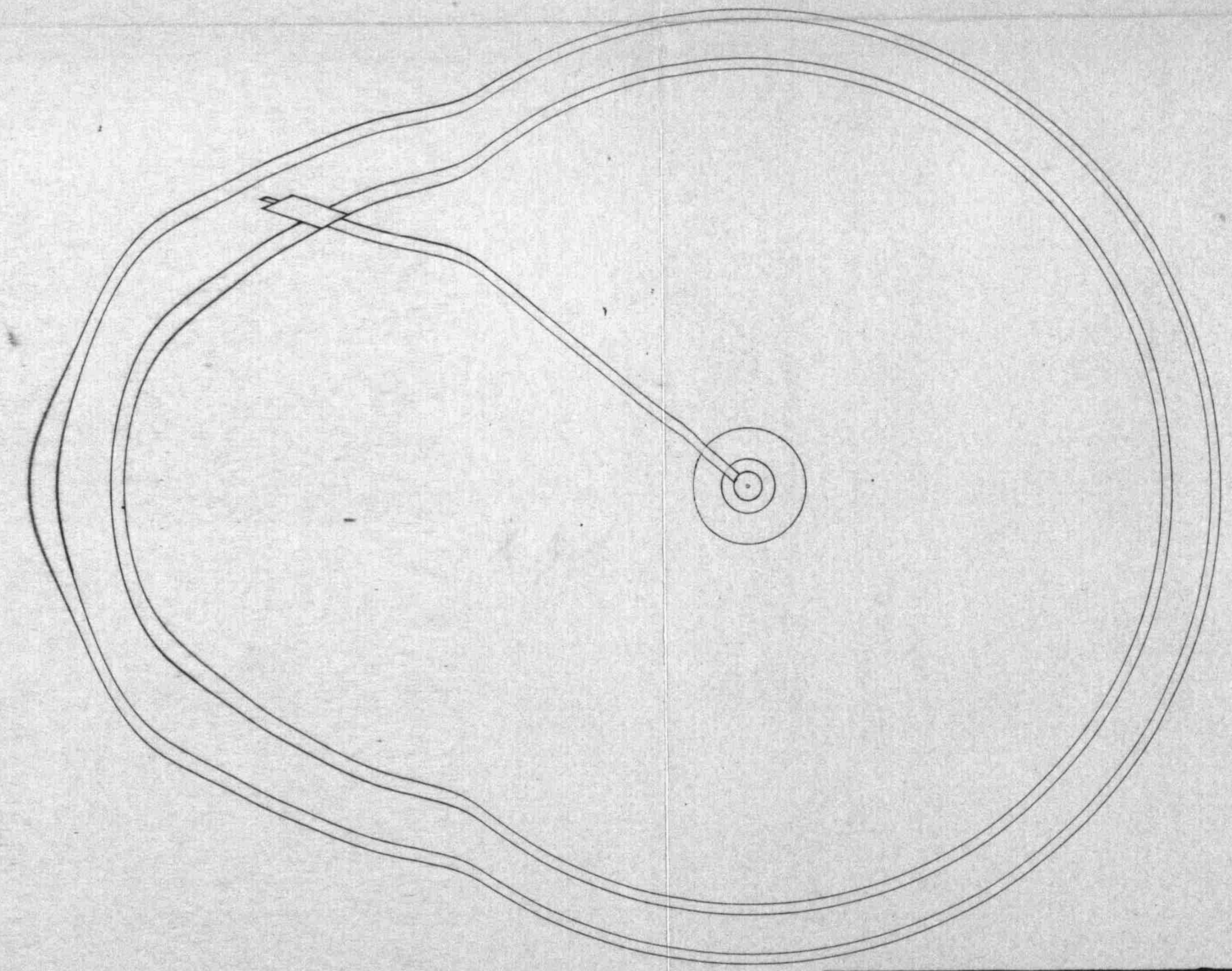


SECTION B-B

SULAIMANIYA SEWERAGE SCHEME	
HIGH RATE TRICKLING FILTER	
SCALE 1/200	DRAWING Nº 12
ENGINEER M. ANIS AL-LAYLA	DATE DECEMBER 1965

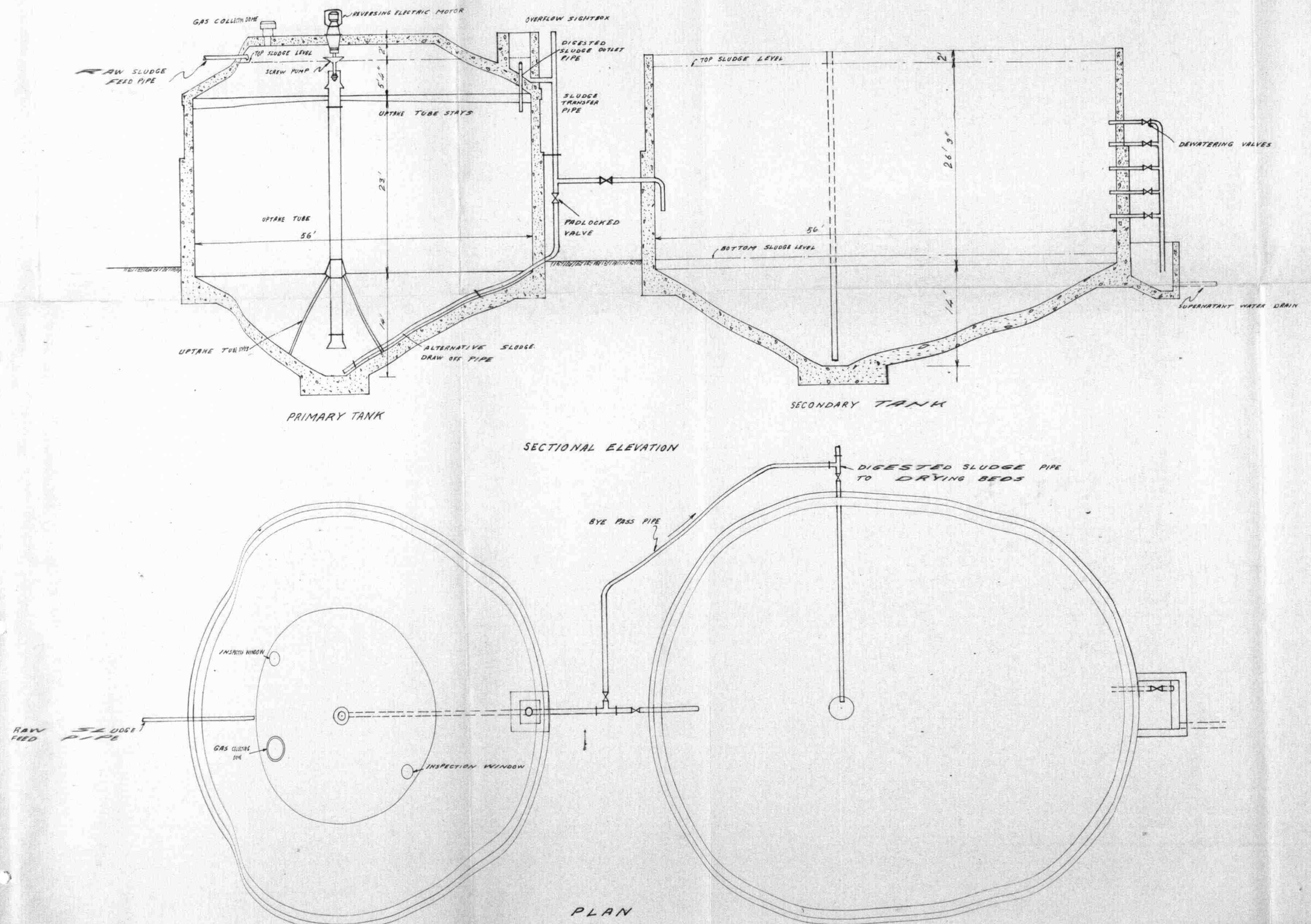


SECTIONAL ELEVATION

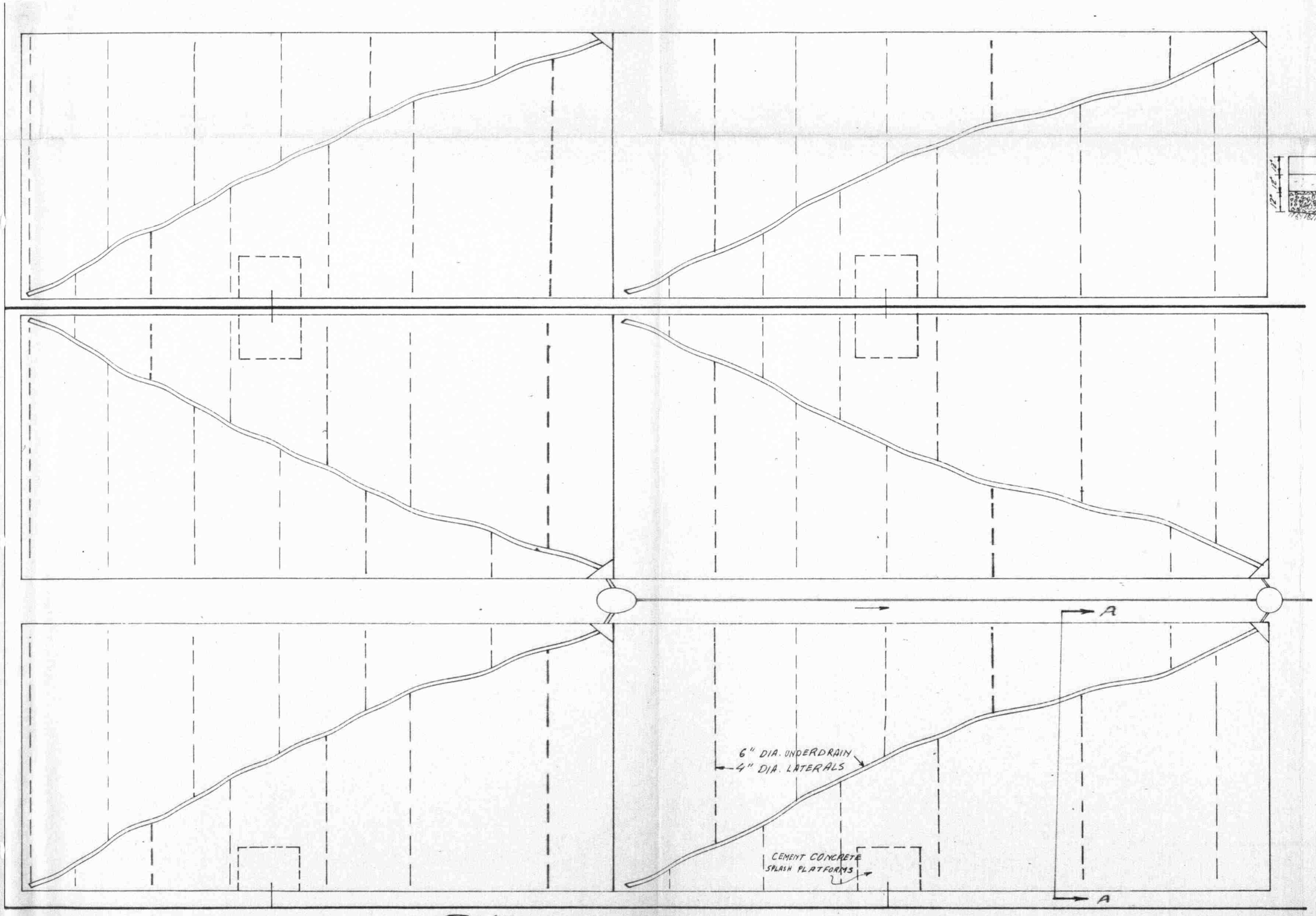


PLAN

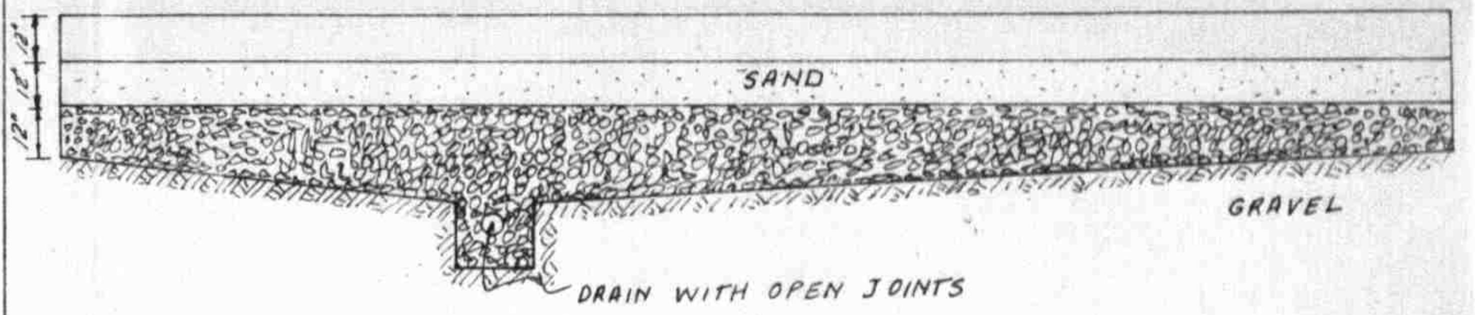
SULAIMANIYA SEWERAGE SCHEME	
FINAL SEDIMENTATION TANK	
SCALE 1/100	DRAWING Nº 13
ENGINEER M. ANIS AL-LAYLA	DATE DECEMBER 1965



SULAIMANIYA SEWERAGE SCHEME	
SLUDGE DIGESTION TANK	
SCALE 1/100	DRAWING Nº 14
ENGINEER: M. ANIS AL-LAYLA	DATE: DECEMBER 1965

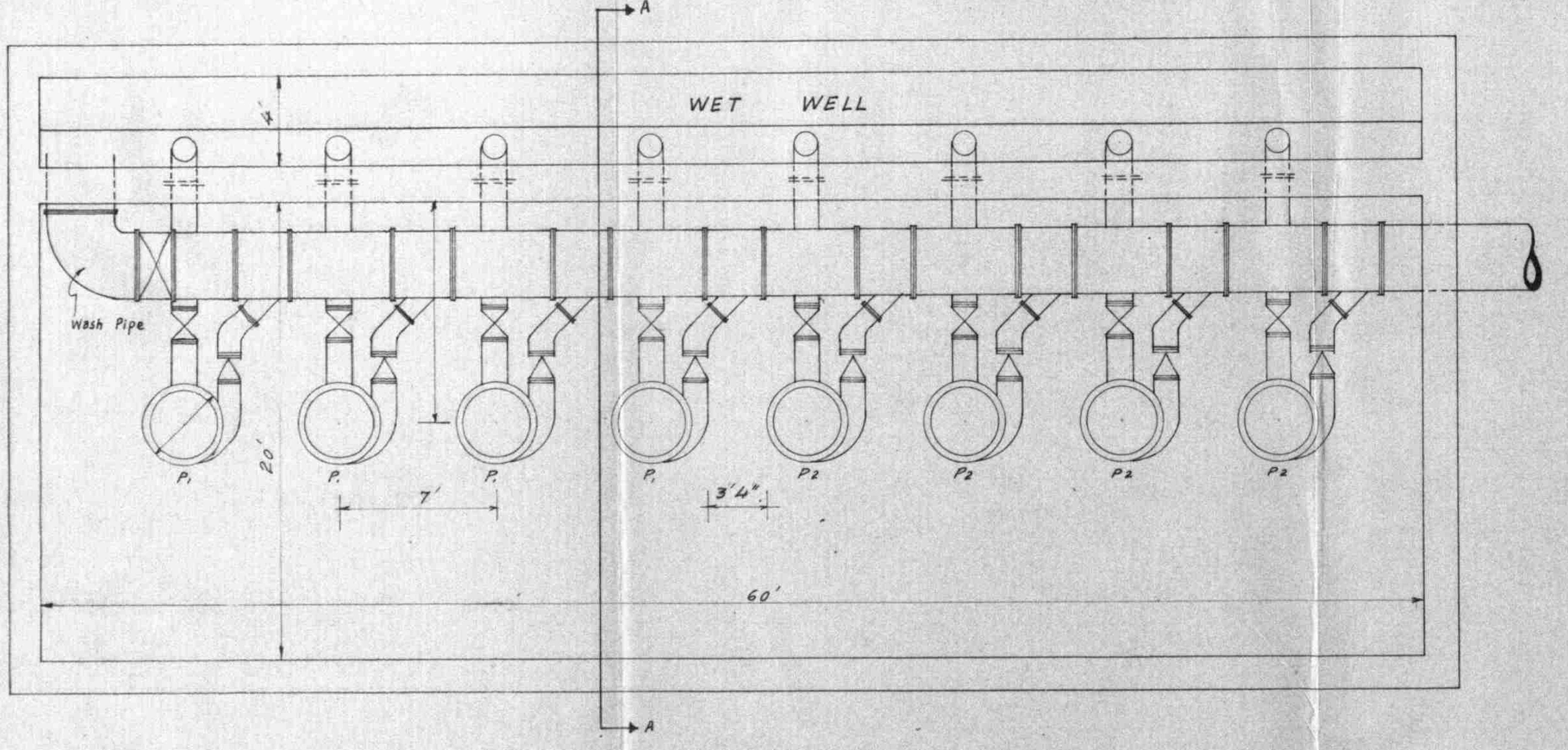


PLAN SCALE 1/100

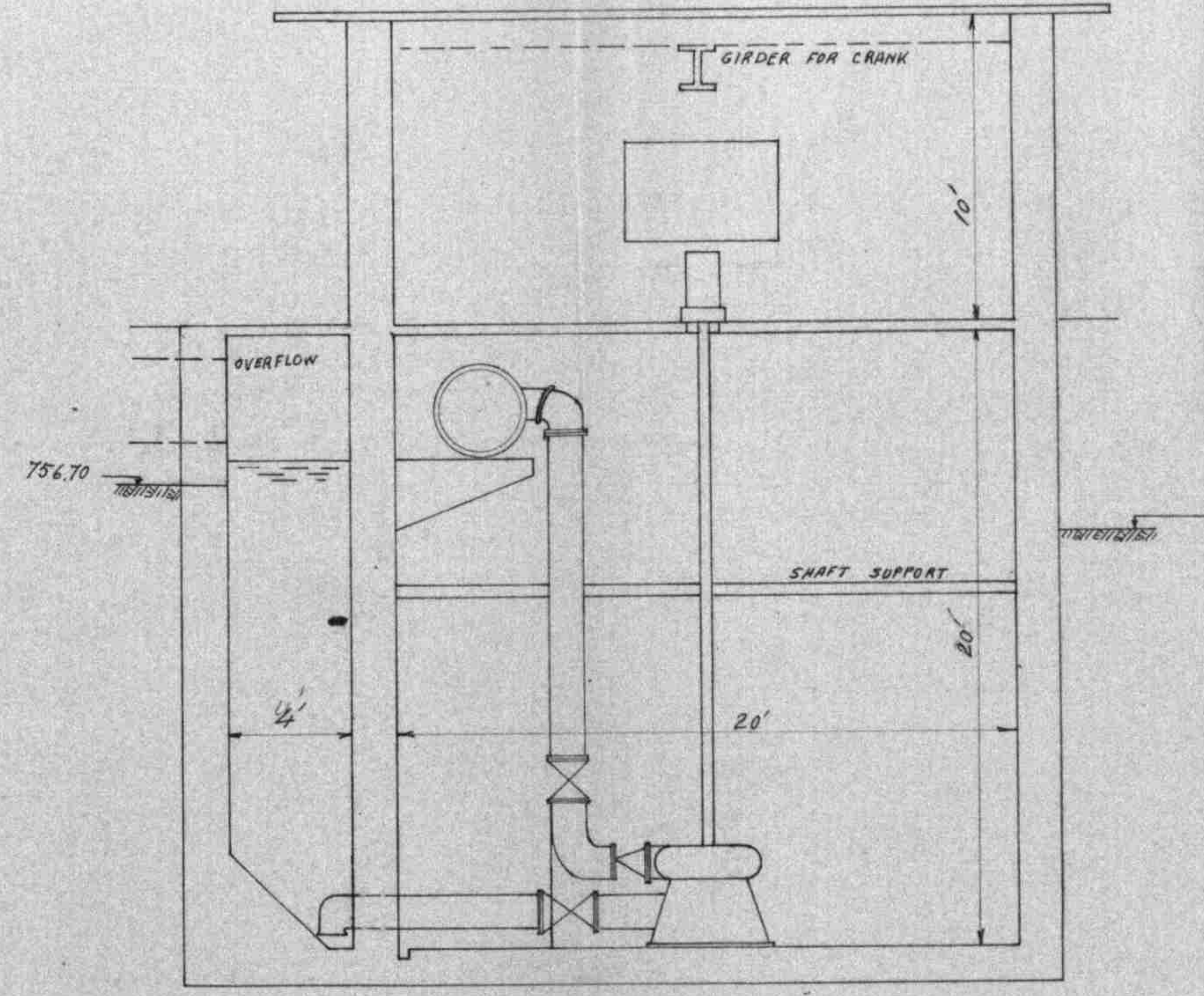


SECTION AT AA
SCALE 1/50

SULAIMANIYA SEWERAGE SCHEME	
SLUDGE DRYING BEDS	
SCALE AS SHOWN	DRAWING Nº 15
ENGINEER: M. ANIS AL-LAYLA	DATE DECEMBER 1965



PLAN



SECTION AT A-A

SULAIMANIYA SEWERAGE SCHEME	
RECIRCULATION PUMPING STATION	
SCALE 1/50	DRAWING No 16
ENGINEER M. ANIS AL-LAYLA	DATE DECEMBER 1965