PRELIMINARY DESIGN
OF
SEWERAGE SYSTEM
FOR BANNU, WEST PAKISTAN

By

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Beirut,
June, 1967

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By

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A Thesis

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Advisor:

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N. H. Afridi

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CHAPTER 1

INTRODUCTION

1.1. Scope of the Work

The idea behind this report is to prepare a preliminary design of Sewerage System for Bannu, West Pakistan. It deals with the following: the design of sewers and their material; the method of treatment and its selection from among the several; the effluent and its utilisation; and the rough cost estimate of the project.

The report being preliminary, therefore, does not deal with the detailed drawings and specifications.

1.2. The City and its Environs

1.2.1 Location

Bannu, having the population of 25,580 persons, is situated on Peshawar - D.I. Khan road at a distance of 120 miles from Pashawar, the capital city of Ex. N.W.F.P., towards its South-West. Its location with respect to the principal cities of West Pakistan is shown in figure 1.1. Its latitude is 32°-59' north and longitude 70°-37' east.

1.2.2 Historical

Bannu is the district headquarter in the D.I. Khan civil division. The jurisdiction of the district is spread over an area of about 4,900 sq. miles. Beyond the boundary of the district are Kohat civil district towards its north, D.I. Khan towards its south and Mianwali towards its east.

The city has developed to its present state from a small town, known as Bazar Ahmad Khan, which lies towards east of it. The development of the city towards its present condition started since 1847 onwards. In 1933 a fort wall, which also served as M.C. limit, was provided all round the city. Later on the M.C. limit was extended beyond the wall for accomodation purposes.

The area around the city, within 5 to 8 miles radius, is very much fertile and full of greenaries while that beyond the limit, and mostly towards its south-east, is composed of vast tracts of desert arid land. With the construction of Baran Dam on Kuram River at a distance of about 7 miles from the city, large areas of this district are now irrigated and its entire complexion is now on its way of alteration. Bananas amongst others is the most common fruit grown here while in crops such as wheat and sugar-can are harvested. Vegetables are supplied from Bannu to Kohat and D.I.K. Districts and thus the standard of the people has risen.

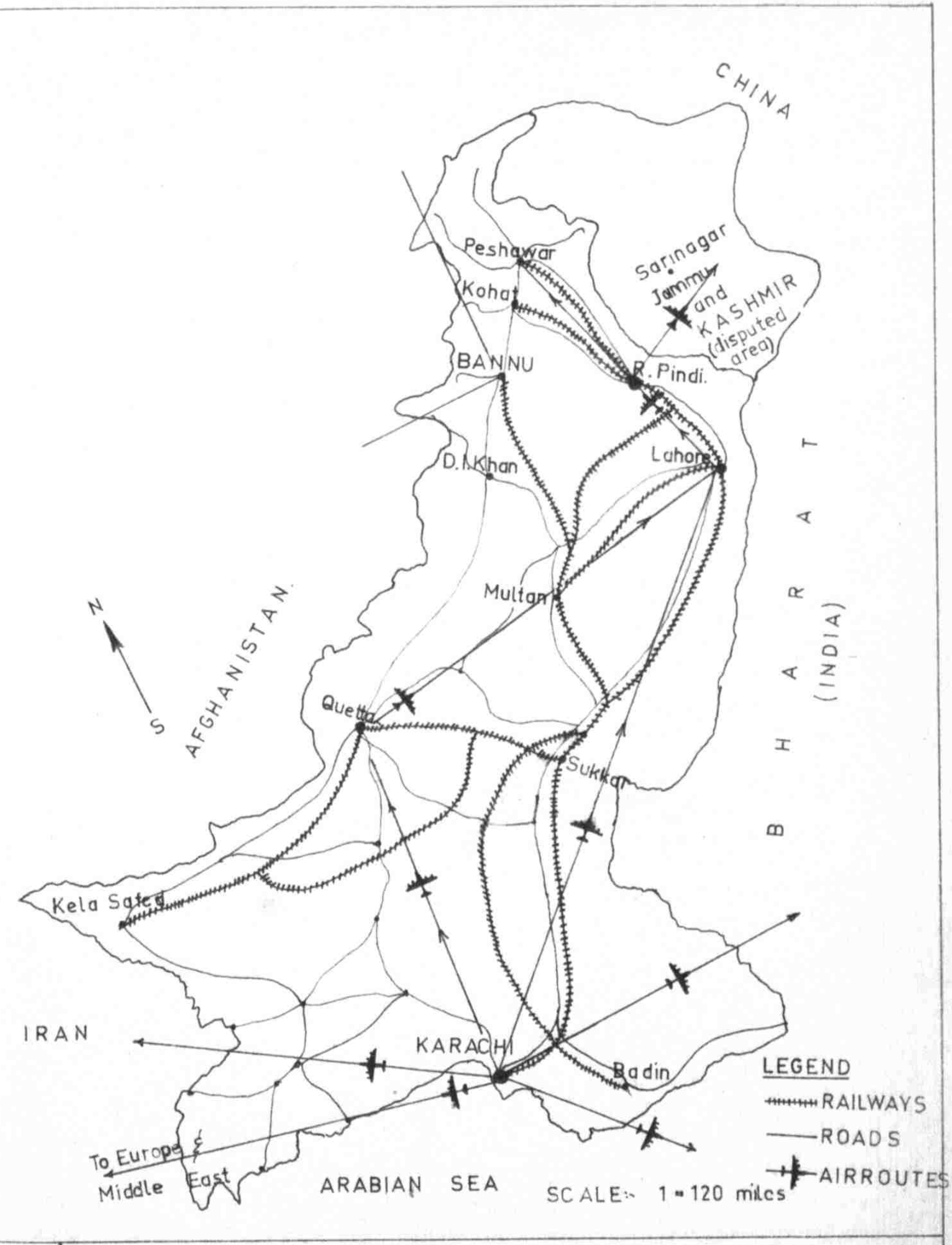


FIG. 1·1 LOCATION AND COMMUNICATION MAP

As a centre of a developing district Bannu has gained immence importance due to its timber market which supplies timber for constructional purposes mostly throughout Peshawar region. Besides this the district is also having few industries like sugar can and textile while the furniture cottage industries are also not uncommon in the city.

1.2.3 Living Quarters

Bannu city is composed of both old and new buildings with the old ones predominating. Some of the new buildings were sparsely located in a haphazard manner due to lack of proper planning. However, at present, most of the new buildings are errected in the northern part of the city. Buildings are generally double storey built in bricks while that of R.C.C. are newly errected. These newly constructed buildings are surrounded by gardens and connected by wide roads.

1.2.4 Topography

Bannu is a plain area with levels ranging from 1278' to 1306'. The river Kuram flows in West-east direction at a distance of about 2 miles towards north of the city. Most of the city slopes away from the river in North-south direction. The river, being non-perenial has a bed elevation above some of the inhabited area of the city. The elevation of the city, above mean sea level at Karachi, is 1299 ft.

1.2.5 Soil

The soil is mostly clay with admixture of sand. The soil is fertile, wheat and sugar can are the main crops which are grown. No hard and rocky excavation will be involved, hence laying of sewers and construction of sewerage treatment plant will be easier and shall cost less.

1.2.6 Sub-soil Water Level

Water and Soil Investigation Department (WASID) is busy in carrying out ground water survey throughout West Pakistan. The department has carried out exploratory borings of the area of Bannu in the middle of 1965. The ground water table along with the soil strata from two representative points of the city are shown in figures 1.2 and 1.3.

The ground water table elevations at the two representative points, within the city as shown in figure 1.2 & 1.3, are 1179' and 1178' respectively. It is therefore obvious that the water table depth would be far beyond the reach of sewers laying and hence this depth, irrespective of the seasonal variation in the water table, would not at all effect the work of laying the sewers and construction of sewerage treatment plant.

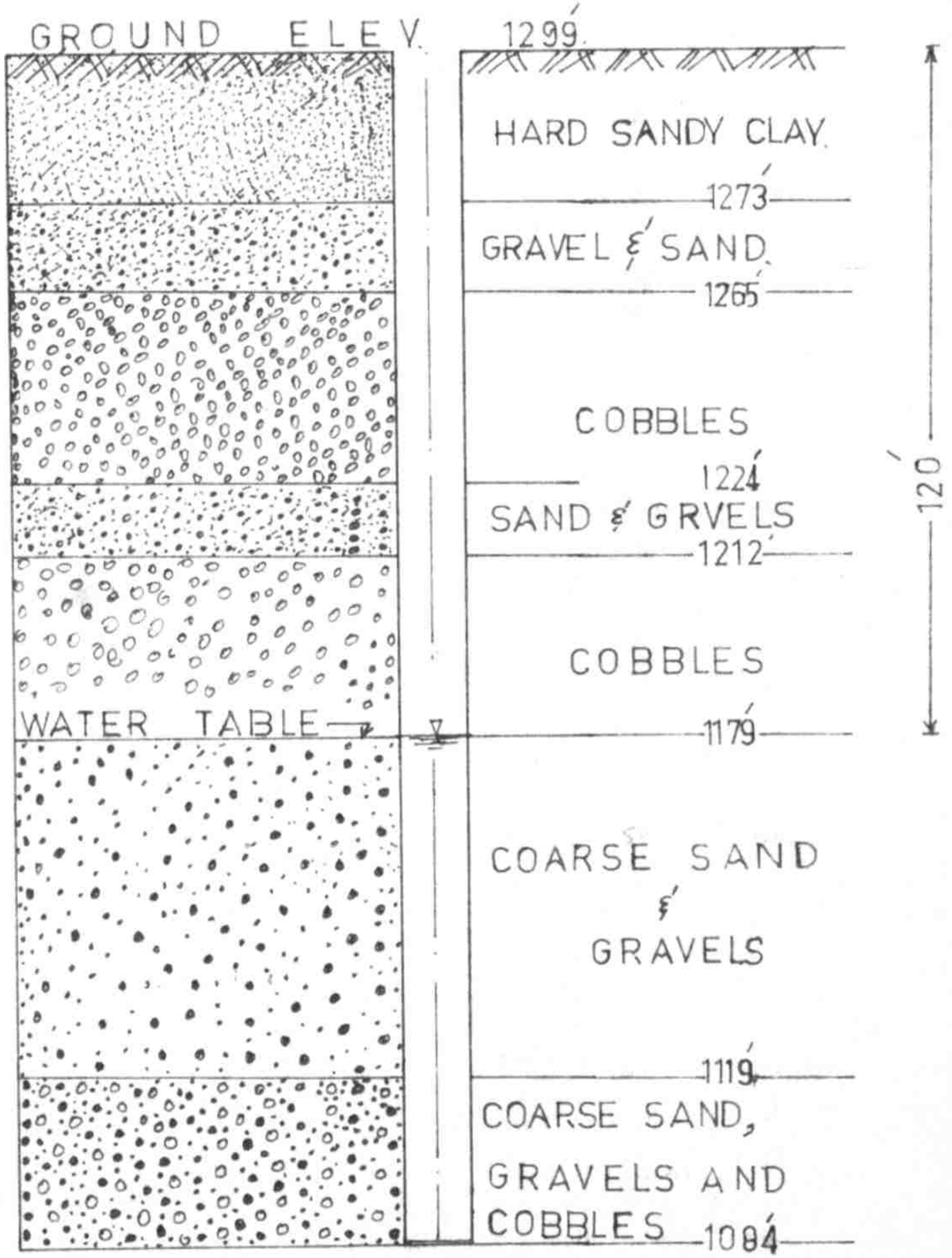


FIG. 1·2 STRATA CHART SHOWING W.TABLE LOCAT.
SCALE 1" = 30'

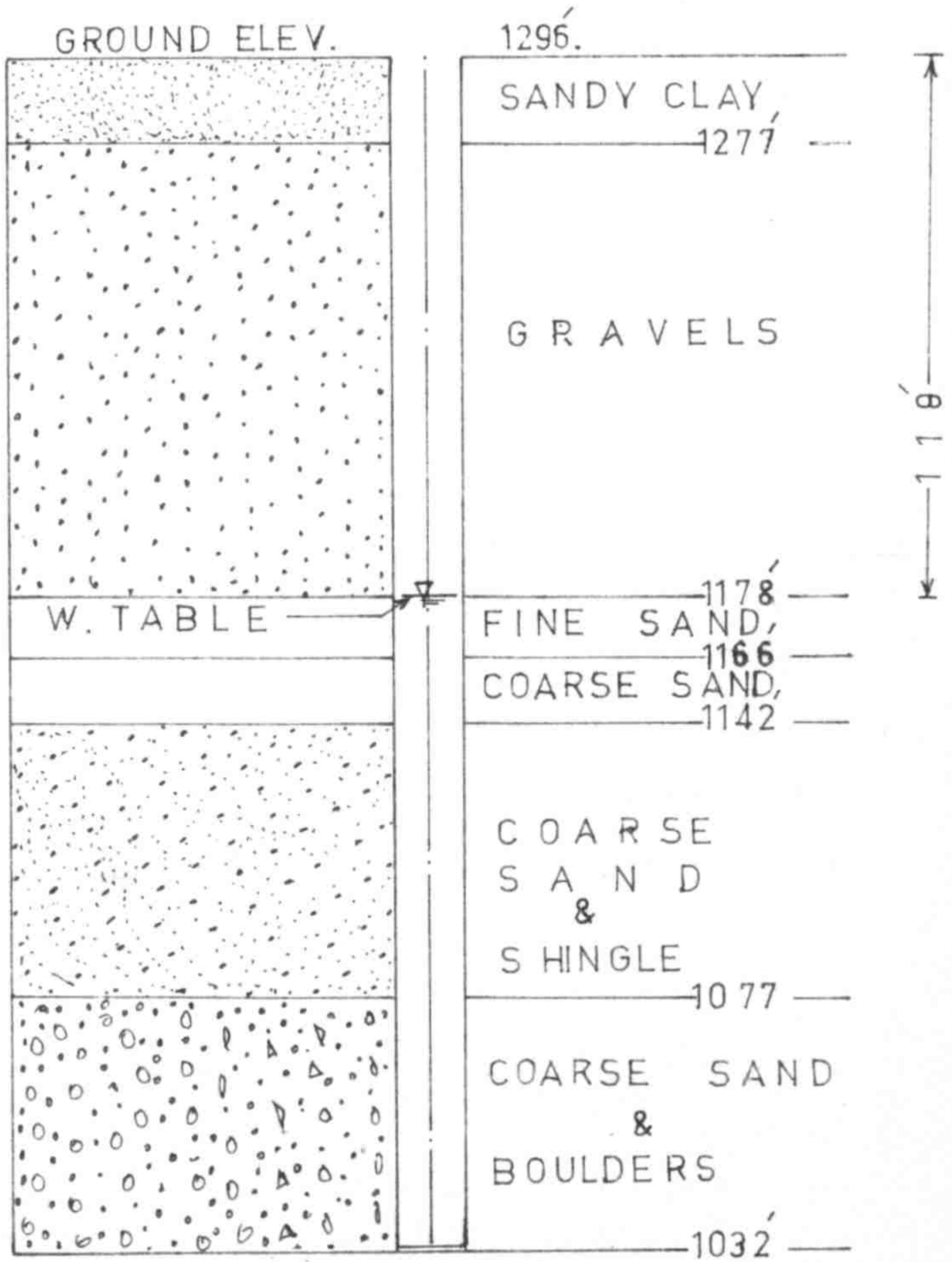


FIG. 1·3 STRATA CHART SHOWING W.TABLE LOCAT.

SCALE 1" = 30'

1.2.7 Communications

Communications of the city, with other cities of West Pakistan,are maintained by roads and railways. The city has neither an airport nor is there any such proposal in the near future, so communication with the city by airways is out of question.

A narrow guage railway line links the city with Mianwali. This is called Mari Indus Railway. Bannu is the terminal station of the railway.

A main road passes through the city which links it to Peshawar towards its North-east and D.I. Khan towards its South-east. A second black topped mettal road connects Miranshah which lies within the district. This is a very busy road from a commercial point of view because all the timber from Miranshah to Bannu is trucked by this road for further distribution throughout the country.

In addition to the above there are innumerable small roads which connect the city to important places within the district.

1.2.8 Health

No health statistics are available however from the local information it has been learned that both the morbidity and mortality rates due to communicable diseases like typhoid, dysentary, malaria are somewhat above the national average. This can be attributed to the in-sanitary conditions prevailing in the city. Flooding of low lying areas give rise to mosquito and fly breeding. Though the ground water table is low enough yet the possibility of the standing sullage percolation down into it is there.

1.2.9 Climatic Conditions

a) Temperature

No temperature statistics are available for Bannu city, however the temperature of D.I. Khan, which is the nearest city, varies from 37°F - 120°F. The rough variation of annual temperature is about 83°F and diurnal 40°F. However the day time temperature even in winter is such as to hasten the biological activities necessary for treatment of wastes. The coldest months are December & January while the hottest are July and August.

b) Precipitation

The mean annual precipitation is about 4.33 inches as shown in figure 1.4, while the annual rainfall since 1951 is given in table 1.1[1] and that the mean monthly derived from table 1.1 is given in table 1.2.

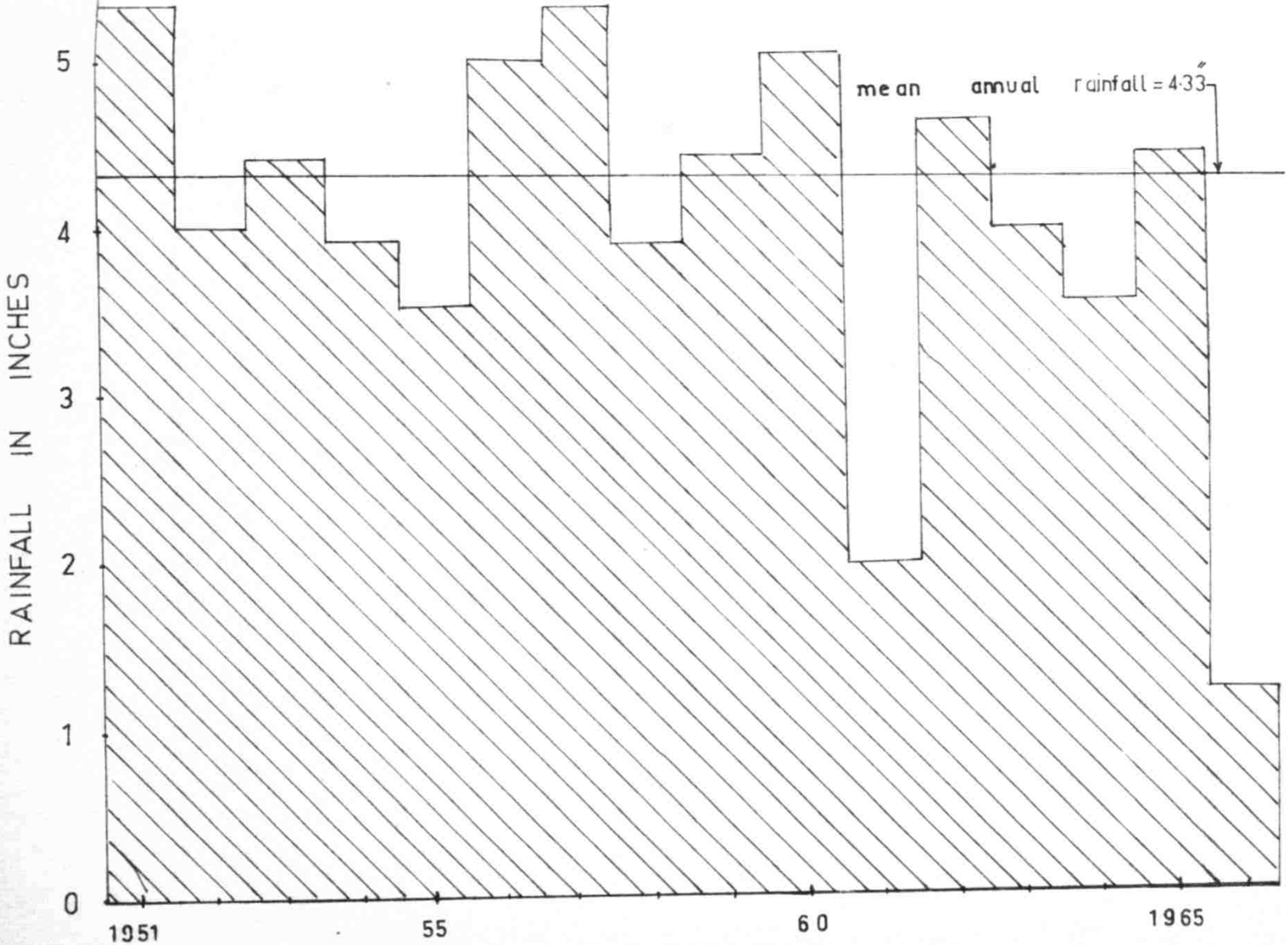


FIG. 1·4 GRAPH SHOWING ANNUAL RAINFALL SINCE 1951

TABLE 1.1 [1]

RAINFALL IN INCHES OVER BANNU

Year	January	February	March	April	May	June	July	August	September	October	November	December
1951	0.28	0.07	0.18	0.26	0.19	0.32	1.16	2.00	0.22	0.21	0.39	0.08
1952	0.24	0.10	0.12	0.14	0.16	0.22	0.50	1.00	0.68	0.20	0.47	0.41
1953	0.38	0.17	0.14	0.20	0.04	0.34	0.87	0.73	1.20	0.00	0.22	0.10
1954	0.40	0.13	0.29	0.08	0.20	0.13	0.77	1.11	0.52	0.00	0.09	0.20
1955	0.30	0.20	0.31	0.14	0.40	0.16	0.46	0.41	0.53	0.08	0.42	0.09
1956	0.25	0.15	0.41	0.38	0.59	0.21	0.39	1.16	0.65	0.21	0.33	0.27
1957	0.18	0.32	0.16	0.19	1.20	0.42	0.32	2.00	0.00	0.00	0.20	1.01
1958	0.18	0.15	0.36	0.16	0.00	0.42	0.29	0.76	0.82	0.17	0.00	0.61
1959	0.62	0.25	0.37	0.38	0.64	0.11	0.66	0.23	0.42	0.00	0.42	0.38
1960	0.16	0.09	0.31	0.86	0.05	0.37	0.58	0.97	0.22	0.13	0.00	1.31
1961	0.19	0.23	0.22	0.45	0.19	0.00	0.37	0.10	0.00	0.00	0.09	0.13
1962	0.06	0.17	0.31	0.06	0.00	0.10	0.69	0.74	1.09	0.06	1.10	0.24
1963	0.00	0.61	0.27	0.39	0.31	0.00	0.46	0.20	1.15	0.00	0.40	0.18
1964	0.36	0.14	0.22	0.13	0.65	0.15	0.36	1.09	0.27	0.00	0.04	0.11
1965	0.14	0.16	0.16	0.47	1.30	0.45	0.36	0.62	0.09	0.00	0.37	0.28
1966	0.00	0.31	0.21	0.19	0.06	0.00	0.05	0.23	0.11	0.04	0.00	0.00

TABLE 1.2

MEAN MONTHLY RAINFALL

Month	Rainfall in inches
January	0.23
February	0.20
March	0.25
April	0.28
May	0.38
June	0.21
July	0.46
August	0.84
September	0.50
October	0.07
November	0.29
December	0.34

c) Winds

The general tendency of the winds is to blow from north in the north-south direction.

1.2.10 Industries

Bannu has great importance as a commercial centre rather than an industrial one. The city is one of the important trade centers in Peshawar region. The transportation of goods is done mainly by roads and railways. Majority of the people are engaged in commerce and agriculture, however certain industries, like small cottage furnitures and a woolen mill, are also there.

1.2.11 Financial Position

Table 1.3 and figure 1.5 give the extracts from the budget report of Municipal Committee Bannu for the last 5 years. Both the income and expenditure have risen from about 0.75 million rupees in 1961-62 to more than one million rupees in 1965-66. The upward trend is bound to continue and there is no reason why the Municipal Committee would not derive a high income provided better civic services are provided.

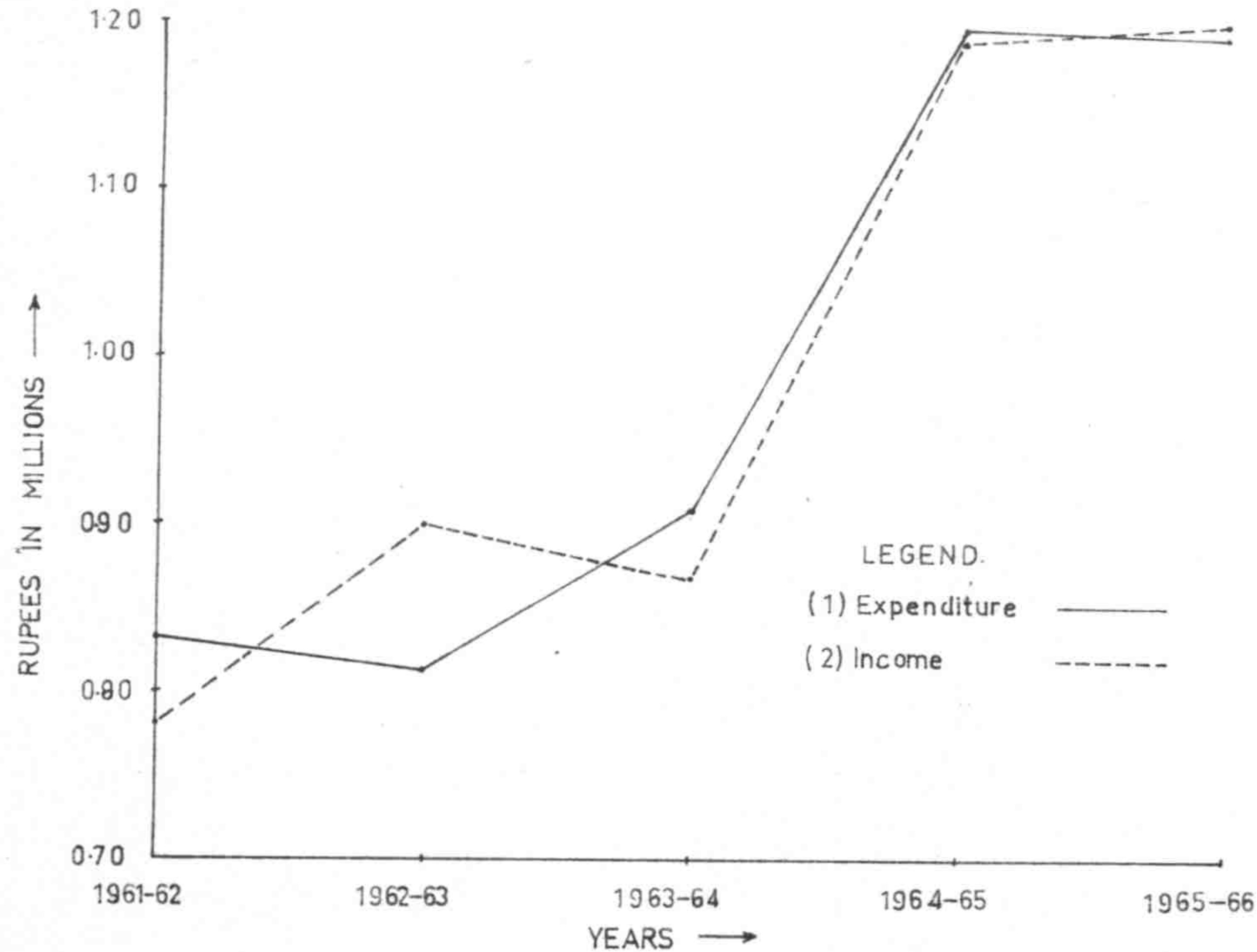


FIG. 1·5 FINANCIAL POSITION OF THE MUNICIPAL COMMITTEE SINCE 1961—62

TABLE 1.3

BUDGET POSITION OF BANNU

MUNICIPAL COMMITTEE FOR THE LAST FIVE YEARS

Year	Expenditure	Income
1961-62	Rs. 833,338	Rs 776,605
1962-63	Rs 814,028	Rs 900,640
1963-64	Rs 907,892	Rs 850,878
1964-65	Rs 1,197,487	Rs 1,190,000
1965-66	Rs 1,195,000	Rs 1,200,000

CHAPTER 2

WATER CONSUMPTION AND EXISTING CONDITIONS

2.1. General

Bannu city has a water supply scheme which is adequate to cope with the situation for about fifty years from now onwards. Record of the municipal committee reveals that the scheme was completed in 1925. It contained usual conventional treatment units, distribution system, supply main of 12" i/d & 7½ miles long and an over head steel tank of 100,000 gallons capacity floating over the supply main. However, in 1963 by reasons of inadequacy, the scheme was augmented and extended by providing additional facilities such as treatment units, a ground clean water reservoir of 50,000 gallons capacity, distribution net work and an over head reservoir of 50,000 gallons capacity.

The source of water to the treatment plant is Kuram River which has been dammed at a distance of about 8 miles from the city, mainly for irrigation and power production purposes. However raw water to the treatment units has also been taken from the power house feeding channel through a 400 ft. long channel, having X-section of 1' x 2'.

Water, after passing through the treatment units, is fed to the supply main, through the clean water reservoir, leading to the distribution net work & over head reservoir in the city. All the flow is by gravity and no pumping has been resorted to at any stage.

There is only one important industry i.e. woolen mill from the point of view of clean water utilization which has got its own supply from tube wells.

2.2. Water Consumption

The average daily per capita water consumption is a **variable** thing depending upon a number of important factors, including size of city, presence of industries, quality of the water, its cost, its pressure, the climate, characteristic of the population, whether supply is metered and efficiency of the water work administration.

2.2.1 Present Water Consumption

So far as the present water consumption is concerned no detailed information is available due to the lack of a metering system in the distribution network, however the quantity of water released to the city as per record of the municipality amounts on the average to 650,000 gallons/day. which gives a figure of 25 gallons/capita/day. The amount of used water discharged into a sewer system is generally less than the amount of water supplied to the community. The entire supply does not reach the sewers owing to leakage, lawn sprinkling manufacturing processes etc., but these losses are largely made up by additions from private water supplies, surface drainage and other accretions. It would therefore be safe to take the value of 25 gpcd. However a value of 30 gpcd would be adopted because on completion of the system water consumption would be somewhat increased.

2.2.2 Future Water Consumption

The water consumption per capita per day is not constant and varies with the passage of time depending mainly on the progress in the standards of living. Same is the case with industrial water consumption. As the quantity of liquid waste is directly proportional to the amount of water used so the waste liquid per capita would also vary with the progress in the standards of living . However it is safe to assume that the water consumption would not be different from the amount of water consumption at other communities which have the same standard of living and environment which the city of Bannu is yet expected to attain within the design period. Such a comparison with the other cities of West Pakistan e.g. Lahore can best fix the value which is 50 gpcd. However a figure of 45 gpcd would be adopted and seems to be reasonable as applied to the city of Bannu.

2.3. Existing Conditions

The city does not have any sewerage system and is served by a system of open surface drainage on the sides of brick and asphalt paved streets. These surface drains discharge into bigger drains which are finally intercepted by one peripherial drain towards the southern part of the city running in West-east direction. The whole flow is by gravity and the sewage is utilized for irrigation.

The drains at some places are in dilapidated conditions and therefore water stagnation at such places is prone to occur on account of broken sections and improper slopes. These places have become the source of mosquitoe and fly breeding. Also foul smell and poor impression is reflected to a passerbye.

Some of the buildings, which are few in number, have septic tanks while from others the excreta along with the garbage is collected daily and trucked away thus producing foul smell all along the way through which the vehicle passes.

So collectively all these result in improper sanitation causing one way or the other ill health, foul smell and unsighty appearance.

2.4. Proposal

From the view point of the aforementioned conditions prevailing in the city it is of utmost importance to do away with the old methods of disposals by the incorporation of a comprehensive sewerage system which would not pose any such sanitation problem producing ill health and undesirable effects. This system would be based on a sewage quantity of 30 gallons per capita per day for the present while the value of 45 gallons per capita per day would be adopted ultimately as discussed in section 2.2.2 .

It is further proposed to design the system for dry weather flow only excluding the wet weather flow. As the mean annual rainfall i.e. 4.33 inch is small and hence it is not advisable at this stage to consider it either for separate or combined system, however it will be discussed later on somewhat in detail.

CHAPTER 3

POPULATION ESTIMATES - PRESENT & FUTURE

Both management and design of waste water disposal system require a knowledge of the quantity of waste water produced and its relation to the population and industries if any. To have the knowledge of dry weather flow it is essential to know the population at present as well as in future. It is an admitted fact that as the population increases the use of water and the disposal of used water increases. So for proper planning for the disposal of used water, the knowledge of sanitary engineer regarding the present as well as future expected population is a must.

3.1. Design Period

It is the period which shows the number of years during which the proposed system and its component structures and equipment are to be adequate before it is abandoned or enlarged by reasons of inadequacy. In fixing the design period however consideration is given to the following factors:[2]

(1) Probable growth of the community.

(2) Availability of funds.

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

(4) Original and maintenance cost.

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

(6) The carrying charges of the sewers having surplus capacity and the difficulty of maintenance due to the system is not loaded to capacity.

The aforementioned factors are all variable however the growth in population is most variable among all of them. It depends upon the degree of industrialization, availability of transportation facilities, availability of area for future developments, initiative shown by city agencies in planning for future developments and in attracting new industries, national trends in both birth and mortality, shifts of population between urban and rural communities and installation of sewerage system and wholesome water supply facilities.

As all the factors are variable and their accurate determination or prediction for future is not beyond the doubt of approximation, so therefore the longer is the time of prediction, the more they are liable to deviate from the actual figures and more is the possibility of either under estimation or over estimation of the system. Thus the estimates for the future size and requirements are attended by uncertainty and it is usually not advisable to predict designs upon estimates of conditions which may exist for more than 50 years.

A design period of 50 years is adopted for Bannu city because of the following reasons:

(1) There is no certainty to the accomodation of increased population beyond 50 years within the municipal limits and therefore accomodation beyond the limits of the M.C. are expected.

(2) The city is already densely populated so its saturation point within 50 years is most probable.

(3) Most of the materials, which will be used in the system, have useful life upto 50 years.

(4) The ground slopes are quite favorable for reduction in the sizes of pipes which can upset the saving due to short design period.

(5) Small flows in the innitial stages will not reduce the velocity beyond permissible on account of favourable slopes.

3.2. Source of Data

As the Government carries out census in the country, the population records are obtainable for each city in the country. The population record for Bannu city for the last four decades, with present population (in 1966) of 25500 persons, is as under in table 3.1.

TABLE 3.1 (3)

CENSUS RECORD

Year	Population
1931	16,650
1941	18,380
1951	20,530
1961	23,979
1966	25,500

The population of Bannu city in 1931 was 16,650 while in 1914 it was 18,380 souls. The increase in population for the decade amounts to 1,730 which comes to be 10.4% for the decade. Similarly the population increase for the subsequent decades are 2,150, 3,449 souls with %ages as 11.7 and 16.8 respectively. The increase in population along with percentage increase is summarized in the table 3.2 decade wise.

TABLE 3.2

PERCENTAGE POPULATION INCREASE

SINCE 1931, DECADE WISE

Year	Population	Total % Increase	Annual % Increase
1931	16,650	-	-
1941	18,380	10.4	1.04
1951	20,530	11.7	1.17
1961	23,979	16.8	1.68
1966	25,500	6.35	1.27

From table 3.2 it is clear that the population increase has been fluctuating between 1 and 1.7% over the past few decades. So apart from graphic extrapolation of the census record and its extension to the period of forecast, it will be worthwhile to analyse the data by mathematical approach as well, by taking the average %age increase/year of the last 36 years. This 1.30%/year increase in population is quite reasonable as the tendency of the city, towards industrialization within the forecast period seems to be very rare, because of the limitted raw materials resources in the district.

During 1947 India was integrated into two independent states i.e. Pakistan and (Bahrat) India and thus there was a general migration of Muslims from (Bahrat) India to Pakistan and that of Hindus and Sikhs from Pakistan to (Bahrat) India. This migration would have effected the population of Bannu, had there been one sided migration. But, due to migration of Hindus etc. from the city and immigration of Muslims to the city, the population was not effected markedly. This can also be judged from the %age increase in population for that decade which yields a figure which is more or less comunsurate with the %age increases of other decades. Therefore it is safe to assume that the population of the city was not effected in 1947.

3.3. Present Population

Fortunately figures for population in 1966 are available i.e. 25,500 persons and the same can be taken as the present population with little error. However to be accurate enough the present population has been arrived at in the following paragraphs.

3.4. Future Population

Population forecasting is a matter of judgement and sanitary engineers are supposed to exercise this judgement as accurately as possible in order to have the forecast figure comunsurate with the actual. Several methods are presented here but the success of any lies in the ability to enhance or augment the judgement of the forecaster.

Changes in population can occur in only three ways: (1) Birth, (2) Death and (3) Migration. Factors effecting births and deaths are infinite in their variety. Some are negligible while others are more pronounced. The usual approach is to determine the effects and trends of pertinent factors in the past then to estimate the extent and nature of probable deviations from these past trends and effects in the future, however it is difficult. Limitations of time, data and resources may require the use of assumptions. In addition to these assumptions, several basic assumption are implicit which are as under.[4]

(i) The form of Government and political, economic and social organizations will remain substantially unchanged.

(ii) No war, internal revolution, nation wide devastation, epidemic or other disaster will occur.

(iii) No large scale epidemic, destruction by military action, fire, earthquake, or other disaster will occur in the area or within the geographical or economic region to which the area is closely related.

Population forecasting is usually achieved by the following methods:-

a) Graphical Methods

One of the common procedure of forecasting the population is the projection of the curve of past population growth. Forecasting by means of graphic extrapolation consists of (1) plotting the populating of past census years against time (2) sketching a line that, in the judgement of forecaster, appears to fit the past data and (3) extending this line into the future to obtain population for future years. This method as applied to the city of Bannu can be of use because of the availability of past records of population. So this method, supplemented by others, is being applied to the population of the city in question for the period of forecast as shown in figure 3.1.

b) Arithmetic Projection

This is plotted on plain co-ordinate paper. A straight line indicates a constant increase in population number. This shows that the population will have equal

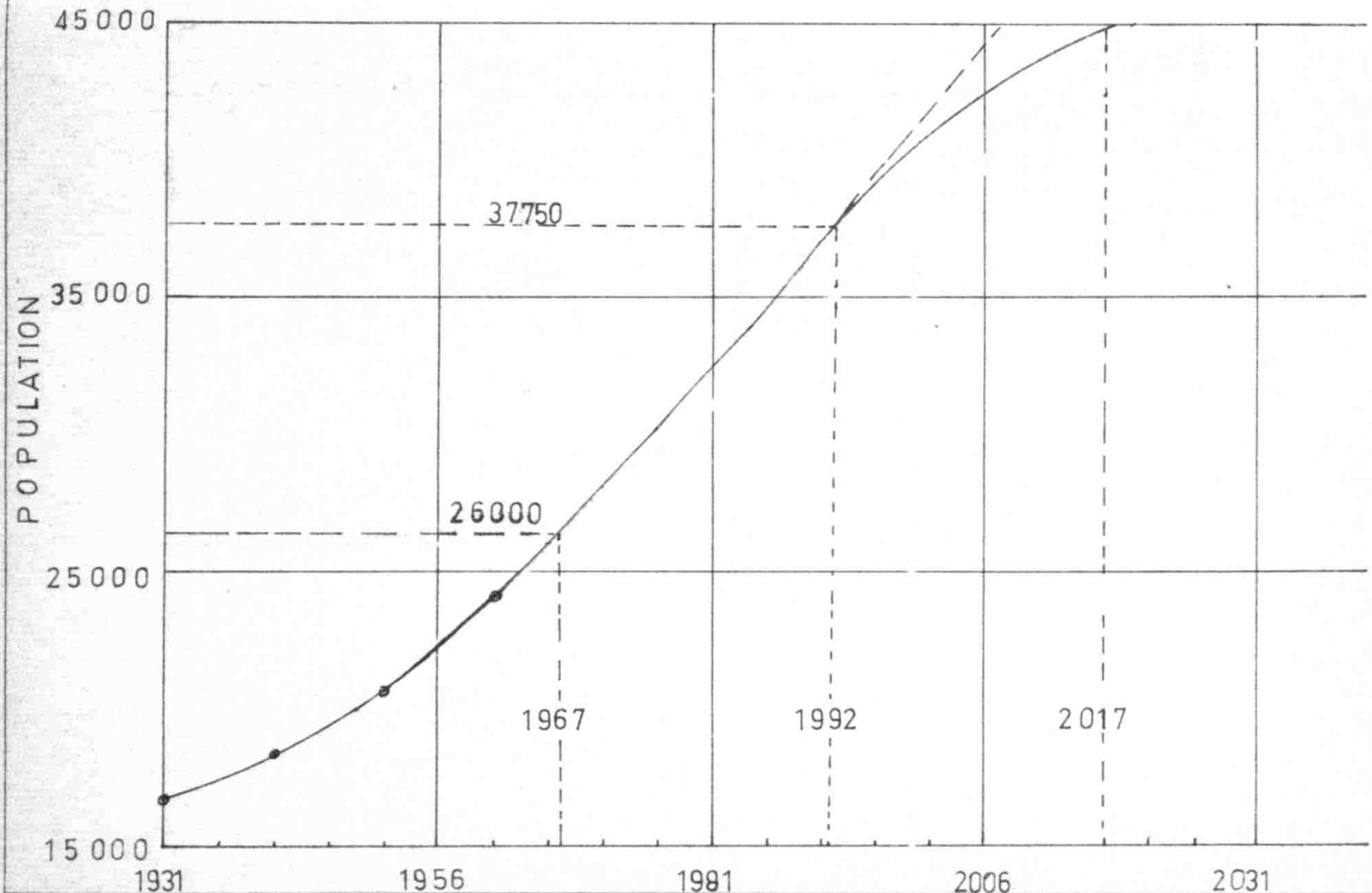


FIG. 3.1 POPULATION GROWTH BY GRAPHIC EXTRAPOLATION

rate of growth in future. This method if used alone will yield a low figure.

c) Geometric Projection

This method implies a uniform percentage rate of growth. When plotted, it produces a curve similar to that of compound interest. This method usually gives a very high result as the rate of increase of population is never constant for a long time. The results are much higher in this than the arithmetic method and as such this method alone is also not applicable to this city. However to obtain reasonable results it will be advisable to use both arithmetic and geometric progression to-gether to have the value in midway between the two i.e. neither too low nor too high.

d) Comparative Method

In this method the future growth of the area under study is assumed to follow the pattern of any older and larger area whose earlier growth has exhibited characteristics similar to those expected of the study area. Thus it consists of plotting curves of cities that one or more decades ago, head reached the present population of the city being studied. The results from this method cannot be accurate as the nature of circumstances affecting rate of growth are usually different in each case. This method requires records of other similar cities, but as there is no similar city like Bannu which may have crossed the stage of population in which the city of Bannu at present is. So this method cannot be used as applied to Bannu.

e) Ratio and Correlation Methods

Many of the factors effecting population growth occur simultaneously throughout a wide region. Thus the rate of population growth for the most areas and communities is related, to the rate of growth of state or national population. As the population forecast for larger areas is done more carefully therefore the growth for smaller areas can be calculated by using the forecast of larger areas

$$P_f/P_{f'} = P_i/P_{i'} = K$$

$$\text{or} \qquad P_f = K.P_{f'}$$

in which P_f symbolises the population forecast for the study area, $P_{f'}$ indicates the population forecast for the regional area, P_i stands for the population of the study area at the last census, $P_{i'}$ denotes the population of the regional area at the last census and K is ratio constant. The ratio K may also be taken as the average ratio of several past census. This method as applied to Bannu is not considered because this method requires firstly the computation of the ratio of local to the national population for several census years and secondly the forecast of national population. Therefore it is time consuming and uneconomical.

f) Mathematical Methods

The use of mathematical equations for population forecasting assume that past population growth has followed some identifiable mathematical relationship in which population change was a function of time and that future change in population will follow a pattern predictable from this relationship. In mathematical term it can be written as $\frac{dP}{dt} = Ka$

This expression assumes that the rate of change of population has been constant. The integration of the expression yields the following expression:

$$P_f = P_i + Ka\ (t_f - t_i)$$

in which P_f indicates the population for the forecast year t_f and P_i indicates the population of the innitial or base year t_i. The constant Ka is evaluated by re-arranging the expression given above.

$$\therefore \quad Ka = \frac{P_f - P_i}{t_f - t_i} = \frac{P_i - P_e}{t_i - t_e}$$ where P_e is the population for some earlier year t_e.

It might resonably contended that the population growth is proportional to existing population at a given moment i.e.

$$\frac{dP}{dt} = KgP$$

where $\frac{dP}{dt}$ represents the change in population per unit of time (t) and Kg is constant. The integration of the above expression yields the following equation

$$\log P_f = \log P_i + Kg\,(t_f - t_i)$$

where P_f, P_i, t_i and t_f are the same as explained previously,

$$\text{or} \quad P_f = P_i\, e^{Kg(t_f - t_i)}$$

$$\text{and } Kg = \frac{\log P_f - \log P_i}{t_f - t_i} = \frac{\log P_i - \log P_e}{t_i - t_e}$$

These two mathematical expressions and arithmatic, geometric progressions are identical and will yield therefore identical results if the later two are applied with precission.

Logistic Curve

This method of determination of population increase by logistic curve as devised by Raymond Pearl, has been found to yield more accurate results than the previously described methods. The population at any time is determined by the following equation:

$$Y = \frac{K}{1 + me^{ax}}$$

where Y = population at any time x

K = saturation population

m, a are constants

The constants are determined by selecting three points uniformly spaced along x-axis on the curve of past population record. However incase of Bannu it is not applied for the reason that the census record is for smaller period and hence does not yield comunsurate value for the constant "K".

g) Decrease Rate of Growth

As a general rule it is found that as the population increases, the rate of growth decreases. Results having considerable accuracy can be derived by study of past trends of decreasing rate over a considerable period and taking into account the present and future political and economic developments. A decreasing rate of growth can be assumed and population forecasting is made. As this method requires sufficient past information and therefore cannot be applied here.

h) Component Methods

As said before population change can occur only in three ways (1) through births (2) through death (3) through migration. Thus the population forecasts are obtained by preparing separate but related projection of natural increase and net migration and adding the component projections. The relationship may be written in the following form:

$$P_f = P_i + N \pm M$$

in which P_f and P_i symbolise forecast and present population respectively. N stands for net natural increase during the forecast period and M stands for net migration during the forecast period. However this method is difficult to apply here because of the lack of data relating to "N" and "M".

Employment Forecast Method

The primary reason for migration is the economic improvement. Economic opportunity may be translated to a large degree as employment opportunity. If the future development of an area could be forecast accurately, the projection of migration in the component methods would be greatly simplified.

Population forecast may also be prepared from employment forecast.

$$\frac{\text{Total out put}}{\text{Out put per worker}} = \text{Employment}$$

This method, needs development forecasting of the area and other employments and then forecasting for population, is therefore not considered further as applied to the city of Bannu.

TABLE 3.3

FUTURE POPULATION ACCORDING TO

GRAPHIC EXTRAPOLATION

Year	Population
1967	26,000
1977	30,000
1987	35,000
1997	39,350
2007	42,600
2017	44,800

TABLE 3.4

FUTURE POPULATION ACCORDING TO

ARITHMETIC PROJECTION

Year	Population with Constant rate growth	Population with decreasing rate growth
1967	25,560	25,560
1977	28,700	28,700
1987	31,800	31,800
1997	34,900	34,800
2007	38,000	37,400
2017	41,200	39,200

TABLE 3.5

FUTURE POPULATION ACCORDING TO

GEOMETRIC PROJECTION

Year	Population (Constant rate)	Population with decreasing rate growth
1967	25,600	25,600
1977	27,200	29,200
1987	33,200	33,200
1997	37,700	37,400
2007	43,000	42,200
2017	48,700	46,200

TABLE 3.6

FUTURE POPULATION FROM CURVES "C"

FIG. 3.2

Year	Population
1967	25,580
1977	28,250
1987	32,500
1997	36,100
2007	39,800
2017	42,700

TABLE 3.7

FUTURE POPULATION FROM FIG. 3.3

Year	Population
1967	26,000
1977	29,500
1987	33,700
1997	37,700
2007	41,500
2017	44,000

Table 3.7 is the final table for future population forecast and has been arrived at by taking the mean of the populations from table 3.3 and table 3.6.

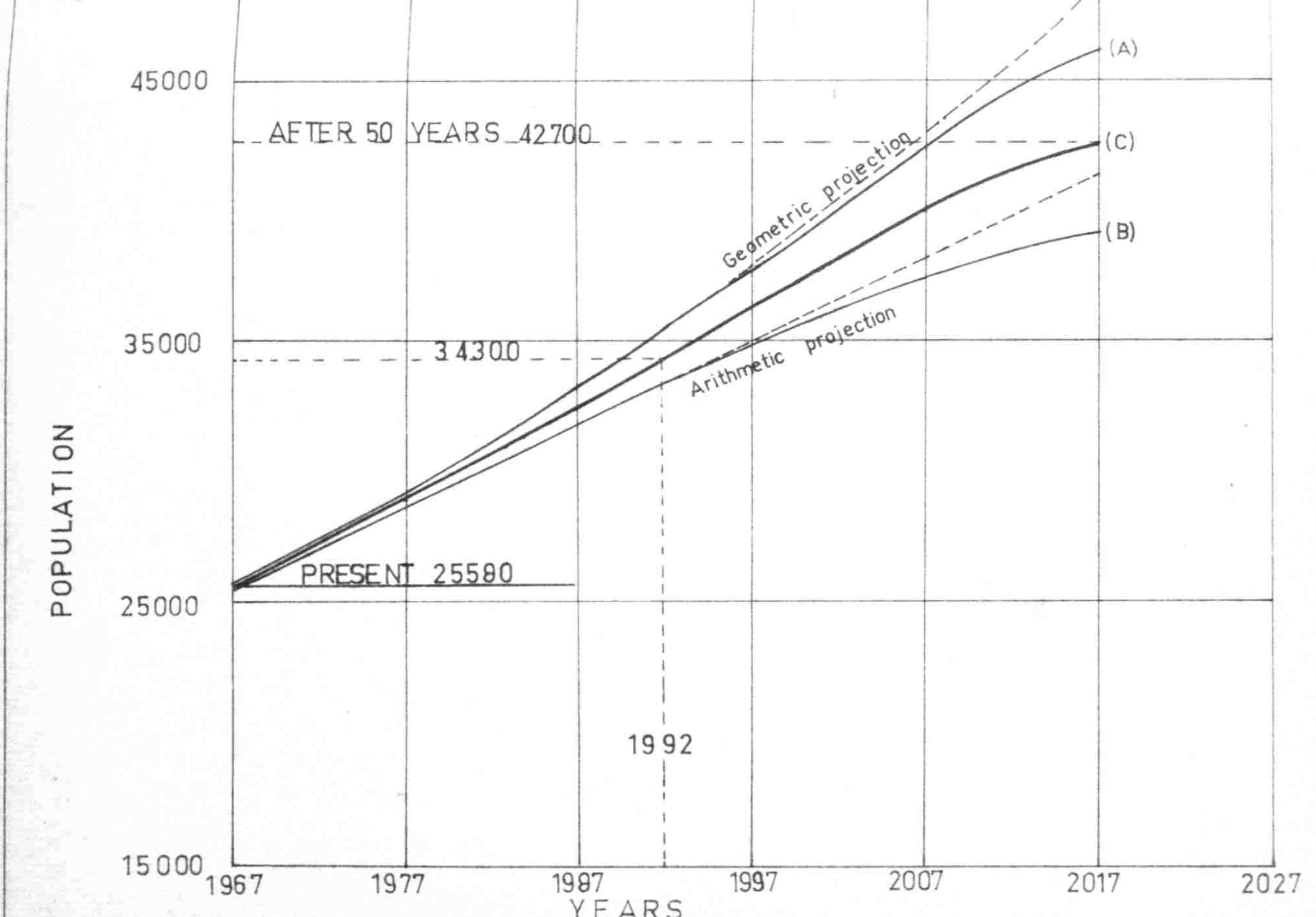


FIG. 3.2 POPULATION GROWTH CURVES.

It is assumed that the rate of growth of population from now 25 years onwards will tend to decrease because of limited space for accomodation purpose within the M.C. limit and also of limited raw material resources for industrial development purpose. Hence therefore the graphs in figures 3.1, 3.2 and 3.3 tend to bend down from 1992 onwards till saturation point is reached.

3.5. Direction of Development

3.5.1 Present Direction

Bannu has developed from Bazar Ahmed Khan which lies in the east of the present city and is characterized with its old houses, haphazardly located buildings with no proper planning. The origin of the city to its present position started since 1947 under planned manner. The city was provided with a fence wall in 1933 which also defined the M.C. limit. Later on the limit of M.C. was extended beyond this limit for the purpose of accomodating more population. The city does not have any Master plan, however for the purpose of checking irregular growth allocation of blocks for commercial, residential and industrial use have been made. Residential erections are taking place towards the north of

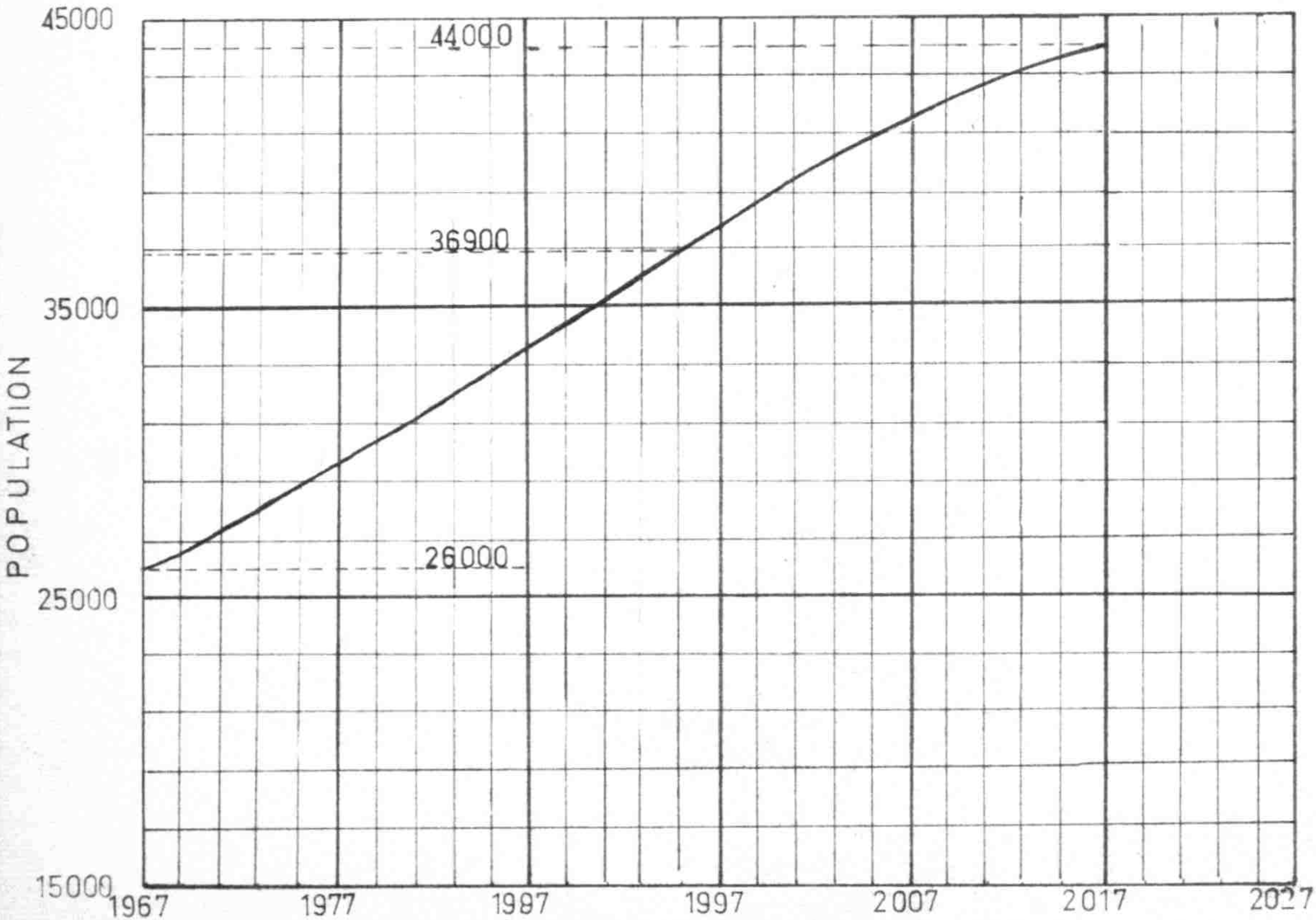


FIG. 3·3 POPULATION GROWTH CURVE FROM FIG 3·1 & 3·2 the city while industries, if any will be errected on Bannu-D.I.Khan road in the eastern side of the city.

The city is a commercial and trading centre for timber and induces the merchants to reside within the city. Transportation of goods to and from the city is maintained by means of a net work of roads and a railway line which has a great effect on the development of the city.

3.5.2 Future Direction

Bannu, apart from being a commercial and trading centre for timber, is also having numerous small cottage furniture industries and a woolen mill within the M.C. limit. With the passage of time and increase in population further extension in these industries is expected. New houses and buildings will be errected whenever and wherever needed in accordance with the blocks allocation. However it is expected that about the whole of the city within the M.C. limit will be built up within the period of 50 years resulting in saturation of the city.

3.6. Population Densities

3.6.1 Present Population Densities

There are different densities in different parts of the city, however in 1961 the census was taken wards wise and its analysis gave an average intensity of population which represent a very correct picture of the densities in various parts of the city. In this way the city is divided into several zones including purely residential, commercial, public buildings and industrial areas. The estimated population densities for the present are given in table 3.8 and are shown in drawing No. 2.

3.7. Future Population Densities

The density of population, in persons per acre, varies greatly within a city, and it is difficult to estimate its probable future changes. A residential section of the present decade may become a commercial or manufacturing district in the next decade, or the detached homes of persons of means may be replaced by crowded tenements or commercial development. However effective zoning plays an important role in this respect. At allows the occupancies under preplanned program and thus gives the relative certainty and economy with which an engineer can adopt the sewers to the service they will be called on to perform.

In arriving at the future population densities considerations are being given to the general tendency of the people e.g. persons of means would hesitate to settle in the thickly populated parts of the city while labor class of people would prefer to settle near to the industries. The future population densities have been presented in table 3.9 and are shown in Drawing No. 2.

TABLE 3.8

PRESENT POPULATION DENSITIES

Zone	Area in acres	Pop. density	Number of persons
A	201	51	10,200
B	252	40	10,100
C	301	19	5,700
Total	754	-	26,000

3.7. Future Population Densities

Population densities for the future i.e. 2017 are given in table 3.9.

TABLE 3.9

POPULATION DENSITIES FOR THE YEAR 2017

Zone	Area in acres	Population density	Number of persons
A	201	70	14,070
B	252	65	16,400
C	301	45	13,530
Total	754	-	44,000

CHAPTER 4

EVALUATION OF DATA FOR SEWERAGE PLANNING

Liquid wastes both sanitary and industrial are derived largely from the water supply. Accordingly an estimate of the amount of such wastes must be prefaced by a study of water consumption both for present and future conditions. What proportion of the water consumed will reach the sewers is decided after careful considerations of the local conditions e.g. some industries may have their own water supply but they may contribute their wastes to the sewers. Bannu woolen mill has its own supply but the contribution of waste will be there. However a good design and proper operation of any sewerage system, throughout its period of design, are of great concern for an accurate determination of the quantity of waste to be taken care of by the system. The quantity of waste upon which to base further planning and design, is of utmost importance and is considered as under:-

4.1. Municipal Wastes

4.1.1 Sewage from Industrial and Commercial Areas

Waste produced by individuals varies, however an average value per person per day is usually less than the per capita daily water consumption because of such uses of water as for irrigation, gardening and sprinkling etc. The quantity of sewage produced by residential and commercial areas may vary for individual cities from 70% to 130%[5] of the water consumed, but frequently it is assumed that the average rate of sewage flow including a moderate allowance for infilteration, equals the average rate of water consumption[5]. Therefore the sewage produced at residential and commercial areas of Bannu city would be taken as 30 gpcd for the present and 45 gpcd for the future (discussed earlier).

4.1.2 Infilteration

Infilteration is the water that finds its way into the sewers through poor joints, cracked pipes, the walls of the manholes, perforated manhole covers, and the drains from flooded cellars. Though the possibility of infilteration existence in areas with low water table during dry weather is very rare yet in wet weather it will be greatly increased and may be augmented by unauthorized roofs draining gutters connection to the sewers.

The quantity of water entering the sewers depends upon the following:

(1) The height of water table in relation to the sewers.

(2) Type of the soil.

(3) Permeability of the soil.

(4) Workmanship exercised in the construction of sewers and their appurtenances.

Since conditions of construction and soil differ widely, the infilteration found in sewer system varies widely. Some of the usual allowances for ground water infilteration are as under[2].

(1) 500-5000 gpd per acre, average 2,000 gpd.

(2) 500-5000 gpd per mile of sewer per inch diameter (average 2500) plus 100 gpd per manhole.

(3) 5,000-100,000 gpd per mile of sewer average 30,000.

The infilteration figures mentioned above cannot be applied to Bannu city for reason of very low water table and low rain-fall. However it is proposed to have water tight joints and manholes in addition to adhering to strict inspection and good workmanship of house and buildings connection in order to further reduce the possibility of ground water infilteration if any. So on these grounds it is not far from justification by providing a nominal allowance for infilteration by taking the quantity of sewage equal to the quantity of water consumed as explained in sub-section 4.1.1 of this chapter.

4.2. Strength of Sewage

Strength of sewage is usually expressed by its B.O.D. value which varies between 0.08 - 0.17 lbs(6) per capita per day depending mainly upon the type of food stuff and domestic habits. As so far no investigations have been made in arriving at a definite figure in determining the strength of sewage with respect to B.O.D., therefore a value of 0.17 lb(7) per capita per day will be adopted in a like-manner as is usually done in case of other cities of Pakistan by American Consulting Engineers to Pakistan. It is probable that this is a reasonable adoption since serious deviations from the adopted value are not likely to occur in the future, as the sewage in question will be almost entirely domestic. Therefore

the ultimate total B.O.D. in lbs = 0.17 x 44,000

= 7480 lbs.

Ultimate volume of waste = 44,000 x 45

= 1,980,000 gallon.

4.3. Industrial Wastes

The treatment of industrial wastes, if possible, with the municipal sewage would be advisable due to the following reasons:

(1) The construction of one treatment work would cost less than a number of scattered smaller treatment work.

(2) The health authorities would be able to deal with one organization instead of a multitude of small entities.

(3) The treatment would be concentrated at one central site, instead of at a number of small scattered plants.

(4) Lower operating cost would be possible, as one large modern treatment plant would cost less to operate than a number of smaller ones. The result would be lessened cost per annum to the tax payers in the group.

(5) The cost of policing would be reduced as there would be only one plant to look after and maintain at efficiency, instead of a number of small ones.

(6) The joint body would be in better position to obtain and retain more experienced and skillful operating personnel.

(7) The industries would be relieved from constructing long and independent sewer lines up to the points of final disposal.

(8) The plant would be comparatively of larger size and therefore could warrant the use of the by-products of the plant processes.

However careful study of the wastes would be ensured so as to avoid deleterious effects of the wastes on the municipal sewerage system. It will, in many cases, be more economical for an industry to reduce its wastes to the acceptable degree at their source, if by that means they can then be discharged into municipal systems, than to install complete treatment works of its own. The wastes and their character are considered in the subsequent sections.

4.3.1 Present Waste

Bannu, at present, is neither an industrial city nor there is any future bright hope for its becoming so. However there are certain industries, like woolen mill, slaughterhouse, and furniture cottage, which will contribute wastes to the sewers. As no tanning industry exists in the city, hides are trucked to Peshawar for tanning purposes. However with the increase in population the erection of a tanning factory is expected in the future.

The volume and strength of wastes produced by each factory vary widely and are discussed as under:

a) Small Cottage Furniture Industries

These are numerous, however the waste produced is mainly in the form of wood trimmings and is, therefore, not of sanitary significance. The process is dry one and does not contribute any organic waste either in dry or liquid form to the sewers. So therefore in arriving at the strength and volume of waste these industries are neglected altogether.

b) Woolen Mill

The wastes produced in the manufacture of woolen textiles are contributed by the wool scouring carbonising, bleaching, dyeing and finishing processes. All may be discharged from the same manufacturing plant, but more often wool scouring and corbonising are done in one plant and dyeing and finishing elsewhere. The composition and quantity of waste are of variable nature depending upon the kind of process and impurities in the raw material to be processed.

The actual wool fiber as taken from sheep's back averages[(8)] only 40%, the remaining 60% being composed of natural impurities like sand, grease, suint and burrs. So as a result 2½ lbs of grease wool on scouring gives only 1 lb of scoured wool. In terms of B.O.D. 200 - 250 lbs are discharged per 1,000 lbs of scoured wool produced. Wool scouring wastes, being high in B.O.D., suspended solids and other impurities, if discharged into a sewerage system would effect the treatment of sewage of a municipality in proportion to the relative volume of waste and sewage. Therefore unless a sewage plant has sufficient capacity to take the heavy load contributed by wool scouring waste they should be excluded from the sewerage system.

Wool scouring and finishing mills produce a composite effluent having the following[(8)]:

(1) pH of 9 to 10.5.

(2) B.O.D. 900 ppm.

(3) Total solids 3000 ppm.

(4) Total alkalinity 600 ppm.

(5) Total chromium 4 ppm.

(6) Suspended solids 1000 ppm.

(7) Liquid waste 70,000 gallons per 1000 lbs of wool.

Bannu woolen mill, however is not on a large scale and does not include all the process starting from raw material up to the finished products, but carries only dyeing and finishing producing a finished product of 3,000 lbs daily.

The wastes produced from dyeing and finishing processes are contributed by spent liquors and subsequent washings after singeing, bleaching, dyeing and finishing. It is usually impractical to separate the rinse or wash waters from the stronger wastes within the plant, and these are collected in common drain for treatment. The quantity of waste varies greatly with a mean volume of about 10 gallons per lb[11].

These wastes also vary greatly in strength because of various types of dyes and other chemicals used in the finishing, however treatment by chemical precipitation with alum or iron salts followed by sedimentation is effective but would not be resorted to, because biological treatment with trickling filters with small amount of domestic sewage has resulted in B.O.D. reduction of about 80%[9]. Therefore treatment of small amount of waste with comparatively large amount of domestic sewage will not create any problem.

The average polluting constituent of wool dyeing wastes are given per 100 lbs of wool dyed[9]

(1) Total dissolved solids	=	0.53
(2) Volatile dissolved solids	=	0.18
(3) Total suspended solids	=	0.085
(4) Volatile suspended solids	=	0.026
(5) Alkalinity	=	0.31
(6) Fats	=	0.056
(7) 5-day B.O.D.	=	0.470

Total B.O.D. Contribution $= \frac{3,000}{100} \times 0.47$

$= 14.10$ lbs.

Total volume of waste $= 10 \times 3,000$

$= 30,000$ gallons

c) Slaughter House

The wastes from slaughter houses are chemically similar to domestic sewage but are considerably more concentrated. Nearly every operation from the stock-yard through the slaughter house is a source of waste. The type of waste produced by the separate operations are shown in table 4.1.

TABLE 4.1[9]

Source	Waste
Stock yard	Manure
Killing floor	Blood
Dehairing	Hair and Dirt
Insides removal	Paunch manure & liquor
Rendering	Stick liquor or press liquor
Carcass dressing	Flesh, grease, blood and manure

The volume of waste produced in the slaughter house is given by table 4.2

TABLE 4.2[6]

	Type of kill	Vol. per animal in gallons	Suspended solid ppm	Org. Nitrogen ppm	B.O.D. ppm	Population equivalent
Slaughter House Waste	Mixed	359	929	324	2240	40.2
	Cattle	395	820	154	996	19.6
	Hogs	143	717	122	1046	7.5

The most common method for these wastes is in municipal treatment plants, because usually they are small in volume and consequently will not greatly effect the capacities of sewage treatment units. In small cities however these wastes may be a factor in the treatment and may necessitate some control work like the prevention of blood and paunch manure into the sewers and provision of grease traps in the sewer line from slaughter house. The rest of the waste does not constitute any problem[(6)].

The average number of animals slaughtered daily are as under:-

Sheep and goats	80 - 100
Cows	7 - 10

Adopting a daily kill of 110 animals per day, the quantity of waste produced per kill is 359 gallons.

So total quantity of waste $= 359 \times 110$

$= 39{,}490$ gallons/day

The strength of waste is considered to be 2240 ppm of B.O.D.

Therefore the amount of pollution contributed by the slaughter house $= \dfrac{39{,}490 \times 2240 \times 8.34}{10^6}$

$= 740$ pounds of B.O.D. per day.

d) Tannery

At present there is no tannery and the hides are tucked to Peshawar for tanning purposes.

4.3.2 Future Industrial Waste

It is natural that an industrial plant is located[10] (1) where it may receive its raw materials at low cost, (2) where there is a ready local market for its product, (3) where there is an ample and cheap supply of water, (4) where ample cheap power is available, (5) where an ample supply of labor exists, and (6) where a stream exists into which its wastes may be discharged.

Due to lack of some of the aforementioned factors as well as non existence of a proper projected planning for the future industrial developments of the city, it is very difficult to predict the type and size of industry but even then it can be said that the city should develop in order to support its increased population.

The industries which could be forecast are as under:

a) Extension in slaughter house in proportion to the increase in population.
b) Installation of a tanning industry.

a) Slaughter House

The average daily kill in 1967 = 110

The average daily kill in 2017 would be

$$= \frac{44000}{26000} \times 110 = 186$$

Number of cows $= \frac{44000}{26000} \times 10 = 17$

Therefore number of sheep = 186 - 17

= 169

Taking the daily kill of 186 animals per day in 2017, the quantity of waste produced per kill is 359[(6)] gallons as shown in table 4.2.

Total quantity of waste = 359 x 186

= 67000 gall./day

The amount of pollution contributed by slaughter house

$$= \frac{67000}{10^6} \times 2240 \times 8.34$$

= 1250 lbs.

b) Tanning Industry

The tannery will produce on average 186 hides per day as received from slaughter house. The volume and strength of the waste are calculated as under:

Volume of waste per sheep hide = 4 gallons[(6)]

Volume from 169 sheep hide = 676 gallon/day

Volume of waste per cow hide = 360 gallons[(6)]

Volume from 17 cow hides = 6,120 gallons/day

Total volume of waste from tanning house

= 676 + 6,120

= 6,796 gallons/day

The strength of waste is considered to be 1,200 ppm

Total pollution from tannery

$$= \frac{1,200 \times 6,796 \times 8.34}{10^6}$$

= 68 lbs.

The volume of tannery waste as compared to the municipal waste is small, but still it would be advisable to give it pretreatment such as screening and mixing of the span tan liquor with the waste from beam house and then settling before it is admitted to the sewers.

Assuming 30% reduction in B.O.D. Then the B.O.D. load per day = 68 x 0.70 = 47.60 lbs.

c) Other Industries

The industrial development other than stated in (a) and (b) cannot be forecast due to the lack of information given earlier in this chapter. However a value of 5,000 population equivalent is assumed on the presumption that the pace of progress in this aspect would not be markable.

Therefore waste contribution per day

= 5,000 x 45 = 225,000

B.O.D. contribution in lbs/day

= 5,000 x 0.17 = 850 lbs.

Summary of the Data

(1) Volume of waste in gallons/day in 1967

= 849,490

(2) B.O.D. load in lbs/day in 1967 = 5,174

(3) Volume of waste in gallons/day in 2017

= 2,308,796

(4) B.O.D. load in lbs/day in 2017 = 9,641

4.4. Storm Water

Due to small rainfall over Bannu it is advisable not to make any provision in the report for storm water either in combined form or in separate system due to the following reasons:

a) Damage

No property damage has been reported so far.

b) Economic Considerations

Existing surface drains, with little repairs, can function nicely without any pumping due to naturally available slope, thus resulting in sufficient saving.

c) Municipality Financial Position

Capacity of storm sewers in comparison with sanitary sewers is usually large enough and therefore will result in heavy expenditure which will be an unnecessary burden to the municipal committee in the light of its present financial position.

However certain basic principles, regarding evaluation of data for storm sewers, are outlined as a guide for the designer of the storm sewers in future if needed.

4.4.1 Intensity, Duration and Frequency of Rainfall

Rainfall intensity, duration and frequency all have important bearing on the capacity of storm water sewers. The shorter the duration of a rainfall, the greater will be the expected average intensity during that period. The critical duration of rainfall for any drainage area is that which produces maximum runoff and this is also called the time of concentration. Also of equal importance is the time interval between two consecutive occurences of a certain intensity rainfall with certain duration. So establishment of relation among intensity, duration and frequency should be obtained from past rainfall data. The time period is generally taken from 5 min. to 50 min.(5) and very seldom up to 2 hrs, because the time of concentration is usually less than 50 min. So in this way several curves of different frequency can be obtained.

Now the curve, among the several, to be used depends upon the selection of the time interval at which the sewer may be expected to be over-taxed. Neither too large sewers to care for the greatest rainfall nor too small, to be frequently over-taxed are desirable. Over-taxing between 10 to 15 years(5) interval is generally provided for. However, consideration of the resulting damage from over-taxing may fix the interval best.

No data for rainfall concerning the relation among intensity, duration and frequency for Bannu proper is available. Moreover data relating to the same information for any other nearest city like Peshawar cannot be used because of the tremendous variation of rainfall over the region. However it is advisable to collect rainfall data in this connection for Bannu for future use.

4.4.2 Percent Imperviousness. (Run off Co-efficient)

The present imperviousness or runoff coefficient accounts for the rainfall which after reaching ground surface does not appear as an overland flow due to interception, evaporation, percolation and filling of depressions. Therefore the runoff co-efficient is not constant but tends to become larger as the rainfall continues. The most important factors upon which the co-efficient depends are summarized below:

(1) Type of surface.

(2) Vegetative cover.

(3) Topography.

(4) Whether the area is built up or not.

(5) Temperature.

The run off co-efficient for various surfaces are given in table 4.4.

TABLE 4.4 [11]

VALUES OF PERCENT IMPERVIOUSNESS

Type of Surface	Percent Imperviousness
1. Water tight roof surfaces	0.70 - 0.95
2. Asphalt Pavement in good order	0.85 - 0.90
3. Most densely populated area	0.70 - 0.90
4. Stone, brick and wood-block pavements with rightly cemented joints	0.75 - 0.85
5. As above but uncemented joints	0.50 - 0.70
6. Inferior block pavements with open joints	0.40 - 0.50
7. Macadamised roadways	0.25 - 0.60
8. Gravel road ways and walks	0.15 - 0.30
9. Unpaved surfaces, railroad yards and vacant lots	0.10 - 0.30
10. Parks, garden, lawns and meadows depending on surface slope and subsoil character.	0.05 - 0.25
11. Wooded areas of forest land - depending on surface slope and characteristic of subsoil	0.01 - 0.20

The above values are given for the sake of easy reference, however the values to be used in the design should be the representative values of the various sections of the city. Therefore no definite values can be proposed at this stage as it would be too early.

CHAPTER 5

SEWERAGE SYSTEM

Sewerage implies the collecting of waste wasters from occupied areas and conveying them to some point of disposal with or without treatment. A well designed system is of great importance from the health point of view and therefore providing an urban area with sewerage facilities requires careful engineering. Cities are generally sewered either by Combined System or Separate System, however the choice of one or the other depends upon the consideration of certain factors in relation to the prevailing conditions and circumstances in the city.

5.1. Choice of Sewerage System

Separate System is considered to serve the city better than a combined system due to the following reasons:-

(1) Sewers in combined system are usually of larger sizes than those in separate system, therefore small depths and low velocities of dry-weather flow result in solids deposition and anearobic conditions in the

sewers. The former results in sewers clogging while the later is responsible for foul smell and crown corrosion of sewers due to generation of H_2S gas. This H_2S gas in the presence of moisture at the crown is converted to H_2SO_4 which reacts with Ca of the concrete forming a soluble $CaSO_4$ compound, leaving behind cavities in the sewers at the crown[12]. Likewise is not the case with the separate system due to smaller sizes of the pipes and hence result in lesser maintenance and operational cost.

(2) Storm water is not offensive and therefore can be disposed off directly on the adjoining land for irrigation purposes, thus resulting saving in construction, operation and maintenance cost of the treatment plant.

(3) The annual rainfall over Bannu is small and therefore it would not be advisable to consider it at this stage for either of the systems due to the existence of proper naturally draining surface drains, leading to the nearby agricultural land for irrigation purposes.

(4) It has been learnt that the existing surface drains have been functioning so far very smoothly in disposing of storm waters and no damage what soever has been reported. Therefore likewise service from them is expected in future, however if storm water is to be accounted for in future due to one or the other reason then it would be easier to finance the two systems separately because of less finances required at one time.

5.2. Sewer Pipes

Various factors enter into consideration for the selection of pipe material. They are summarised as under:-

(1) Availability of the material.

(2) Smoothness or roughness of surface.

(3) Resistance to scour.

(4) Resistance to acids, alkalis, gases, solvents etc.

(5) Life expectancy (Durability).

(6) Ease of handling and installations.

(7) Strength to resist structural failure.

(8) Availability and ease of installation of fittings and connections.

(9) Type of joint-water tightness and ease of assembly.

(10) Availability in sizes required.

(11) Cost of material, handling and installation.

It is quite impossible for a single material to embody all the aforementioned qualities. It is therefore necessary to consider several materials and then select one among them which fits best to the requirements.

The most commonly used materials in sewers construction are as under:-

(1) Cast iron pipes.

(2) Vitrified clay pipes.

(3) Bituminized-fiber pipes.

(4) Asbestos cement pipes.

(5) Concrete pipes.

(1) Cast Iron Pipes

Cast iron pipes are used for buried sewers where external loads or absolute water tightness or both inhibit the use of other materials. They are used in situation where movements and vibration of earth may occur or in situations where the sewers are subjected to alternate thawing and freezing and internal pressure like that in pumping stations.

These pipes are not manufactured locally and would therefore be imported, in case they are employed, thus resulting in higher cost. They are also subject to corrosion. Their use in this report is, however, confined to the pressure sewers instead of the ordinary sewers due to the following reasons.

(a) Water tightness.

(b) High resistance to external and internal pressures.

(c) Availability indifferent diameters and lengths.

(2) Vitrified Clay Pipes

Vitrified clay pipes meet most of the requirements of a ideal material under all conditions, except strength, wight, availability and cost all of which depend on local conditions. Experience has shown its durability, resistance to corrosion and errosion, wide availability and general satisfactory performance. The disadvantages of these pipes are:-

1) Difficult in handling large sizes on account of heavy weights.

2) Liable to damage due to brittleness in handling during transportations.

As these pipes are not manufactured locally and hence cannot be recommended in this report.

(3) Bituminized-fiber pipes

These pipes are available in sizes from 3 to 8 inches[13] in diameters. The advantages claimed for these pipes are:

(a) Lightness

(b) Ease of Construction

(c) Water tightness

(d) Durability

(e) Highly crushing strength

(f) Resistance to corrosion and erosion

(g) Flexibility.

These, inspite of being ideal, cannot be recommended for use in this report on account of the following:

1) Non availability in local markets.

2) Availability of the pipes in small sizes.

(4) Asbestos Cement Pipes

These pipes are available in several classes and wide range of sizes. According to manufacturers' claim its crushing strength is comparable with other types of pipe. It has the following advantages: a lower coefficient of roughness thus allowing flatter grades and smaller pipe sizes; tight joints, using sleeves and rubber rings as for water, which reduce infilteration and do not allow root penetration; long lengths, which reduces the number of joints; light weights which makes installation easier. All necessary specials and fittings are available. However investigations have shown that better velocities in these pipes are due to fewer joints rather than smooth surfaces. They are subject to crown corrosion. Asbestos cement pipes are used where flate topography or high-water table, or both, justify their use.

(5) Cement Concrete Pipes

Though asbestos cement pipes can be used in this work yet cement concrete pipes are considered more suitable due to the following reasons:-

(a) It has high strength and can resist heavy overburdens even in absence of concrete cradles.

(b) Availability in different sizes.

(c) It has got self healing property due to which minor cracks are[13] healed automatically when put into service.

(d) Several type of joints are used. The type to be adopted depends upon the extent of water tightness desired.

(e) Specials and fittings are obtainable with great ease.

(f) The possibility of corrosion can be avoided by maintaining reasonable velocities in the sewers so as not to cause the sewage to become septic or the sewers clogged.

(g) The possibility of using sulphate resistance cement or the application of coating can prevent the corrosion of the pipes.

(h) Scouring and erosion of the pipe material from inner surfaces are also not expected because the velocities in the sewers will not be allowed to exceed the maximum permissible limits.

(i) Concrete pipes are manufactured locally on large scale, therefore they can be had with much ease and relatively less cost.

(j) Topography and low water table both are favorable.

5.3. Design Criteria

Sewers are to receive the liquid wastes and to transport them from one point to the other in such a manner as not to cause any inconvenience to the public. They must be deep enough so as to receive the liquid wastes from their points of origin in smoothly running manners without causing any back building of sewage towards those points or clogging of the house connections. Their sizes and slopes must be adequate for both minimum and peak flows. Therefore it is necessary to establish certain permissible limits upon which to base the design of the sewers.

5.3.1 Maximum and Minimum Flow of Sewage

Liquid wastes from the various parts of the city are not contributed at equal rates throughout day and night, therefore estimation of the minimum,average and the maximum rates of flow is desirable.

As the quantity of water supplied to a certain community appears in the form of dry weather flow after it has been used, therefore the dry weather flow is more or less similar in quantity to the minimum, average, and maximum rate of consumption not only with the time of the day but with the day of the week and the season of the year with the extreme flow occuring between 2 and 6 a.m. in winter, and peak flow occuring during day light hours in summer.

Since no record of variation in water consumption for the city in question is available therefore it is impossible to establish any definite relation from the water consumption in arriving at the variation in dry-weather flow. Moreover neither it is easy nor practicable to have a fixed flow at a fixed time. It is therefore advisable to adopt certain standards for having maximum and minimum flow requirements in sewers.

In absence of pertinent data, and for such places where water consumption is 100 g.p.c.d., following are the minimum acceptable design flow rates[14].

(1) For laterals 400 g.p.c.d.

(2) For trunk sewers 250 g.p.c.d.

The smaller design flow rate for the trunk can be reasoned that the peak is not attained in all the sections of the town due to differences in the habits of the people residing in the different sections. However, in case of the city under consideration the ratios will serve the purpose rather than the actual values because the water consumption is less than 100 g.p.c.d.

The criteria is as under:-

(1) Peak flow in laterals = 4 x Average daily flow

(2) Peak flow in main sewers = 2.5 x Average daily flow

(3) Minimum flow in all sewers = 0.5 x Average daily flow

5.3.2 Velocities of Flows

a) Minimum Velocity

An important consideration in sewers design is the velocity obtained in sewers. The minimum acceptable limit for velocity is 2 ft/sec when the sewer is flowing full and the liquid will not be allowed to leave behind its suspended load content, resulting in clogging of the sewers. However, small flows particularly in the innitial stages of the system will not be able to make the sewers to run full or half full. Therefore small depths and lower velocities of flow would prevail in the sewers resulting in undesirable conditions. In such cases a higher velocity of 3 to 3.5 ft/sec is usually adopted[14], but as topography has much to do with the maintenance of reasonable velocities in pipes, therefore it is not economical to maintain a velocity higher than 2.5[3] ft/sec at the expense of deep excavations and pumping. So a velocity of 2.5 ft/sec is to be used in this report.

b) Maximum Velocity

Maximum permissible velocity is of as much concern as is the minimum permissible velocity because the former is limited from the tractive force point of view which results in erosion of the pipes while the later results in deposition of solids in the pipe if allowed to fall below the minimum permissible limit. In order to avoid erosion of the sewers invert, an effort is generally made to keep velocities below 8 ft/sec[5] for concrete sewers. Therefore this maximum limit of 8 ft/sec will be observed strictly in this report for all concrete sewers.

5.3.3 Slopes

a) Minimum Slopes

Minimum desirable slopes are those for which generally very little excavation is involved, following the natural topography of the ground and at the same time keeping the velocity within the permissible limits. However for this report the minimum slope will be that which will maintain a velocity not less than 2.5 ft/sec when the pipe is flowing full.

b) Maximum Slopes

The maximum slopes limits are arrived at from the consideration of the maximum permissible velocities. The maximum velocity limit has been fixed at 8 ft/sec, so the maximum permissible slope is corresponding to this maximum permissible velocity.

5.3.4 Shape of Sewers

Pipes with circular sections are generally used in all works and would be adopted in this design also on account of the following reasons:

(1) They give maximum cross-sectional area for equal quantity of material in the wall.

(2) They possess excellent hydraulic properties.

(3) They offer great convenience in manufacturing.

5.3.5 Depth of Sewers

Depths of sewers are fixed from the buildings outlets consideration. The depths of sewers must be such as to provide adequate slopes in the buildings connecting pipes in order to maintain a reasonable velocity so as to avoid clogging of these pipes. Since the buildings in the city are without basements, therefore a depth of 4 ft (thickness of cover) is being adopted in the report.

5.3.6 Minimum Size of Sewers

A minimum size of 8 inches[5] is adopted due to the following reasons:-

(1) Cost of cleaning of the smaller sizes of sewers is more than the saving by providing smaller dia. sewers.

(2) Smaller sewers clog more frequently.

(3) This size can serve for a longer period

Summary of Design Criteria

The following design criteria is adopted in this report.

(1)	Maximum flow in laterals	=	4 x Average daily flow
(2)	Maximum flow in submains	=	4 x Average daily flow
(3)	Maximum flow in mains and trunks	=	2.5 x Average daily flow
(4)	Minimum flow in all sewers	=	0.5 x Average daily flow
(5)	Minimum size of sewers	=	8 inches
(6)	Maximum velocity	=	8 ft/sec.
(7)	Minimum velocity flowing full	=	2.5 ft/sec.
(8)	Maximum slope	=	According to topography and to keep the max. velocity as in item 7.

(9) Minimum slope = Such as not to allow the velocity to fall below the limit of item 8.

(10) Shape of sewers = Circular.

(11) Depth from ground surface to the tops of sewers = 4 feet.

The following criteria will also be observed in addition to the above:

(12) Manholes will be provided at all changes in alignments, sizes, grades and at all junctions and intermediate points and at spacing preferably not more than 300 ft. for providing facility in the cleaning.

(13) The inverts of mains at junction shall be above those of trunks by at least half their difference in diameter of the small pipe and 3/4th difference in diameter of the large pipes or the tops of the pipes shall be at the same elevation.

5.4. Manholes

A variety of sewer appurtenances are required for proper operation and maintenance of sewers, however most of common and numerous are the manholes. These are used for inspection and cleaning of sewers.

5.4.1 Spacing

Manholes are generally placed at interval of 300 to 500 ft[5], and at points where there is a change in direction, change in pipe size or considerable change in grade. Large sewers, 60 in or more in diameter can be entered for inspection and therefore need very few manholes. A spacing interval of 300 ft. between two consecutive manholes will be maintained in this report.

5.4.2 Shape

Manholes are generally provided with cast iron cover which fits into a cast iron frame with clear opening varies between 20 to 24 inches diameter. The frame rests on a corbelled portion which ends 3 to 5 ft below the top. Manholes are usually circular in shape with clear inside diameter as 4 ft.[5] The cover must rest tightly against the frame and should be strong enough to resist traffic loads.

5.4.3 Material

Manholes are constructed either of bricks with cement mortar or cement concrete, however the choice of material used depends upon several factors like:-

(1) Labour, material and equipment cost.

(2) Durability.

(3) Adaptability of the material to field conditions.

(4) Depth of manhole and character of surrounding materials.

After careful consideration of the above factors, it is concluded that cement concrete will be costlier as compared to bricks with cement mortars on account of its low material, labour and equipment cost. Moreover in case of bricks no form work is used. Therefore greater progress in work is obtained. Experience has shown that this material can suit the local conditions and therefore will render service in a good manner for at least 50 years. Therefore bricks with cement mortar will be used in the walls while concrete will be used for the bottom of the manholes.

5.4.4 Wall Thickness

Wall thickness of a manhole depends upon its depth. Wall thickness of 8 inches[5] is generally adopted upto a depth of 12 ft. in case of cement concrete while in case of bricks the thickness would be one brick if the height of the wall does not exceed 12 ft. For each 4 ft. addition in height ½ brick thickness will be added.

5.4.5 Bottom

Bottom of manholes would be of concrete about 8 inch in thickness in order to support the walls and to facilitate the construction of U-shaped channels in it. The sides of the channels will be kept high enough to prevent the flow of sewage onto the sloping floor. Smooth curves will be provided in the bottom of the manhole if it serves as a junction for several sewers. Tops of the pipes should be placed at the same elevation in order to avoid back flows in smaller pipes when the larger pipes are flowing full.

5.5. Excavation and Embedment

Several investigations have shown that the load caused by earth filling upon pipes in trench is carried entirely by the top 90° degree sector while the foundation reaction is likewise taken by the bottom quadrant. However if proper shaping of the bottom is not done then concentration of reaction at a point may result in pipe failure. Therefore proper excavation for and laying of sewers are important.

5.5.1 Excavation

The width of the trench where 90° Sector of the pipe ends, or just below the top of the pipe, is the width factor that effects the load on the pipes. If the ditch is wider below this point, friction between the settling earth and undisturbed earth of the sides of the trench does not operate to reduce the load materially on the pipe. However trench width should be kept as minimum as just to allow good workmanship in laying and jointing of pipes. One rule for trench width is 3/2d + 12 in where is the internal diameter of pipe in inches.

5.5.2 Embedment

The method of laying has much to do with the strength developed by the pipes. Figure 5.1 shows several methods of bedding the sewer pipes. Type 1 is the most frequently used one but is very much poor in imarting strength to the pipes. Type 2 has wide recommendations but is considered to be uneconomical on account of the high labours cost involved in shaping the trench. However in our case this method is recommended because it will not be costly on account of comparatively cheaper labor availability. This method of laying, therefore, will be adhered to for all sewers except those which are subject to heavy traffic loads. In the later case Type 3 or 4 would be adopted due to higher load factors.

Type 5 involves partial concrete imbedment while in type 6 the pipe is fully imbeded in concrete. Both of the type being expensive would not be considered in this report.

5.6. Sewerage Plan

The plan of a sewerage system is governed by two prime factors, the topography of the city and the place of disposal of the sewage.

The arrangement of small and large sewers which makes up a sewer system is influenced largely by the topography of the city. In large cities with flate topography without any neighbouring rivers or lakes into which the sewage can be discharged, the radial system may prove that best. But in most cases such an arrangement is rendered impracticable by the configuration of the ground and the idea of utilizing the existing old sewers which in large cities is not unusual.

However in case of Bannu city, the absence of old sewers makes the matter easier for radial system but the

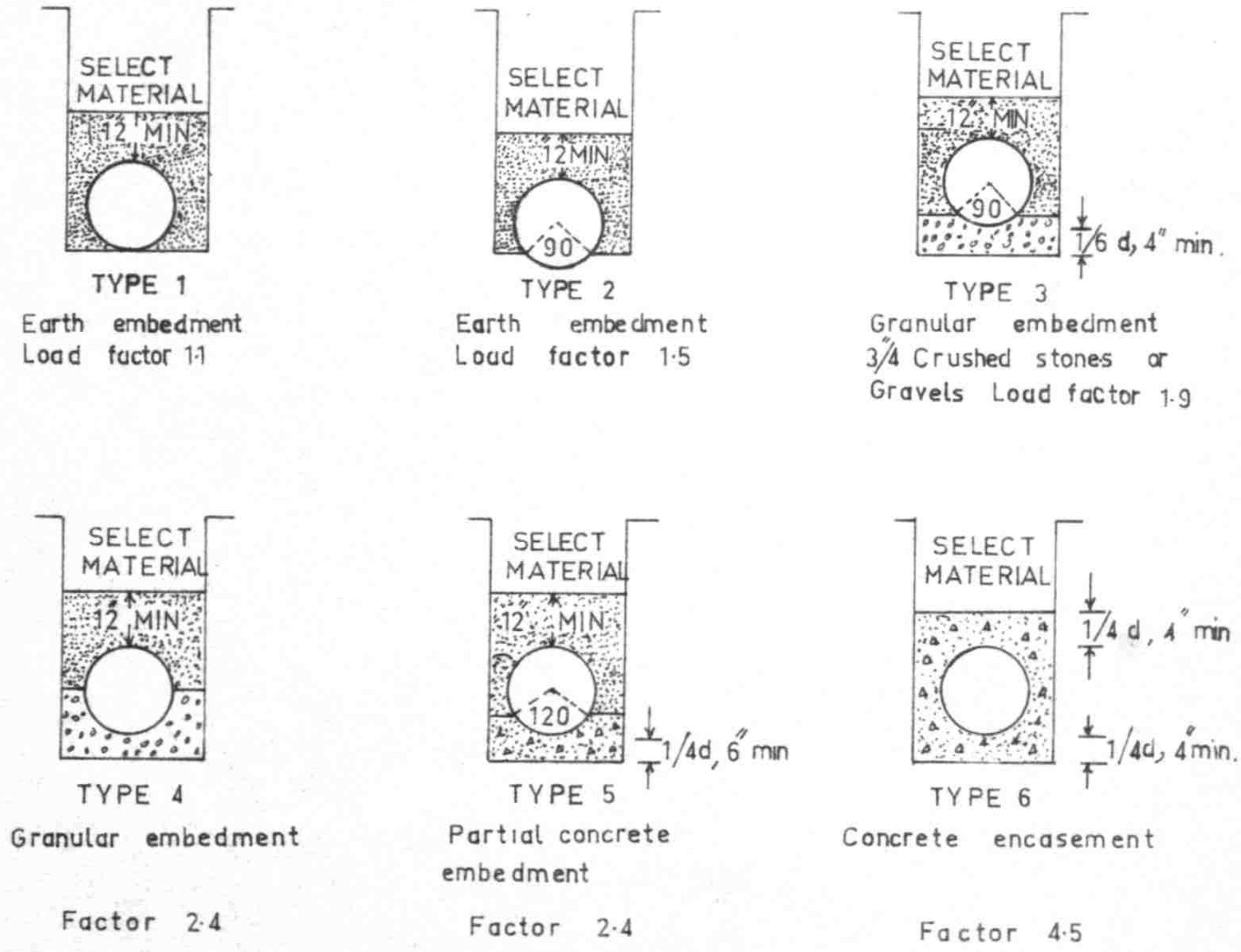


FIG 5·1 SEWER EMBEDMENT

configuration of the ground coupled with the idea of the treatment of the sewage inhibit the use of this plan and therefore the plan of Drawing No. 3. would be adopted in order to facilitate the collection of sewage of the whole city to a single point for the purpose of treatment.

The city would be provided with seven mains, intercepted by the trunk running in the North-south direction as shown in Drawing No. 3.

The whole of the sewage flow would be by gravity except that to the treatment plant where lifting of the sewage may require pumping

5.7. Design of Sewers

5.7.1 Design of Storm Water Sewers

The design of storm water sewers would not be taken in this report on account of naturally drainage property of the city due to its favorable topography. However the distribution of the city into the catchment areas as per Drawing No. 3. does hold good for the design of storm water sewers as well if and when needed though not expected in light of the above reason and due to the considerations discussed in section 4.4 of this report.

5.7.2 Design of Sanitary Sewers

Lines have been drawn to represent the main sewers in the streets. The connections of the sewers to the point of disposal by the shortest possible routes, being more economical, have been dually considered.

The whole city has been divided into seven catchment areas thus fixing the limits of tributary areas for each main. The catchment areas have been designated from A through G.

Area A would be served by main sewer A which joins the trunk through Manhole No. 21. Area A is in the northern part of the city and at present it is sparsely populated.

Area B is towards east of A and is comparatively densely populated. This area would be served by main sewer B which also joins the trunk through manhole No. 21.

Catchment area C served by main sewer C has the same population density as B, both for present and future. The sewer of this area passes near to the industrial area and therefore carries some of the industrial liquid wastes too.

Catchment area D served by the main sewer D is composed of mainly residential and commercial areas. This sewer joins the trunk through the Manhole No. 17.

Catchment area G served by main sewer G is towards east of the city and is adjacent to the catchment area C and includes mostly of the industrial area, therefore all the industrial wastes balance from sewer C are carried by this sewer. The sewer joins the trunk through the Manhole No. 10.

Similarly catchment areas E and F are served by main sewers E and F which join the trunk through the manholes No. 14 and 10 respectively.

The amount of waste waters from the various catchment areas are given in table 5.1 while the design of sewers is presented in table 5.2 and 5.3.

As the use of formulas for the design of the sewers would be cubersome, therefore it would be advisable to resort to the nomogram with the value of n as 0.013.

The minimum depths of flow should not be less than 2 inches and the velocity not less than 1.5 ft/sec, as obtained from the partial flow curves.( )

TABLE 5.1

AREA, POPULATION AND AMOUNT OF WASTE WATER

Catchment Area	Area (acres)	Future Population	Sewage in gal/day	Industrial Waste gal/day	Total Waste in gal/day
A	103.49	5,133	230,000	-	230,000
B	32.00	2,043	92,000	-	92,000
C	47.00	3,011	135,000	225,000	360,000
D	196.14	11,207	506,000	-	506,000
E	182.23	11,646	525,000	-	525,000
F	114.33	7,365	332,000	-	332,000
G	78.81	3,595	160,000	103,796	263,796
Total	754.00	44,000	-	-	2,308,796

TABLE 5.2

FLOW IN SEWERS

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
A	A-MH 10	A-MH 9	300	9.29	475	475	21,400	0.0332
	A-MH 9	A-MH 8	300	11.30	520	995	44,800	0.0695
	A-MH 8	A-MH 7	300	14.35	721	1,716	77,000	0.1180
	A-MH 7	A-MH 6	300	14.35	721	2,437	110,000	0.1700
	A-MH 6	A-MH 5	300	13.80	690	3,127	140,000	0.2180
	A-MH 5	A-MH 4	300	13.80	690	3,817	172,000	0.2670
	A-MH 4	A-MH 3	300	13.30	658	4,475	200,000	0.3100
	A-MH 3	A-MH 2	300	13.30	658	5,133	230,000	0.3570
	A-MH 2	A-MH 1	280	-	-	5,133	230,000	0.3570
	A-MH 1	T-MH 21	280	-	-	5,133	230,000	0.3570

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
B	B-MH 5	B-MH 4	300	10.80	700	700	31,500	0.0480
	B-MH 4	B-MH 3	300	8.20	535	1,235	55,600	0.0860
	B-MH 3	B-MH 2	300	7.20	457	1,692	76,400	0.1180
	B-MH 2	B-MH 1	300	3.60	234	1,926	86,500	0.1340
	B-MH 1	Trunk	300	1.80	117	2,043	92,000	0.1430
C	C-MH 6	C-MH 5	300	16.00	1,040	1,040	47,000	0.0730
	C-MH 5	C-MH 4	300	8.85	575	1,615	72,700	0.1120
	C-MH 4	C-MH 3	300	7.50	449	2,064	93,000	0.1440
	C-MH 3	C-MH 2	300	6.55	425	2,489	112,000	0.1860
	C-MH 2	C-MH 1	300	5.30	340	2,829	353,000	0.5470
	C-MH 1	Trunk	150	2.80	182	3,011	360,000	0.5570

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
D	D-MH 30	D-MH 29	300	32.50	1,460	1,450	65,400	0.1000
	D-MH 29	D-MH 28	300	13.80	620	2,070	93,000	0.1440
	D-MH 28	D-MH 27	300	12.70	570	2,640	118,000	0.1830
	D-MH 27	D-MH 26	300	10.50	475	3,115	140,000	0.2170
	D-MH 26	D-MH 25	225	9.30	420	3,535	159,000	0.2480
	D-MH 25	D-MH 24	300	12.20	550	4,085	181,000	0.2800
	D-MH 24	D-MH 23	225	9.00	585	4,670	210,000	0.3260
	D-MH 23	D-MH 22	190	6.00	390	5,060	228,000	0.3540
	D-MH 22	D-MH 21	190	5.00	325	5,385	242,000	0.3760
	D-MH 21	D-MH 20	300	6.00	390	5,775	259,000	0.4000
	D-MH 20	D-MH 19	225	2.60	130	5,905	266,000	0.4130
	D-MH 19	D-MH 18	75	2.50	162	6,067	272,000	0.4230
	D-MH 18	D-MH 17	300	1.00	65	6,132	276,000	0.4280
	D-MH 17	D-MH 16	150	1.00	65	6,197	279,000	0.4310
	D-MH 16	D-MH 15	225	3.50	228	6,425	290,000	0.4500

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Popula-tion	Total Popula-tion	Sewage in Flow gal/day	Sewage flow (cfs)
	D-MH 15	D-MH 14	225	3.52	229	6,654	300,000	0.4650
	D-MH 14	D-MH 13	300	4.05	284	6,938	312,000	0.4840
	D-MH 13	D-MH 12	240	4.00	280	7,218	322,000	0.5000
	D-MH 12	D-MH 11	205	4.06	285	7,503	339,000	0.5250
	D-MH 11	D-MH 10	205	4.08	286	7,789	350,000	0.5430
	D-MH 10	D-MH 9	300	6.50	455	8,244	370,000	0.5750
	D-MH 9	D-MH 8	200	4.00	280	8,524	384,000	0.5950
	D-MH 8	D-MH 7	200	4.00	280	8,804	386,000	0.6000
	D-MH 7	D-MH 6	230	5.80	405	9,209	415,000	0.6450
	D-MH 6	D-MH 5	230	5.82	406	9,615	434,500	0.6740
	D-MH 5	D-MH 4	300	3.32	235	9,850	444,000	0.6900
	D-MH 4	D-MH 3	300	6.20	435	10,285	463,000	0.7180
	D-MH 3	D-MH 2	300	5.15	356	10,641	478,000	0.7440
	D-MH 2	D-MH 1	230	3.28	230	10,871	488,000	0.7740
	D-MH 1	T-MH 17	230	4.80	336	11,207	506,000	0.7850

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Popula-tion	Total Popula-tion	Sewage in flow gal/day	Sewage flow (cfs)
E	E-MH 31	E-MH 30	300	33.60	1,609	1,609	72,500	0.1120
	E-MH 30	E-MH 29	300	9.25	600	2,209	90,500	0.1400
	E-MH 29	E-MH 28	300	9.80	630	2,839	127,000	0.1970
	E-MH 28	E-MH 27	240	5.58	363	3,202	144,000	0.2220
	E-MH 27	E-MH 26	85	0.75	48	3,250	146,00	0.2260
	E-MH 26	E-MH 25	150	1.00	65	3,315	149,000	0.2310
	E-MH 25	E-MH 24	250	0.80	62	3,377	153,000	0.2370
	E-MH 24	E-MH 23	300	12.00	780	4,157	187,000	0.2900
	E-MH 23	E-MH 22	300	17.80	1,150	5,307	238,000	0.3690
	E-MH 22	E-MH 21	150	5.00	325	5,632	253,000	0.3920
	E-MH 21	E-MH 20	150	4.80	310	5,942	267,000	0.4150
	E-MH 20	E-MH 19	225	2.00	130	6,072	272,000	0.4220
	E-MH 19	E-MH 18	300	1.80	117	6,189	278,000	0.4320
	E-MH 18	E-MH 17	300	1.00	70	6,259	282,000	0.4370

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
	E-MH 17	E-MH 16	185	3.80	266	6,525	293,000	0.4550
	E-MH 16	E-MH 15	260	4.40	308	6,833	207,000	0.4760
	E-MH 15	E-MH 14	150	3.20	224	7,057	318,000	0.4830
	E-MH 14	E-MH 13	100	2.30	161	7,218	324,000	0.5020
	E-MH 13	E-MH 12	300	6.65	465	7,683	346,000	0.5360
	E-MH 12	E-MH 11	150	3.20	224	7,907	356,000	0.5520
	E-MH 11	E-MH 10	160	3.40	238	8,145	366,200	0.5670
	E-MH 10	E-MH 9	150	3.20	224	8,369	375,800	0.5820
	E-MH 9	E-MH 8	300	6.65	465	8,834	397,000	0.6150
	E-MH 8	E-MH 7	300	6.60	460	9,294	417,000	0.6370
	E-MH 7	E-MH 6	185	4.00	280	9,574	430,000	0.6670
	E-MH 6	E-MH 5	185	4.00	280	9,854	444,000	0.6880
	E-MH 5	E-MH 4	185	4.06	281	10,135	450,000	0.6980
	E-MH 4	E-MH 3	185	4.06	284	10,419	467,000	0.7240

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Popula-tion	Total Popula-tion	Sewage in flow gal/day	Sewage flow (cfs)
	E-MH 3	E-MH 2	255	3.32	232	10,651	478,000	0.7400
	E-MH 2	E-MH 1	300	6.71	470	11,121	500,000	0.7750
	E-MH 1	T-MH 14	300	7.50	525	11,646	525,000	0.8120
F	F-MH 20	F-MH 19	150	9.20	580	580	26,000	0.0403
	F-MH 19	F-MH 18	188	5.10	520	900	40,500	0.0627
	F-MH 18	F-MH 17	188	5.10	320	1,220	55,000	0.0850
	F-MH 17	F-MH 16	280	7.00	435	1,655	74,500	0.1145
	F-MH 16	F-MH 15	300	7.15	475	2,120	95,200	0.1470
	F-MH 15	F-MH 14	300	6.35	420	2,540	116,000	0.1800
	F-MH 14	F-MH 13	300	6.00	400	2,940	134,100	0.2080
	F-MH 13	F-MH 12	240	4.41	310	3,250	144,000	0.2240
	F-MH 12	F-MH 11	240	4.42	310	3,560	160,000	0.2480
	F-MH 11	F-MH 10	250	4.94	322	3,882	174,000	0.2700
	F-MH 10	F-MH 9	250	4.90	320	4,202	190,000	0.2950

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
	F-MH 9	F-MH 8	300	5.98	400	4,602	208,000	0.3220
	F-MH 8	F-MH 7	300	6.00	402	5,004	225,000	0.3480
	F-MH 7	F-MH 6	300	6.00	402	5,406	246,000	0.3820
	F-MH 6	F-MH 5	250	5.34	284	5,690	256,000	0.3970
	F-MH 5	F-MH 4	250	5.35	285	5,975	269,000	0.4170
	F-MH 4	F-MH 3	300	6.30	420	6,395	288,000	0.4460
	F-MH 3	F-MH 2	280	5.25	350	6,745	315,000	0.4870
	F-MH 2	F-MH 1	225	4.77	310	7,055	317,200	0.4920
	F-MH 1	T-MH 10	225	4.77	310	7,365	332,000	0.5150
G	G-MH 8	G-MH 7	300	23.20	1,000	1,000	148,796	0.2320
	G-MH 7	G-MH 6	300	7.70	346	1,346	169,496	0.2630
	G-MH 6	G-MH 5	300	5.42	245	1,591	175,396	0.2720
	G-MH 5	G-MH 4	200	5.08	228	1,819	185,796	0.2880

LINE	From Manhole	To Manhole	Length (ft)	Increament of Area (acres)	Increament of Population	Total Population	Sewage in flow gal/day	Sewage flow (cfs)
	G-MH 4	G-MH 3	200	7.25	326	2,145	200,296	0.3100
	G-MH 3	G-MH 2	300	10.00	450	2,595	220,296	0.3260
	G-MH 2	G-MH 1	300	10.08	450	3,045	240,496	0.3720
	G-MH 1	T-MH 10	300	10.08	450	3,495	263,796	0.4070

TABLE 5.3

DESIGN OF SANITARY SEWERS

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
A	A-MH 10	A-MH 9	300	0.0332	0.0830	1303.00	1301.00	8	0.0067	2.85	0.950	1298.25	1296.25
	A-MH 9	A-MH 8	300	0.0695	0.1740	1301.00	1299.00	8	0.0067	2.85	0.950	1296.25	1294.25
	A-MH 8	A-MH 7	300	0.1180	0.2960	1299.00	1297.50	8	0.0050	2.50	0.875	1294.25	1292.75
	A-MH 7	A-MH 6	300	0.1700	0.4250	1297.50	1296.50	8	0.0050	2.50	0.875	1292.75	1291.25
	A-MH 6	A-MH 5	300	0.2180	0.5460	1296.50	1295.80	8	0.0050	2.50	0.875	1291.25	1289.75
	A-MH 5	A-MH 4	300	0.2670	0.6670	1295.80	1295.00	8	0.0050	2.50	0.875	1289.75	1288.25
	A-MH 4	A-MH 3	300	0.3100	0.6820	1295.00	1294.20	8	0.0050	2.50	0.875	1288.25	1286.75
	A-MH 3	A-MH 2	300	0.3570	0.8930	1294.20	1291.50	10	0.0038	2.50	1.350	1286.58	1285.44
	A-MH 2	A-MH 1	280	0.3570	0.8930	1291.50	1290.00	10	0.0038	2.50	1.350	1285.44	1284.38
	A-MH 1	T-MH 21	280	0.357	0.8930	1290.00	1288.50	10	0.0038	2.50	1.350	1284.38	1283.32

LINE	From Man-	To Man-	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flow-ing full ft/sec	Cap. flow-ing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manholw
B	B-MH 5	B-MH 4	300	0.0480	0.1200	1291.00	1290.40	8	0.005	2.50	0.875	1286.25	1284.75
	B-MH 4	B-MH 3	300	0.0860	0.2150	1290.40	1288.40	8	0.005	2.50	0.875	1284.75	1283.25
	B-MH 3	B-MH 2	300	0.1180	0.2950	1288.40	1288.80	8	0.005	2.50	0.875	1283.25	1281.75
	B-MH 2	B-MH 1	300	0.1340	0.3350	1288.80	1288.90	8	0.005	2.50	0.875	1281.75	1280.25
	B-MH 1	T-MH 21	300	0.1430	0.357	1288.90	1288.50	8	0.005	2.50	0.875	1280.25	1278.75
C	C-MH 6	C-MH 5	300	0.073	0.1830	1285.00	1285.20	8	0.005	2.50	0.875	1280.25	1278.75
	C-MH 5	C-MH 4	300	0.112	0.280	1285.20	1285.40	8	0.005	2.50	0.875	1278.75	1277.25
	C-MH 4	C-MH 3	300	0.144	0.360	1285.40	1285.60	8	0.015	2.50	0.875	1277.25	1275.75
	C-MH 3	C-MH 2	300	0.186	0.466	1285.60	1286.20	8	0.005	2.50	0.875	1275.75	1274.25
	C-MH 2	C-MH 1	300	0.547	1.370	1286.20	1286.40	10	0.004	2.60	1.400	1274.25	1273.05
	C-MH 1	T-MH 18	150	0.550	1.380	1286.40	1286.20	10	0.004	2.60	1.400	1273.05	1272.45

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
D	D-MH 30	D-MH 29	300	0.100	0.250	1305.00	1303.80	8	0.005	2.50	0.875	1300.25	1298.75
	D-MH 29	D-MH 28	300	0.144	0.360	1303.80	1302.20	8	0.005	2.50	0.875	1298.75	1297.25
	D-MH 28	D-MH 27	300	0.183	0.447	1302.20	1301.00	8	0.005	2.50	0.875	1297.25	1295.75
	D-MH 27	D-MH 26	300	0.217	0.542	1301.00	1300.20	8	0.005	2.50	0.875	1295.75	1294.25
	D-MH 26	D-MH 25	225	0.248	0.620	1300.20	1299.00	8	0.005	2.50	0.875	1294.25	1293.13
	D-MH 25	D-MH 24	300	0.280	0.700	1299.00	1298.00	8	0.005	2.50	0.875	1293.13	1291.63
	D-MH 24	D-MH 23	225	0.326	0.815	1298.00	1298.00	8	0.005	2.50	0.875	1291.63	1290.50
	D-MH 23	D-MH 22	190	0.354	0.885	1298.00	1298.00	10	0.0038	2.50	1.350	1290.33	1289.61
	D-MH 22	D-MH 21	190	0.376	0.940	1298.00	1298.20	10	0.0038	2.50	1.350	1289.61	1288.88
	D-MH 21	D-MH 20	300	0.400	1.000	1298.20	1298.20	10	0.0038	2.50	1.350	1288.88	1287.74
	D-MH 20	D-MH 19	225	0.413	1.030	1298.20	1298.40	10	0.0038	2.50	1.350	1287.74	1286.89
	D-MH 19	D-MH 18	75	0.423	1.060	1298.40	1299.00	10	0.0038	2.50	1.350	1286.89	1286.61
	D-MH 18	D-MH 17	300	0.428	1.070	1299.00	1300.00	10	0.0038	2.50	1.350	1286.61	1285.47
	D-MH 17	D-MH 16	150	0.431	1.080	1300.00	1299.30	10	0.0038	2.50	1.350	1285.47	1284.90

LINE From Man-hole	To man-hole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flow-ing full ft/sec	Cap. flow-ing full (cfs)	Invert Elevation	
					Upper manhole	Lower manhole					Upper manhole	Lower manhole
D-MH 16	D-MH 15	225	0.450	1.120	1299.30	1298.50	10	0.0038	2.50	1.350	1284.90	1284.05
D-MH 15	D-MH 14	225	0.465	1.160	1298.50	1298.00	10	0.0038	2.50	1.350	1284.05	1283.20
D-MH 14	D-MH 13	300	0.484	1.210	1298.00	1297.00	10	0.0038	2.50	1.350	1283.20	1282.16
D-MH 13	D-MH 12	240	0.500	1.250	1297.00	1296.00	10	0.0038	2.50	1.350	1282.16	1281.25
D-MH 12	D-MH 11	205	0.525	1.310	1296.00	1295.50	10	0.0038	2.50	1.350	1281.25	1280.47
D-MH 11	D-MH 10	205	0.543	1.330	1295.50	1294.80	10	0.0038	2.50	1.350	1280.30	1279.52
D-MH 10	D-MH 9	300	0.575	1.440	1294.80	1293.80	12	0.0030	2.50	1.800	1279.52	1278.62
D-MH 9	D-MH 8	200	0.595	1.490	1293.80	1292.80	12	0.0030	2.50	1.800	1278.62	1278.02
D-MH 8	D-MH 7	200	0.600	1.500	1292.80	1291.80	12	0.0030	2.50	1.800	1278.02	1277.42
D-MH 7	D-MH 6	230	0.645	1.610	1291.80	1291.50	12	0.0030	2.50	1.800	1277.42	1226.73
D-MH 6	D-MH 5	230	0.674	1.680	1291.50	1291.00	12	0.0030	2.50	1.800	1276.73	1276.04
D-MH 5	D-MH 4	300	0.690	1.720	1291.00	1290.50	12	0.0030	2.50	1.800	1276.04	1275.14
D-MH 4	D-MH 3	300	0.718	1.800	1290.50	1289.00	12	0.0030	2.50	1.800	1275.14	1274.24

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft) Upper manhole	Ground Elevation (ft) Lower manhole	Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation Upper manhole	Invert Elevation Lower manhole
	D-MH 3	D-MH 2	300	0.744	1.850	1289.00	1288.00	15	0.0024	2.50	3.000	1273.99	1273.27
	D-MH 2	D-MH 1	230	0.774	1.930	1288.00	1287.00	15	0.0024	2.50	3.000	1273.27	1272.72
	D-MH 1	D-MH 17	230	0.785	1.960	1287.00	1286.00	15	0.0024	2.50	3.000	1272.72	1272.17
E-	E-MH 31	E-MH 30	300	0.112	0.270	1304.00	1303.40	8	0.0050	2.50	0.875	1301.00	1299.50
	E-MH 30	E-MH 29	300	0.140	0.350	1303.40	1303.20	8	0.0050	2.50	0.875	1299.50	1298.00
	E-MH 29	E-MH 28	300	0.197	0.492	1303.20	1303.60	8	0.0050	2.50	0.875	1298.00	1296.50
	E-MH 28	E-MH 27	240	0.222	0.555	1303.60	1303.00	8	0.0050	2.50	0.875	1296.50	1295.30
	E-MH 27	E-MH 26	85	0.226	0.565	1303.00	1302.00	8	0.0050	2.50	0.875	1295.30	1294.90
	E-MH 26	E-MH 25	150	0.231	0.577	1302.00	1301.00	8	0.0050	2.50	0.875	1294.90	1294.16
	E-MH 25	E-MH 24	250	0.237	0.592	1301.00	1301.40	8	0.0050	2.50	0.875	1294.16	1293.96
	E-MH 24	E-MH 23	300	0.290	0.725	1301.40	1301.40	8	0.0050	2.50	0.875	1293.96	1292.46
	E-MH 23	E-MH 22	300	0.369	0.922	1301.40	1300.50	10	0.0038	2.50	1.350	1292.29	1291.15
	E-MH 22	E-MH 21	150	0.392	0.980	1301.00	1301.00	10	0.0038	2.50	1.350	1291.15	1290.58

LINE From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft) Upper manhole	Ground Elevation (ft) Lower manhole	Dia. (inch)	Grade of Sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation Upper manhole	Invert Elevation Lower manhole
E-MH 21	E-MH 20	150	0.415	1.040	1300.00	1300.00	10	0.0038	2.50	1.350	1290.58	1290.01
E-MH 20	E-MH 19	225	0.422	1.054	1300.00	1299.00	10	0.0038	2.50	1.350	1290.01	1289.16
E-MH 19	E-MH 18	300	0.432	1.070	1299.00	1298.50	10	0.0038	2.50	1.350	1289.16	1288.02
E-MH 18	E-MH 17	300	0.437	1.090	1298.50	1298.00	10	0.0038	2.50	1.350	1288.02	1287.88
E-MH 17	E-MH 16	185	0.455	1.140	1298.00	1298.00	10	0.0038	2.50	1.350	1287.88	1287.18
E-MH 16	E-MH 15	260	0.476	1.190	1298.00	1297.50	10	0.0038	2.50	1.350	1287.18	1286.19
E-MH 15	E-MH 14	150	0.483	1.210	1297.50	1297.00	10	0.0038	2.50	1.350	1286.19	1285.62
E-MH 14	E-MH 13	100	0.502	1.252	1297.00	1296.30	10	0.0038	2.50	1.350	1285.62	1285.24
E-MH 13	E-MH 12	300	0.536	1.340	1296.30	1295.60	10	0.0038	2.50	1.350	1285.24	1284.10
E-MH 12	E-MH 11	150	0.552	1.380	1295.60	1295.30	12	0.0030	2.50	1.800	1283.93	1283.48
E-MH 11	E-MH 10	160	0.567	1.420	1295.30	1294.50	12	0.0030	2.50	1.800	1283.48	1283.00
E-MH 10	E-MH 9	150	0.582	1.460	1294.50	1294.00	12	0.0030	2.50	1.800	1283.00	1282.55
E-MH 9	E-MH 8	300	0.615	1.540	1294.00	1292.00	12	0.0030	2.50	1.800	1282.55	1281.65
E-MH 8	E-MH 7	300	0.637	1.590	1292.00	1291.00	12	0.0030	2.50	1.800	1281.65	1280.75

LINE	From man-hole	To man-hole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flow-ing full ft/sec	Cap. flow-ing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
	E-MH 7	E-MH 6	185	0.667	1.670	1291.00	1290.00	12	0.0030	2.50	1.800	1280.75	1280.20
	E-MH 6	E-MH 5	185	0.688	1.730	1290.00	1289.50	12	0.0030	2.50	1.800	1280.20	1279.65
	E-MH 5	E-MH 4	185	0.698	1.750	1289.50	1289.00	12	0.0030	2.50	1.800	1279.65	1279.10
	E-MH 4	E-MH 3	185	0.724	1.810	1289.00	1288.50	15	0.0024	2.50	3.000	1278.85	1278.40
	E-MH 3	E-MH 2	255	0.740	1.850	1288.50	1287.40	15	0.0024	2.50	3.000	1278.40	1277.78
	E-MH 2	E-MH 1	300	0.775	1.940	1287.40	1286.00	15	0.0024	2.50	3.000	1277.78	1277.06
	E-MH 1	T-MH 14	300	0.812	2.030	1286.00	1284.40	15	0.0024	2.50	3.000	1277.06	1276.34
F	F-MH 20	F-MH 19	150	0.0403	0.101	1303.00	1302.40	8	0.0050	2.50	0.875	1298.25	1297.50
	F-MH 19	F-MH 18	188	0.0627	0.156	1302.40	1301.20	8	0.0050	2.50	0.875	1297.50	1296.56
	F-MH 18	F-MH 17	188	0.0850	0.212	1301.20	1300.60	8	0.0050	2.50	0.875	1296.56	1295.62
	F-MH 17	F-MH 16	280	0.1140	0.285	1300.60	1298.20	8	0.0050	2.50	0.875	1295.62	1294.22
	F-MH 16	F-MH 15	300	0.1470	0.367	1298.20	1297.80	8	0.0050	2.50	0.875	1294.22	1292.72
	F-MH 15	F-MH 14	300	0.1800	0.450	1297.80	1296.00	8	0.0050	2.50	0.875	1292.72	1291.22

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
	F-MH 14	F-MH 13	300	0.208	0.520	1296.00	1295.40	8	0.0050	2.50	0.875	1291.22	1289.72
	F-MH 13	F-MH 12	240	0.224	0.560	1295.40	1294.60	8	0.0050	2.50	0.875	1289.72	1288.52
	F-MH 12	F-MH 11	240	0.248	0.620	1294.60	1294.20	8	0.0050	2.50	0.875	1288.52	1287.32
	F-MH 11	F-MH 10	250	0.270	0.675	1294.20	1293.80	8	0.0050	2.50	0.875	1287.32	1286.07
	F-MH 10	F-MH 9	250	0.295	0.735	1293.80	1293.40	8	0.0050	2.50	0.875	1286.07	1284.82
	F-MH 9	F-MH 8	300	0.322	0.805	1293.40	1293.00	8	0.0050	2.50	0.875	1284.82	1283.32
	F-MH 8	F-MH 7	300	0.348	0.870	1293.00	1292.50	8	0.0050	2.50	0.875	1283.32	1281.82
	F-MH 7	F-MH 6	300	0.382	0.955	1292.50	1291.00	10	0.0038	2.50	1.350	1281.65	1280.51
	F-MH 6	F-MH 5	250	0.397	0.990	1291.00	1290.00	10	0.0038	2.50	1.350	1280.51	1279.56
	F-MH 5	F-MH 4	250	0.417	1.040	1290.00	1288.00	10	0.0038	2.50	1.350	1279.56	1278.61
	F-MH 4	F-MH 3	300	0.446	1.120	1288.00	1286.00	10	0.0038	2.50	1.350	1278.61	1277.47
	F-MH 3	F-MH 2	280	0.487	1.220	1286.00	1284.00	10	0.0038	2.50	1.350	1276.01	1275.05
	F-MH 2	F-MH 1	225	0.492	1.230	1284.00	1282.00	10	0.0038	2.50	1.350	1276.01	1275.05
	F-MH 1	T-MH 10	225	0.515	1.290	1282.00	1281.00	10	0.0038	2.50	1.350	1275.05	1274.09

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
G	G-MH 8	G-MH 7	300	0.232	0.580	1283.00	1283.00	8	0.0050	2.50	0.875	1280.00	1278.50
	G-MH 7	G-MH 6	300	0.263	0.655	1283.00	1282.50	8	0.0050	2.50	0.875	1278.50	1277.00
	G-MH 6	G-MH 5	300	0.272	0.680	1282.50	1281.50	8	0.0050	2.50	0.875	1277.00	1275.50
	G-MH 5	G-MH 4	200	0.288	0.700	1281.50	1281.00	8	0.0050	2.50	0.875	1275.50	1274.74
	G-MH 4	G-MH 3	200	0.310	0.775	1281.00	1281.00	8	0.0050	2.50	0.875	1274.74	1273.98
	G-MH 3	G-MH 2	300	0.326	0.815	1281.00	1281.00	10	0.0038	2.50	1.350	1273.98	1272.84
	G-MH 2	G-MH 1	300	0.372	0.930	1281.00	1281.00	10	0.0038	2.50	1.350	1272.84	1271.70
	G-MH 1	T-MH 10	300	0.405	1.010	1281.00	1282.00	10	0.0038	2.50	1.350	1271.70	1270.56

LINE	From manhole	To manhole	Length (ft)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation (ft)		Dia. (inch)	Grade of sewers	Vel. flowing full ft/sec	Cap. flowing full (cfs)	Invert ~~Ground~~ Elevation	
						Upper manhole	Lower manhole					Upper manhole	Lower manhole
Trunk	T-MH 21	T-MH 18	590	A+B	1.250	1288.50	1286.40	8	0.0107	3.60	1.25	1278.58	1272.28
	T-MH 18	T-MH 17	150	A+B+C	2.630	1286.00	1286.00	15	0.0024	2.50	3.00	1271.68	1271.32
	T-MH 17	T-MH 14	900	A+B+C+D	4.950	1286.00	1284.40	21	0.0014	2.50	5.90	1270.82	1269.56
	T-MH 14	T-MH 10	1125	A+B+C+D +E	6.620	1284.40	1281.00	21	0.0018	2.80	6.75	1269.56	1267.53
	T-MH 10	Lifting Pumping Station	2700	A+B+C+D +E+F+G	8.920	1281.00	1269.50	21	0.0028	3.70	8.10	1267.53	1259.43

CHAPTER 6

SEWAGE TREATMENT

The main difficulty in sewage disposal is due to the presence of puterescible organic substances. These substances necessitates treatment of the sewage in order to avoid health hazards, pollution of natural waters, bathing beaches, and other objectionable conditions. Though these deleterious organic substances are small in amount - generally considerably less than one-tenth of 1 percent, the remaining 99.9% of the sewage being water - their successful handling and treatment constitute the essential problem in sewage disposal.

There are several different methods of treatment. The one to be selected among these several must be such as to fit the local conditions and give desirable results to a certain standard. Therefore in developed countries, like U.S.A. and U.K., basic design criteria are established by the regulatory bodies. Most of the designs for treatment as well as the strength of the effluents have to follow certain specific official standards. But in developing countries where no such official restrictions are imposed, deviations are not ~~usual~~ unusual. However efforts will be made in this design to keep the method of treatment and the standard of its effluent such as not to have undesirable effects.

6.1. Installation Planning

The fulfillment of a sewerage project of any considerable size requires much time and the expenditure of large public funds. The construction often causes public inconvenience, so that in general it will prove economical to design the system so that its components will be capable of serving the community for the maximum forecast development. But the planning should be such that installation could be done as and when needed due to the following considerations.

6.1.1 Economy

6.1.2 Proper Operation

6.1.1, **Economy:** - Installation of all the required units at one time will result in public inconvenience on account of an undue financial burden.

6.1.2, Proper Operation:- To put all the units at the same time would result in poor operating conditions.

So, therefore, it is advisable to carry out installation in two stages. Half of the units will be installed for the present while the remaining would be added as and when their need arises.

6.2. Extent of Treatment

The extent upto which the effluent should be rendered harmless depends upon the standards set by the regulatory agencies which in turn depend upon the quality of the raw sewage and the method of final disposal of the effluents. In case the receiving body is a river or stream the extent of treatment will depend upon the discharge of the stream or river and its utility downstream from the point of discharge of the effluent into the water body. However, if the disposal is on land the degree of treatment should be such as to render the effluent harmless and unobjectionable for human beings, cattles and fish.

6.3. Method of Disposal

Bannu, no doubt, has a river flowing towards its North in west-east direction at a distance of about 2 miles from the city. But the river remains dry most of the time throughout the year due to the construction of Baran Dam and a diversion weir upstream. It is therefore unwise to dispose off the effluent into this river on account of the following reasons:

(1) Flow in the river comes once in a while i.e. the river is non pereneal therefore the treated effluent without dilution may create nuisance.

(2) Effluent or raw sewage depending upon the location of the plant may necessitate pumping on account of unfavourable topography towards the river.

(3) It will necessitate a longer trunk line.

In light of the above discussion it is advisable to resort to the disposal on land.

6.4. Effluent Utilization

Effluents from the treatment plants can be utilized for such purpose like irrigation, ground water recharge, industrial uses and other purposes depending upon the need and scarcity of water for the purpose. However, for Bannu it would be too early to resort to the purification of sewage for purposes other than irrigation. It would therefore be economical to utilize the effluent for irrigation purposes on account of the following reasons:-

(1) Non availability of a suitable perennial water body to receive the effluent.

(2) Availability of sufficient land for the purpose.

(3) Need for irrigation water.

(4) Arid climatic conditions.

(5) Favourable topography for gravity flow.

(6) Deep water table, providing longer percoalation path and thus avoids the danger of ground water contamination.

(7) Percoalating water will recharge ground water.

(8) Effluent utilised for irrigation would bring sale proceeds at the rate of Rs 40.00[14] per acre per season.

6.5. Quality of the Effluent

Quality of the effluent greatly depends upon its utilization but it is imperative to render it safe from health hazards to human beings and cattle etc. before it is put in the fields for broad irrigation. It should be free from pathogens. Its B.O.D. and suspended solids load must be low enough so as not to impaire with the usual yield capacity of the soil.

Sewage effluents, from stabilization lagoons, with reduction in B.O.D. from 70 to 80%[7] have been utilized successfully for broad irrigation. Illinois and New York standards[13] state that a complete treatment plant employing biological process should be capable of producing an effluent with B.O.D. not greater than 15 p.p.m. and suspended solids not more than 30 p.p.m., while the Royal Commission on Sewage Disposal has laid down limits of 20 p.p.m. and 30 p.p.m. on B.O.D. and suspended solids of effluents. The effluent of the plant would be utilized for irrigation purposes and therefore would not be treated to such a high degree of purity. The effluent will be chlorinated so as to render it harmless with respect to pathogens.

However the following additional precautionary measures would also be ensured by proper health education.

(1) Restriction on type of crops grown.

(2) Application of scientific farming methods.

6.6. Treatment Plant Location

Location of treatment plant depends upon the following:

(1) Prevailing wind direction.

(2) Topography.

(3) Health Protection.

(4) Flooding danger.

(5) Cost.

(6) Characteristic of the soil.

(7) Public attitude.

(8) Method of final disposal of the effluent.

(9) Availability of land.

(10) Access to the site.

(11) Sludge disposal considerations.

Keeping the above factors in mind the site for the treatment plant is selected towards south-east of the city. It is located at a distance of about 2,000 ft. from the populated area, so that the nearby inhabitants may not feel inconvenient on account of the operating and maintenance troubles which are not unusual.

6.7. Method of Treatment

The choice of a certain method of treatment from among several depends upon certain considerations which are summarized as under:

(1) Disposal of effluents.

(2) Required quality of the effluents.

(3) Characteristic of the waste to be treated.

(4) Operation and maintenance requirements.

(5) Availability of skilled personnels.

(6) Operation and maintenance costs.

(7) Innitial cost.

(8) Availability of land.

(9) Topography of the area.

(10) Size of the community to be served.

(11) Climatic conditions of the area.

(12) Relative health hazards and other objectionable conditions.

(13) Availability of materials employed.

(14) Ease of increasing capacity.

(15) Quality and quantity of sludge produced, its handling facilities and area requirements.

The method of treatment to be adopted will be considered in the light of the aforementioned factors but availability of land, operation and maintenance cost and the availability of skilled personnel will be given greater importance due to their marked effect as applied to the city of Bannu.

The following method in relation to their applicability to the city of Bannu will be studied.

6.7.1 Disposal by Irrigation

Besides sewage disposal into a body of water, called dilution, there is a method of sewage disposal upon land called irrigation. Two types of irrigation projects are practiced - sewage farming or broad irrigation or land treatment and subsurface irrigation.

When the sewage is applied to the fields its treatment starts with a physical processes namely filteration and aeration. The action of the soil is thus to strain out the suspended material while oxygen is taken from atmosphere and soil air. The filteration results in soil clogging while aeration needs intermittent sewage application to the land. Soil texture is also an important consideration because filteration will not be permitted by stiff clay.

The area requirement, given in the table greatly depends upon the type of soil. On the average one acre of land is required per 200 persons.

Fats and soaps found in sewage usually clog the pores of the soil and the land is said to be sewage sick[15]. This will not only result in poor crop yield of the land but the problem of nuisance will also result.

Subsoil conditions should also be favourable otherwise the sewage may find its way down directly into the ground water thus resulting in its contamination. Lime formations for example are not considered desirable.

TABLE 6.1 (11)

AVERAGE AMOUNT OF LAND REQUIRED

FOR TREATING A DRY WEATHER FLOW

Class of Soil	Method of Working	Vel. of settled sewage which can be treated per acre per 24 hrs	Total Area for 1,000,000 gal. in acres
I. Good soil and subsoil	Filteration with cropping	14,400	70
I. Good soil and subsoil	filteration with little cropping	30,000	33
I Good soil and subsoil	Surface irrigation with cropping	8,400	121
II Heavy soil on clay	Surface irrigation with cropping	6,000	167
III Stiff clay soil on dense clay	Surface irrigation with cropping	3,600	278

From the above discussion and due to costly land and unfavourable soil conditions of Bannu, this method is not recommended.

6.7.2 Imhoff Tank

(1) It employs both sedementation and sludge digestion in a two compartment tank unlike septic tanks which carry both of the function the same compartment. This is an old method of treatment and is considered to be inefficient in light of the most efficient modern plants. Moreover this method is claimed to be relatively cheap which actually is not so, because the desirability for high quality effluent from this process may necessitate the construction of filter beds and secondary sedementation units. However these tanks are not recommended in this report for the following reasons:-

1) Difficult in operation and maintenance on account of undesirable conditions, foaming and odours etc.
2) These tanks are usually employed for very small communities like institutions etc. while for big community like Bannu they will cost more due large area requirements.
3) They are not of desired efficiency.
4) They would require secondary treatment for the effluent.

6.7.3 Oxidation Ponds or Lagoons

It is the simplest and cheapest method for the stabilization of sewage provided adequate land and sunshine conditions permit its use. Adequacy and cheapness of land is necessary because of the large area requirements for oxidation lagoons while maximum possible sunshine is necessary for photosynthetic processes of algae. This method gives effluents full of green manure comparable with those of treatment plant processes.

The recommendation for this method cannot be done due to the following reasons:

(1) Large area requirements cannot be met by a city like Bannu.

(2) Improperly operated lagoons may result in undesirable conditions, odour and flies breeding etc.

(3) Longer transmission lines may be necessary due to No. (2) above.

(4) Lining of walls and bottoms may also add considerable sum to the capital cost.

6.7.4 Oxidation Ditch

Oxidation ditch basically resembles the activated sludge process. It gives an effluent which is comparable with other modern treatment plants employing biological processes. It has been used successfully for populations upto 12,000. The advantages claimed for this method are:

(1) Simple in construction.

(2) Simple in operation.

(3) Low maintenance.

(4) Low area requirement as compared with stabilization lagoons.

The method, however, has not yet been used for population greater than 12,000,therefore its feasibility for Bannu too cannot be justified. Moreover it has certain additional disadvantages also, like:-

(1) Sensitivity to shock loadings.

(2) Large quantity of sludge production.

(3) Problem of foaming.

(4) Effluent is not highly nitrified.

6.7.5 Activated Sludge Process

One of the most important methods of sewage treatment is that employing activated sludge which by aeration or agitation has become flocculent and accumulated a population of aerobic bacteria. This process was developed because of a need for a method for treating a large volumes of sewage which the trickling filters or the older methods could not handle.

Activated sludge process has the advantages of having small innitial cost, requiring small land area and very little head. Therefore this process is of great service where area is small as happens frequently in industrial plants. It can produce an effluent that is very completely treated and that may be discharged with safety where little or no dilution is available. Reduction of suspended, solids, B.O.D. and coliforms are expected upto 95% for each. Danger of odor and other nuisance is small.

However this process has the following disadvantages too.

(1) Greater sensitivity to shock loadings.

(2) High operational cost.

(3) Skilled personnel required for operation.

(4) No flexibility in operation on account of such obstacles like building up an activated sludge and the power rate changes by electrical utilities.

(5) Its sludge commonly gets upset with the resultant poor effluent.

(6) A common difficulty is the accumulation of froth at the aeration tank.

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

6.7.6 Trickling Filters

Trickling filters have the following advantages:-

(1) Can withstand shockloads and over loads to a greater degree than any other method, without up-setting the operation of the plant unless a toxic material is in the sewage or the temperature of the medium throughout falls close to freezing.

(2) Low operational cost.

(3) Comparatively low skilled attendance.

(4) Little foaming problem.

(5) Efficiency in B.O.D. reduction varies from 75 to 90%, and can be designed for any degree of treatment.

(6) Effluent is highly vitrified.

(7) Sludge produced in the final settling tank is low.

(8) Works efficiently in hot weather.

Disadvantages of trickling filters over an activated sludge process:-

(1) Higher head losses.

(2) Fly breeding and odor nuisance.

(3) Higher innitial cost.

(4) Relatively large area requirements.

(5) Higher suspended content in the filter effluent.

(6) Reduction in efficiency upto 20% due to fall in temperature and in some areas with low temperature complete cloggings have been observed.

Innitial costs for activated sludge process and trickling filter for their various capacities, according to Schroepfer, are summarized in table 6.2[13] while summary for operating costs is given in table 6.3.

TABLE 6.2

COMPARATIVE CONSTRUCTION COST OF

TRICKLING FILTER AND ACTIVATE SLUDGE PROCESSES

(Cost in Thousands of Dollars)

Plant Capacity or Sewage Treated m.g.d.	Trickling Filter Plants		Activated Sludge Plants			
	Usual Lower Limit	Usual Higher Limit	Usual Lower Limit	Average aeration 5hrs	Average aeration 6hrs	Usual Higher Limit
10	900	1,500	560	560	700	940
25	2,130	3,550	1,280	1,600	1,800	2,120
50	4,130	6,850	2,500	3,200	3,500	4,200
75	6,000	10,000	3,700	4,750	5,200	6,200
100	7,900	13,200	6,300	6,300	6,300	8,200

TABLE 6.3

COMPARATIVE COST OF OPERATION & MAINTENANCE

FOR TRICKLING FILTER AND ACTIVATED SLUDGE

PROCESSES

(Cost in Thousands of Dollars)

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

Comparisons for both innitial and operating costs shown in the above tables indicate that trickling filters though cost more to install yet can be operated with cheaper cost, therefore it is important to consider the ultimate cost in deciding for adoption of a certain method because a daily operating cost of Rs. 10 per day seems a small item to apply to the cost of production, but this is Rs. 3650 per year, which is equal to the interest at 5% on a principal sum of Rs 73,000. Much additional filter units could be built for less than that sum.

Moreover the test of any unit or method of treatment is in the cost per unit of work done by that method or unit. A sample unit which cost little may not be as effective in reducing B.O.D. etc. as a larger one though it may cost more yet the cost per unit of B.O.D. etc. reduction may be low, thus the larger and the more expensive unit may cost cheaper per unit of work done. However cost per unit of work done in case of trickling filters must also be low, as both processes are not differing much in efficiencies from each other.

No doubt trickling filters too have certain disadvantages but these are not of a serious nature and can be minimized or altogether removed if proper precautions are taken. Odours, for example, are rarely troublesom where rotary distributors are used, and would be easily prevented by proper operation and maintenance. Moreover, even if it could not be cured for a while due to one or another reason, the location of the treatment plant is such that its odor will not reach the inhabited area.

The problem of fly nuisance is controlable by flooding the filter bed as and when required.

Further more the larger area requirement for trickling filters as applied to the city of Bannu at the cost of a higher and continuous financial burden due to operation and maintenance of activated sludge process would not be a hurdle.

Bannu has favourable temperatures for biological processes, therefore the efficiency or working of trickling filters would not be impaired on account of low temperatures as is usually the case with cold areas.

From the aforementioned discussion and comparison, it is concluded that trickling filters are more economical and suitable for Bannu than any other treatment process and would therefore be adopted in this report.

CHAPTER 7

DESIGN OF THE TREATMENT PLANT

7.1. Primary Treatment

Primary sewage plant units may remove from raw sewage substances which may interfere with the successful performance of later secondary sewage treatment processes. Primary treatment of raw sewage consists of the following:

(1) Screening.

(2) Grit removal.

(3) Pre-aeration (Most oftenly omitted than provided)

(4) Primary sedimentation.

7.1.1 Screens

Screening of raw sewage is necessary at all sewage treatment plants, except possibly those employing septic tanks.

The main function of screens is to remove large suspended and floating material which otherwise may:

(1) Clog pipes, pumps, trickling filter nozzles and air diffusers.

(2) Wrap around plain settling tank sprocket wheels, chains, and other moving mechanical equipments.

(3) Resist digestion and increase scum troubles.

(4) Impede the proper conditioning of raw sludges in sludge conditioning tanks.

Design Criteria

(1) Screens will consist of rectangular bar with clear spacing between them as 1 inch.

(2) The bars will be 3/8 by 1½ in in cross-section with the longest dimension placed parallel to the flow.

(3) Velocity through the screens should be neither too high nor too low. The former defeat the very function of the screens by pushing the material out while the later results in grits deposition in the screening chamber[17]. However the lower the velocity the greater will be the amount of screenings removed[17]. A velocity of 2 ft/sec through the screens is desirable.

(4) The inclination of bars to the horizontal varies from 30° to 60°, therefore an angle of 45° is adopted.

(5) As the screens will be mannually cleaned on account of the availability of cheap unskilled labour therefore a horizontal drainage plateform, for partial dewatering of the sewage, would also be provided.

(6) The bottom of the screens shall be at least 6 inches below the invert of the sewer to allow space for collection of large particles.

Screenings are disposed off generally by incineration, burial or grinding, however in this case these would be disposed off with the general garbage collection of the city at frequent intervals.

7.1.2 Grit Channel

Grit chambers remove sand, particles of dust silt, stones and other heavy debris from raw sewage. Following screening of raw sewage, this grit should be removed to prevent the mechanical wear which occurs through abrasive action on packing, pumps and other mechanical equipments, and to avoid clogging difficulties from grit lodging in pipe lines, values, tank hoppers and other places where low velocities occur. The presence of grit in the sludge also reduces the effective capacity of settling tanks and digestors.

It is desirable to remove in grit chambers all grit particles having a size of 0.2 mm or larger.

Design Criteria

(1) As mentioned earlier all grit particles having size of 0.2 mm or larger are to be removed.

(2) The settling velocity of 0.2 mm size particle is taken at 0.1 ft/sec.

(3) The horizontal velocity through the channel will be kept at 1 ft/sec[16].

(4) Two channels will be provided so that the variable flow difficulty can be overcome by bringing one or both of them depending upon the rate of flow.

(5) Velocity control device in the channel is important and therefore a partial flume is adopted for this purpose on account of its simplicity in construction and minimum head loss.

(6) The grit will be cleaned mannually on account of cheap labor availability and high maintenance and operation cost of mechanically cleaned chambers.

(7) The channel will be provided with an open jointed pipe so as to facilitate draining of the sewage from the channel before its cleaning.

(8) As the grit is cleaned mannually, therefore organic matter content presence in the grit is sure therefore a separate grit wash chamber, with its over flow back to the grit channel, will be constructed.

Clean grit containing not over 15 percent[16] volatile solids can usually be disposed of as fill without nuisance. Well washed grit may be used on sludge drying beds. It can also be used for surfacing walks and roads. Grit can also be used as covering material for burial of screenings.

7.1.3 Pre-aeration Tank

Aeration tanks following screens and grit channels aerate the raw sewage for the following purposes:-

(1) To remove gases like H_2S which creates odor and increases the Cl_2 demand of the sewage. The release of gases and the addition of O_2 reduce odors in septic sewage.

(2) To promote floatation of excessive grease.

(3) To aid in the coagulation of colloids in raw sewage for the purpose of high removal of suspended solids.

However pre-aeration of raw sewage is more often omitted rather than provided and therefore will not be provided in this report due to the following reasons:-

(1) No such excessive grease load is handled.

(2) The sewage will arrive at the treatment plant in a fresh condition on account of the reasonable velocities maintained in the sewers.

(3) It will add unnecessarily to the operation and maintenance cost.

7.1.4 Primary Sedimentation Tank

These are being employed for the purpose of retaining the raw sewage long enough, and at a low enough velocity, to deposit the settleable solids which are then removed before anaerobic conditions prevail. They help in scaling down the size of secondary treatment units and also avoid the danger of clogging etc.

Design Criteria

(1) Would be operated on continuous flow basis on account of lower cost and less head loss as compared with the fill and draw system.

(2) Would be circular in plan because circular tanks

offer less operational troubles as compared with the rectangular shaped tanks.

(3) Sludge withdrawal and draining of the tank would be facilitated by radial sloping bottom towards the centre. A slope of 2 inch per foot would be provided.

(4) The units would be designed for average rate of flow.

(5) The over flow rate would not be allowed to exceed 1,500 gallons per sq. ft. of the surface area per day.

(6) Weir loading shall not exceed 15,000 gal per day per lineal foot and preferably should be upto 10,000 gal. per lineal foot.

(7) A concentric influent baffle, having diameter 15 percent that of the tank diameter and extension of 4 ft. below surface[(5)], would distribute the flow uniformly.

(8) A detention period of 2 hrs would be provided.

(9) Baffles will be provided allroad ahead of the overflow weir which shall extend at least 8 inches below the water surface in order to retain floating material.

(10) Effluent weir would be provided all round the tank periphery.

(11) Sludge removal, from the hopper, would be by pumping.

(12) Revolving sludge scrapers would be used to scrape the sludge towards the hopper.

(13) Scum removal would be faciliated by providing a van with sludge scrapping equipment.

(14) Capacity of the sludge hopper should be sufficient so as to prevent the liquid from finding its way out through the sludge withdrawal line.

(15) Effort would be made to have tanks of equal capacities with equal elevations so as to facilitate equal distribution of sewage.

(16) Required machineries and other equipment etc. would be supplied by the manufacturers accordingly.

7.2. Secondary Treatment

The reduction in B.O.D. by primary sedimentation tank ranges from 25 to 35% thus producing an effluent which is still high in B.O.D. so further reduction is attained by secondary treatment plants employing biological processes.

Most of these methods, employing biological processes, have been discussed in general with due consideration to their merits and demerits. Activated sludge process, disposal by irrigation, Imhoff tanks, oxidation ditches and stabilization lagoons were not considered preferable to trickling filters. However trickling filters were not analyzed with respect to intermittent sand filters which are also considered to be unfit for the area due to larger area and great quantity of sand requirements.

Trickling filters, being more suitable than any other method, would be used. Trickling filters are usually followed by humus tanks, therefore the secondary treatment would consist of the following.

(1) Trickling filters.

(2) Humus tanks.

7.2.1 Trickling Filters

There are certain variables which effect the efficiency and design of trickling filters, the most important of which are as under:

(1) Character and strength of raw sewage.

(2) Loading to be applied both hydraulic and organic.

(3) Recirculation ratio.

(1) Character and Strength of Sewage:- The sewage is of a normal concentration with B.O.D. load as 500 p.p.m. The industrial waste is also not of unusual character for handling by the municipal sewage plant. Moreover the volume of waste is not large as compared to the sewage flow, however the waste would be proportioned to the flow of sewage for safe handling by the plant.

(2) Hydraulic and Organic Loadings:- Classification of the trickling filters is done according to the hydraulic and organic loads applied to the filter. Though there are no definite figures of these loadings for both yet certain ranges have been fixed which are given in table 7.1 and would be adopted in this report.

TABLE 7.1

ALLOWABLE FILTER LOADINGS

Loading	Type of filter	
	Low rate	High rate
Hydraulic	25 - 100 gal/day/ft^2 (1.1 to 1.4) 10^6 gal/ acre/day	200-1,000 gal/day/ft^2 (8.7 to 44) 10^6/gal/ acre/day
Organic	5 - 25 lbs/1,000 ft^3/ day	25 - 300 lbs/1,000 ft^3/ day

Table 7.1 shows that the hydraulic loading in case of high rate filter is about 8 to 10 times more than low rate filter. The high rate filter will be adopted in this report because:

(1) Smaller area requirements.

(2) Low innitial cost.

(3) Lees head required for operation.

(4) Less effect of sewage temperature on account increased organic and hydraulic loadings.

Trickling filters may be either single or double stage, depending upon the organic load to be handled. A single stage filter is adopted for this report due to the following reasons:

(1) Single stage filters cost less, both for construction and maintenance.

(2) Desirable result can be achieved by increase in filter size.

(3) Recirculation System:- Recirculation is the most accepted method for obtaining high B.O.D. reduction and uniform application of sewage to the trickling filters. However it results in higher operation and maintemance cost, therefore would be kept minimum at the expense of

increased filter size, but cannot be completely eliminated due to the following reasons:

(1) It feeds the filter continuously irrespective of the variation in sewage flow.

(2) It helps in dilution of strong sewage.

(3) The organic content of recirculated effluent is brought in contact with micro-organisms more than once resulting in higher B.O.D. reduction.

(4) It freshens the influent and reduces odor.

(5) It reduces filter clogging and helps in fly control.

(6) It equalises and reduces loading and thus improves efficiency.

Various schemes of recirculations can be used depending upon the number of filter stages, head available and wether the raw sewage must be pumped. A recirculation ratio such as to feed the plant at a constant rate of 2.5 x average flow as shown in figure 7.1 is adopted due to the following reasons

(1) Recirculation will keep the primary tanks influent in fresh condition thus resulting in reduced scum formation.

(2) Uniformity in load application to the filters can be obtained.

(3) Reduction in the final tank capacity is obtained if the recirculation takes off from the tank hopper.

(4) The sluaghed off material from the filters in the recirculated flow would help greatly in making colloidal materials of primary tank to settle.

The recirculation will be controlled with the help of sewage level in the wet well of recirculation cum raw sewage lifting pumping station. (The recirculation system is shown in figure 7.1)

Design Criteria

(1) The filter will be circular in plan.

(2) The filter will be 6 ft deep at the centre, its bottom will slope towards the centre.

(3) Variation in the flow will be damped by the recirculation controlled in the manner as stated earlier.

(4) The filter will operate continuously for 24 hours.

(5) Filter medium will be composed of hard stones free of dust with sizes varying between 2½ to 4 inches with smallest at the top layer.

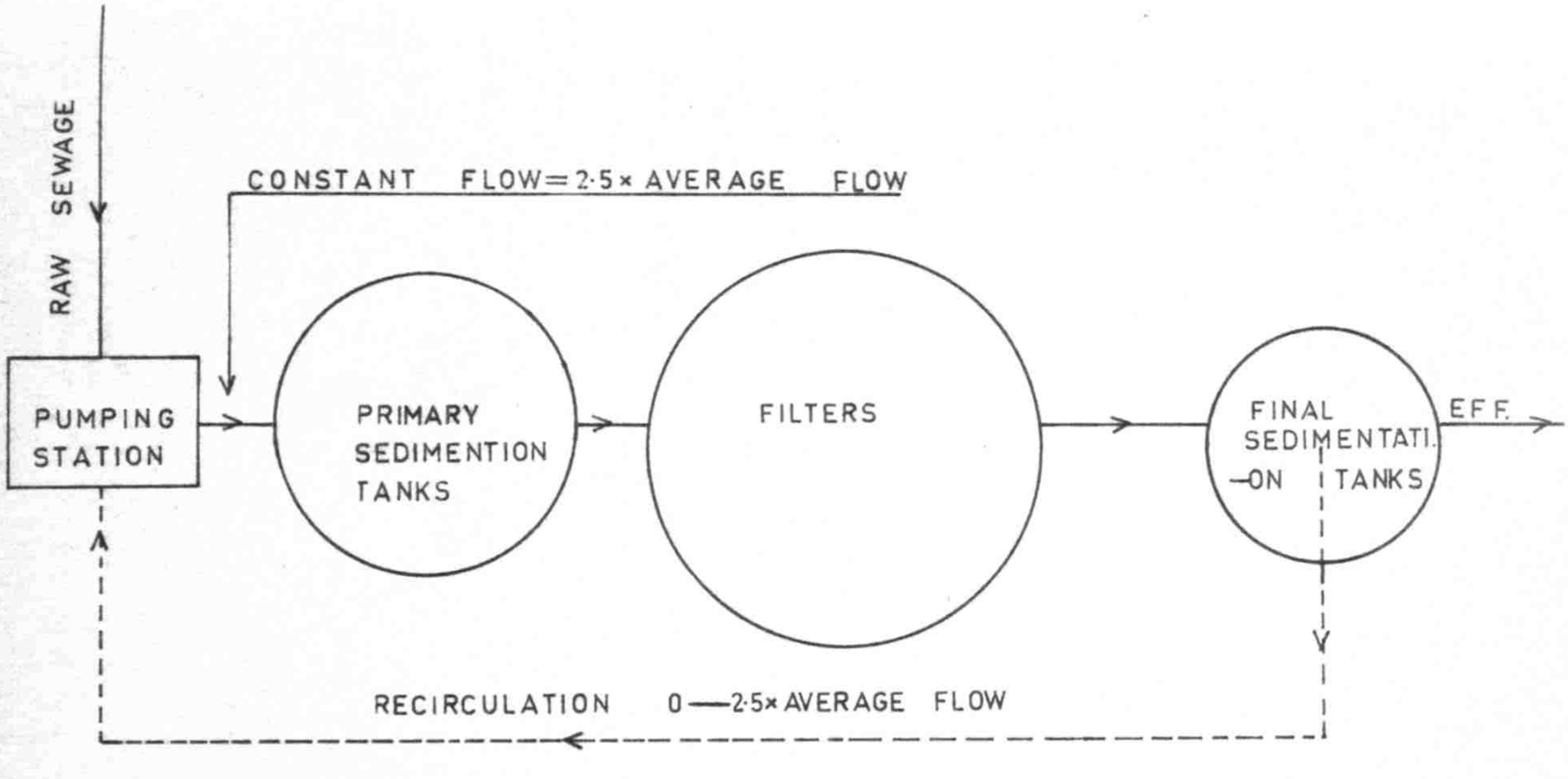


FIG. 7·1 RECIRCULATION DIAGRAME

(6) The arrangement for flooding should be there for the purpose of fly control and therefore, the underdrain should have gate valves for closing purposes.

(7) Ventilation of the filter would be maintained through air vents on the under drain system.

(8) The drains will be covered with concrete blocks.

(9) The drainage channel will be provided to conduct out the flows from under drains and to admit the air to the under drains for ventilation.

(10) Velocity in the drainage channel will be kept 2 to 3 ft/sec.

(11) The walls of the filter will be made of R.c.c. 9" thick.

7.2.2 Humus Tank or Secondary Sedimentation Tank

Those finely divided suspended solids which do not settle in primary sedimentation tanks also find their way out of the trickling filters with modification that they are oxidized by microbial lives in the trickling filters turning them into settleable substances. Therefore secondary sedementation tank is necessary in order to get clearer effluent. Moreover biological growth takes place on the stones surfaces which begin to slough off in the filter effluent which are settled in the humus tanks.

Design Criteria

(1) The tank will be circular in shape.

(2) It will be operated continuously.

(3) The tank bottom will be sloping towards the hopper. A minimum slope of 1.5 horizontal to 1 vertical is desirable, however a slope of 2:1 is preferrable[16].

(4) Cleaning of the tank would be done mannually.

(5) It will be designed for average rate of flow.

(6) Surface loadings will be kept between 600 to 1500 gallons/ft^2/day.

(7) Over flow rate of weir shall not be allowed to exceed 15000 gal/day/R.ft.

(8) Inlet baffles will be provided as in the case of primary sedimentation tank.

7.3. Treatment and Disposal of Sludge

Treatment plants which handle sewage in million gallons per day usually produce sludge in thousands of gallons, Hence safe handling and disposal of these wastes are also of great importance because of the following:

(1) Sludge is composed of such substances which give offensive character to the untreated sewage.

(2) These substances produced from sewage treatment plants are still capable of rapid decomposition.

(3) Only small amounts of sludge are solids the rest being water.

(4) Their polluting potentialities are high.

Sludge may be disposed off by various methods, including lagooning, burning, land filling and dumping into the sea or natural fresh water bodies while in other methods it may be treated before it is disposed off.

7.3.1 Sludge Disposal

(i) Disposal into a body of water

This method of disposal being the cheapest but is out of question on account of the non-availability of such a water body to receive the sludge.

(ii) Lagooning of Sludge

The sludge also cannot be lagooned on account of large area requirements which the city of Bannu cannot afford at the cost of valuable and fertile land. Moreover this method is also not free of objections. Some authorities look to or at this method as an inadequate, incomplete and unsatisfactory method of sludge disposal. It also gives offensive odors. The land used for this purpose should be of little or no value for other purposes.

(iii) Burial

This method, though in use, has not yet been employed extensively on account of large area requirements. Sludge disposed off in this manner may remain moist and malodorous and the land may be rendered unfit for further sludge burial. This method, therefore, would not be adopted in this report.

7.3.2 Sludge Digestion and Drying

Sludge digestion is practiced due to the following reasons and would therefore be adopted.

(1) It breaks the organic matter of the sludge into simple compounds and is consequently rendered inoffensive.

(2) It causes reduction in sludge volume and therefore can be handled easily.

(3) It causes compaction of the sludge and hence reduction in its water content.

(4) It makes the sludge dry easily on drying bed because digested sludge gives up its remaining water content very easily.

(5) Digested sludge is used as a fertilizer.

(6) Coliforms are reduced to about 99.8%.

Sludge after digestion is subjected for further treatment by one of the following methods:

(1) Drying beds.

(2) Centrifuges.

(3) Vacuum filters.

(4) Elutriation.

(5) Floatation.

The first method is, however, adopted after due consideration for the operation and maintenance costs for the rest of the methods.

7.3.3 Sludge Digestion Tank

The satisfactory disposal of the concentrated organic solids removed from sewage in the primary sedementation tanks and the excess biological solids from the trickling filters is brought about in sludge digestion tanks more oftenly by anaerobic bacteria. The design criteria of a sludge digestion tank can be very easily detailed under the following headings:

(1) Daily sludge volume to be digested.

(2) Digestion period.

(3) Method of heating.

(4) Type of tank.

(5) Method of stirring.

(6) Addition and removal of sludge and supernatant.

(7) Type of cover.

(8) Method of gas collection.

(1) Volume of Sludge

The quantity of sludge to be handled daily by the sludge digestion tank mainly depends upon the process of treatment employed for sewage treatment and the strength of the sewage. As no such definite figures in this respect are yet available therefore it would not be far from correctness to assume the applicability of figures holding for normal domestic sewage. The amount of suspended solids in normal sewage may be taken as 0.20 lbs[5] per capita daily. If there is a moderate amount of industrial waste it may rise to 0.22 lb[5].

However 100% exactness cannot be claimed unless actual tests have been carried on the sludge produced.

(2) Period of Digestion

The process of sludge digestion, other factors being favourable, is mainly temperature dependent. With the increase in temperature the digestion rate increases resulting in reduced tank capacity and vice versa. Digestion takes place in two zones of temperature i.e. thermophilic and misophilic. The digestion of sludge is usually carried out in the later zone with the optimum temperature between 80^{o} and $90^{o}F$.

Bannu on average has favourable temperature for the sludge digestion because the maximum range of temperature in summer is about $120^{o}F$ while the lowest in winter is $37^{o}F$. The temperature of summer therefore is an ideal one for the process of digestion. In winter at day times the temperature becomes favourable due to good sunshine, therefore no heating arrangement would be provided on account of the following reasons:

(1) The low temperature for short duration would not add too much to the cost by providing somewhat

larger capacity tank while the provision of heating arrangements would necessitate greater cost.

(2) No skilled operation would be required.

(3) The maintenance cost would be low.

Type of Tank

The digestive action of sludge is more rapid at the beginning and then slows down. It would therefore be advisable to carry out the digestion in two stages due to the following reasons.

(1) Most of the substances, which inhibit activities in subsequent digestion, are produced in the earlier stages and which if confined to one tank can be easily over come.

(2) The difficulty of drawing supernatant liquor from a single tank due to the gas production and mixing would be over come, because this separation could easily be done from the secondary tank.

(3) The supernatant drawn from the secondary stage would be of a better quality.

(4) Stirring in the primary stage would become easier.

(5) Saving in innitial cost on account of no cover provision for the secondary unit due to low gas production in the unit.

(6) The possibility for sludge to pass without digestion would be reduced to a greater extent.

(7) The possibility of scum formation would be reduced to a greater extent and hence no measures for its prevention would be required. However a small amount of scum if formed would be desirable in order to seal back the odors and heat.

Method of Stirrings

For the purpose of achieving active digestion and preventing objectionable conditions, it is necessary to have a thorough mixing of the incoming raw sludge with the actively digesting sludge in the tank. To secure this aim of stirring the tank content, the following three methods are usually adopted.

(1) Recirculation of tank content by pumping.

(2) Recirculation of the collected gas with the help of compressors.

(3) Mechanical mixing.

The last method i.e. mechanical mixing is preferred because recirculation of tank content necessitates heat exchanger and pumping while the recirculation of gas is out of question because the gas would not be collected. Moreover these two methods would result in larger operational and maintenance cost. Therefore the method of mechanical mixing would be adopted.

This mechanical mixing is best achieved with the help of Simplex Screw pump which is composed of a vertical pipe having screwed spindle inside it. This spindle is rotated by an electric motor. The sludge can be taken either up or down through the tube depending upon the direction of rotation. It is generally operated intermittently.

Method of Adding and Removing Sludge and Supernatant

Raw sludge from the treatment plants will be pumped daily to the digestion tank. The entrance of the sludge should be below the surface of digesting sludge in the tank. This raw sludge would displace an equal volume of sludge to the secondary tank through the over flow pipe. An auxiliary sludge draw-off should also be provided at the bottom of the primary tank and would be used as and when required.

Supernatant is taken off from the secondary tank by means of draw off pipes provided at various levels for obtaining supernatant of varying nature. The supernatant removed from the secondary tank would be removed to the treatment plant for treatment with the raw sewage coming from the city. It would be advisable to feed the supernatant to the plant when the flow is high.

The sludge after the removal of the supernatant will be allowed to dry on drying beds before it is subjected to its use as a fertilizer.

Type of Cover

The primary tank would be provided with a cover so as; to prevent the dispersal of the gas which is being produced actively in the primary tank; to check the offensive odors; to prevent heat losses from the surface; to prevent drying of the top layer of sludge.

Fixed type of cover would be adopted due to the following reasons:

(1) It gives satisfactory service.

(2) It is low in cost.

(3) No maintenance and operation difficulties would be encountered.

(4) There would be no danger of drawing air into the tank because the volume of sludge added would be equal to the volume of sludge removed and moreover

withdrawal of the supernatant would be from the secondary unit and not from the primary tank.

Method of Gas Collection

The gas unless needed by the municipality would not be collected and would be taken off the tank by means of pipe and would be burnt to avoid hazards and nuisance.

Sludge Drying Beds

Sludge after digestion will be applied to sludge drying beds due to the following reasons:

(1) Reduction of liquid sludge to a spadable condition, suitable for disposal as fertilizer or as fill for low lands.

(2) Reduction of bulk of the material to be disposed off.

(3) Limiting the bacterial activities by removal of its moisture content.

The area requirements mainly depend upon the characteristic of the sludge and the climatic conditions of the area. However following design criteria would be adopted.

(1) Usually for digested sludge open drying areas of

1/3 to 1 sq. ft[5] per capita are common for separate sewage system, the lower valve being associated with strictly domestic sewage. However an area of 1 sq. ft. per capita would be adopted in this report on account of the presence of moderate amount of industrial wastes.

(2) Bed depth of 12 in would be adopted.

(3) The sludge beds would be surrounded by concrete curbing about 12 inch high from the bed.

(4) The under drains would be open jointed tiles placed in trenches.

(5) The total thickness of the bed would be 1 foot with coarse sand at the top underlain by layers of graded gravel ranging in size from 1/8 to 1/4 in. at the top to 3/4 to 1½ in. at the bottom[5].

(6) The floor of the bed would be sloping for the purpose of giving proper slopes to the under drains.

(7) The bed dimensions would be 20' x 50'.

(8) Splash plates of concrete would be provided to distribute the wet sludge over the bed without scouring effects.

(9) Dried sludge would be removed mannually and would be dumped into wheel barrows and driven to the site of disposal.

7.4. Design

7.4.1 Design of Grit Channel

Parshall Flume:-

Present discharge = 1.32 cfs.

Ultimate discharge = 3.57 cfs.

Hence $Q_{max.}$ = 3.57 x 2.5 = 8.920 cfs.

and $Q_{min.}$ = 1.32 x 0.50 = 0.660 cfs.

For velocity control by Parshall flume[12]

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

Where:

Q max. = Maximum rate of flow

Q min. = Minimum rate of flow

W = Throat width of the flume

Z = Distance as indicated in figure.

Taking the throat width as 6 inches, then

$$\frac{0.660}{8.920} = \frac{1.1\left(\frac{0.66}{4.1 \times 0.5}\right)^{0.67} - Z}{1.1\left(\frac{8.92}{4.1 \times 0.5}\right)^{0.67} - Z}$$

or

$$0.074 = \frac{1.1 \times 0.468 - Z}{1.1 \times 2.68 - Z}$$

or

$$0.218 - 0.074Z = 0.515 - Z$$

or

$$Z = \frac{0.297}{0.926} = 0.32 \text{ ft.}$$

$$= 3.84 \text{ inches.}$$

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

$$= 2.95 - 0.32$$

$$= 2.63 \text{ ft.}$$

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

$$= 1.1 \left(\frac{0.66}{4.1 \times 0.5}\right)^{2/3} - 0.32$$

$$= 0.515 - 0.32$$

$$= 0.195 \text{ ft.} = 2.34 \text{ inches.}$$

$$\text{Channel width} = \frac{Q \text{ max.}}{d \text{ max.} \times V} = \frac{Q \text{ min.}}{d \text{ min.} \times V}$$

$$= \frac{8.92}{2.63 \times 1} = \frac{0.66}{0.195 \times 1}$$

$$= 3.39 \text{ ft.}$$

Now from table given on page 387 of H. E. Babbit the following values are obtained corresponding to 6 inch throat width (Pl. refer to figure 7.2 also).

A = 2.02 ft.

B = 2.00 ft.

C = 1.302 ft.

D = 1.302 ft.

F = 1.00 ft.

G = 2.00 ft.

K = 3.00 inches.

N = 4.50 inches.

Again Q max. $= 130 \cdot W \cdot N^{3/2}$

where N = 0.375 ft.

Q max. obtained from this formula must be greater than Q max. in the report.

So Q max. $= 130 \times 0.5 \times 0.230$

$= 15.00$ cfs.

Since this flow of ~~24.4~~ 15.0 c.f.s. is larger than 8.92 c.f.s. in the report, hence free flow conditions will exist.

For unaffected velocity in the channel between min. and max. flow, the following condition must be observed.

Area (3) = Area (2) + Area (1)

$(3.39 - 2 \times 0.195) \times \quad = 0.195 \times 0.195$

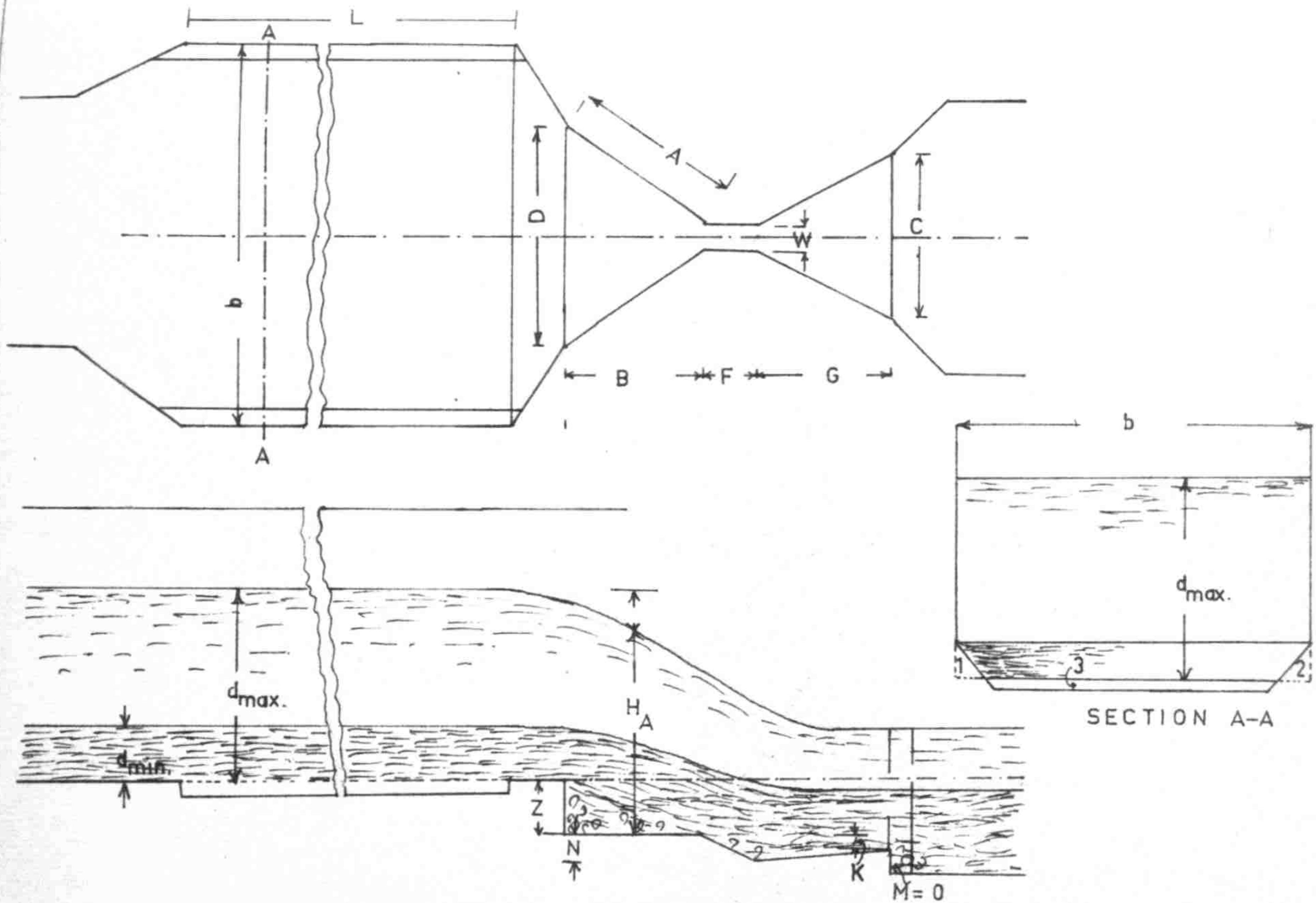


FIG. 7·2 PARSHAL FLUME FOR GRIT—CHAMBER CONTROL or

$$(3.39 - 0.390)\, x \qquad = 0.38$$

or

$$x = \frac{0.38}{3.00} \qquad = 0.127 \text{ ft.}$$

$$B = 1.50 \times Q\ \text{max.}^{1/3} \qquad \text{(should not be less than 2 ft)}$$

$$= 1.50 \times 8.92^{1/3}$$

$$= 1.50 \times 2.07$$

$$= 3.12 \text{ ft.} \qquad \text{(satisfactory)}$$

As $D = 1.302$ ft. (from table)

and $B = 2$ (from table)

Therefore

$$\frac{D}{B} = \frac{1.302}{2} \qquad = 0.65$$

or $\frac{D}{3.12} = 0.65$

or $D = 3.12 \times 0.65$

$$= 2.06 \text{ ft.}$$

As $Q\ \text{max.} = 4.1\ WH_A^{3/2}$

or

$$H_A = \left(\frac{Q\ \text{max.}}{4.1 \times W}\right)^{2/3}$$

$$= \left(\frac{8.92}{4.1 \times 0.5}\right)^{2/3}$$

$$= 2.68 \text{ ft.}$$

For prevention of submergence at the tail the following conditions should be kept in mind.

$$d_c + k > d_{c'}$$

and

$$d_{c'} + M \geq d_c$$

As $d_c = \sqrt[3]{\dfrac{Q}{g b^2}}$ where "b" denotes the throat width.

$$= \sqrt[3]{\frac{8.92}{32.2 \times 0.25}}$$

$$= 1.032 \text{ ft.}$$

and $k = 0.25$ ft.

therefore

$$d_c + k = 1.032 + 0.25$$

$$= 1.282 \text{ ft.}$$

Now $d_{c'} = \sqrt[3]{\dfrac{8.92}{32.2 \times (1.302)^2}}$

$$= 0.575$$

Hence there will be no submergence because $d_c + k$ is greater than $d_{c'}$

Channel Length:-

As d max. $= 2.63$ ft.

By taking settling velocity as 0.1 ft/sec, then the time taken by a particle to settle through the distance

$2.63 \text{ ft} = \frac{2.63}{0.1} = 26.30$ sec.

Taking the horizontal velocity through the grit channel as 1 ft/sec.

$\therefore$ Length $= 26.3$ ft.

$= 27$ ft. (say)

Channel Depth:-

Taking the volume of grit as 2 ft.^3 per million gallon/day, then

Volume of grit $= \frac{8.92}{2.5} \times \frac{60 \times 60 \times 24 \times 7.48}{10^6}$

$= 4.6 \text{ ft.}^3$

Taking the cleaning interval as one week, therefore

Total grit volume $= 32.20 \text{ ft.}^3$

Depth of grit $= \frac{32.20}{27 \times 3.39} = 0.37$ ft.

Taking free board as $= 0.50$ ft.

$\therefore$ Total depth $= 2.63 + 0.37 + 0.50$

$= 3.50$ ft.

Providing two such channels inorder to facilitate cleaning.

7.4.2 Design of Screens

Using racks of $1\frac{1}{2}$" x 3/8" in section with clear distance between then as 1" .

Channel width = 3.39 ft.

c/c spacing of bars = 1 3/16"

therefore number of bars

$$= \left(\frac{3.39}{0.099} - 1 \right)$$

$$= 33$$

Portion of the width occupied by bars

$$= 33 \times 3/8 \times 1/12$$

$$= 1.03 \text{ ft.}$$

Effective width of the screens

$$= 3.39 - 1.03$$

$$= 2.36 \text{ ft.}$$

Velocity at the peak flows through the screens

$$= \frac{Q \text{ max.}}{\text{Effective width x d max.}}$$

$$= \frac{8.92}{2.36 \times 2.63}$$

$$= 1.6 \text{ ft./sec (satisfactory)}$$

Velocity at minimum flows

$$= \frac{Q \text{ min.}}{\text{Effective width x d min.}}$$

$$= \frac{0.66}{2.36 \times 0.195}$$

$$= 1.45 \text{ ft./sec.}$$

$$= 1.5 \text{ ft./sec (say}$$

which is O.K.

7.4.3 Design of Primary Sedementation Tanks

Ultimate flow = 2.309 mgd.

Present flow = 0.85 mgd.

2.5

Taking recirculation as 1.5:1

$\therefore$ Ultimate "Q" av. = 2.5 x 2.309

= 5.96 mgd.

Present "Q" av. = 2.5 x 0.85

= 2.12 mgd.

Taking detention period as 2 hrs and the number of tanks to be used ultimately as 4.

Therefore flow/tank $= \frac{5.96}{4}$ mgd.

= 1.49 mgd.

Tank volume $= \frac{1.49 \times 10^6 \times 2}{7.48 \times 24}$

$= 16700 \text{ ft}^3$

Taking side depth as 10 ft.

$\therefore$ Surface area $= \frac{16700}{10}$

$= 1670 \text{ ft}^2$

Diameter of the tank $= \sqrt{\frac{4}{\pi} \times 1670}$

$= 46$ ft.

Surface loading per day in gallons

$= \frac{1.49 \times 10^6}{1670}$

$= 892$ gal/day/ft^2, which is O.K.

Weir overflow rate per lineal foot

$= \frac{1.49 \times 10^6}{\pi \cdot D}$

$= \frac{1.49 \times 10^6}{3.143 \times 46}$

$= 10300$ gal/foot/day

which is satisfactory.

Design of the Inlet Baffle:-

Inlet baffle diameter $= 0.15 \times$ tank diameter

$= 0.15 \times 46$

$= 6.90$ ft.

Depth of baggle ff = 4 ft.

Two tanks would be provided at the present which would work under the following loading conditions:-

(i) Surface loading $= \frac{2.12}{2} \times 10^6 \times \frac{1}{1670}$

$= 635$ gal/day/ft^2

which is satisfactory.

(ii) Weir loading $= \frac{2.12}{2} \times 10^6 \times \frac{1}{3.143 \times 46}$

$= 7350$ gal/ft/day

which is satisfactory.

(iii) Detention time $= t$

$= \frac{16700 \times 24 \times 7.48 \times 2}{2.12 \times 10^6}$

$= 2.80$ hrs, which is O.K.

7.4.4 Design of Trickling Filters

(1) Design of filter size:

Q av. $= 2.309$ m.g.d.

B.O.D. $= 9641$ lbs/day

$= \frac{9641 \times 10^6}{2.309 \times 10^6 \times 8.34}$

$= 500$ p.p.m.

Taking the B.O.D. value in final sedementation tank as 30 p.p.m.

Therefore B.O.D. applied to the primary sedementation tank $= \frac{1.5 \times 30 + 500 \times 1}{2.5}$

$= \frac{45 + 500}{2.5}$

$= 218$ p.p.m.

Taking the B.O.D. removal in primary sedementation tanks as 36.5% from the curve of 200 - 300 p.p.m. 5-day B.O.D. against 2 hrs detention period[5].

Therefore B.O.D. removed $= \frac{36.5}{100} \times 218$

$= 79.50$ p.p.m.

$\therefore$ B.O.D. applied to filters $= 208 - 76$

$= 138.5$ p.p.m.

or $= 6000$ lbs/day. wrong 2660 lbs/day

Acceptable B.O.D. in the effluent

$= 30$ p.p.m.

Therefore filter + final sedimentation tank efficiecy

$$= \frac{(138.50 - 30)}{138.50} \times 100$$

$= 78\%$

From the curves against 78% efficiency, the filter loading = 1 lb. of B.O.D. per cubic yard of filter media.

Total B.O.D. applied to filters

$= 6000$ lbs/day.

$\therefore$ Filters volume $= 6000\ yd^3$

$= 54000\ ft^3$ 1 cu yard = 27 cft and not 9 cft

Adopting 4 filters

$\therefore$ Each filter medium $= 13500\ ft^3$

Taking filter depth $= 6$ ft.

$\therefore$ Surface area $= \frac{13500}{6}$

$= 2250 \text{ ft}^2$

Diameter of filter $= \sqrt{\frac{4}{\pi} \times 2250}$

$= 53$ ft.

Hydraulic loading $= \frac{1.49 \times 10^6}{2250}$

$= 665 \text{ gal/ft}^2\text{/day}$,

which is O.K.

Taking two filters for the present, the hydraulic loading $= \frac{2.12}{2} \times \frac{10^6}{2250}$

$= 472 \text{ gal/ft}^2\text{/day}$

which is O.K.

a) Design of Pipes

(i) Design of inlet pipes to the trickling filters

Ultimate flow = 8.92 cfs. (Recirculation included).

Flow per filter $= \frac{8.92}{4}$

$= 2.23$ c.f.s.

Adopting 12 inches diameter pipe

$\therefore$ Ultimate velocity $= \frac{Q}{D^2} \times \frac{4}{\pi}$

$= \frac{2.23 \times 4}{1 \times 1 \times \pi}$

$= 2.84$ ft/sec (satisfactory)

Using two filters for the present:-

$\therefore$ Present velocity $= \frac{Q}{D^2} \times \frac{4}{\pi}$

$= \frac{3.30 \times 4}{2 \times 1 \times 1 \times \pi}$

$= 2.10$ ft/sec (satisfactory)

(ii) Design of distributors

Adopting a rotary distributor with four arms:-

Ultimate flow per arm $= \frac{2.23}{4}$

$= 0.556$ c.f.s.

Present flow per arm $= \frac{3.3}{2} \times \frac{1}{4}$

$= 0.413$ c.f.s.

Adopting 6 inches diameter pipe

therefore Ultimate velocity

$= \frac{Q}{D^2} \times \frac{4}{\pi}$

$= \frac{0.556}{\frac{1}{2} \times \frac{1}{2}} \times \frac{4}{\pi}$

$= 2.82$ ft/sec.

(which is O.K.)

Present velocity $= \frac{0.413}{\frac{1}{2} \times \frac{1}{2}} \times \frac{4}{\pi}$

$= 2.10$ ft/sec.

which is satisfactory.

(iii) Size, Number and Spacing of Nozzles:

Ultimate flow per arm = 0.556 c.f.s.

Discharge through the nozzles per arm (17)

$= N.C_d.A\sqrt{2gh}$

Where:

N = Number of nozzles/arm

C_d = Coefficient of discharge and its value can be taken between 0.61 and 0.64[17].

A = Cross-sectional area of a nozzle.

h = Head on nozzle

Radius of the filter $= \frac{53}{2}$ = 26.5 ft.

Allowing 1.5 ft. for central pipe and end clearance etc.

$\therefore$ Length of arm = 26.5 - 1.5

= 25 ft.

Keeping head on the nozzle= 4 ft and adopting nozzle

size $= \frac{1}{4}''$

$\therefore Q = N.C_d.A\sqrt{2gh}$

or $\quad 0.556 \quad = N. \; 0.61 \times \frac{\pi}{48} \times \frac{1}{48 \times 4} \cdot \sqrt{2 \times 32 \times 4}$

or $\quad N \quad = 45$

c/c distance between the nozzles

$$= \frac{25 - 1}{45}$$

$$= 6.4 \text{ inches.}$$

(iv) Design of effluent pipe:

Adopting pipe size $= 12$ inches.

Velocity $= \frac{Q}{D^2} \times \frac{4}{\pi}$

$$= \frac{2.23}{1 \times 1} \times \frac{4}{\pi}$$

$$= 2.84 \text{ ft/sec.}$$

(which is O.K.)

b) Design of Outlet Channel

Adopting velocity of 2.5 ft/sec.

Area $= \frac{2.23}{2.5}$

$$= 0.9 \text{ ft}^2$$

Adopting section of 1' x 0.9'

7.4.5 Design of Final Sedimentation Tanks

Ultimate flow (Recirculation included)

$= 5.96$ m.g.d.

$= 8.92$ c.f.s.

Recirculation ratio varies between 0 and 2.5, therefore an average 1.25 is being recirculated from the hopper of the final sedimentation tanks.

Volume of recirculated sewage

$= 1.25 \times 2.309$ or $\frac{5.96}{2}$

$= 2.98$ m.g.d.

Volume of sewage to be retained by the final sedimentation tanks $= 5.96 - 2.98$

$= 2.98$ m.g.d.

Adopting the diameter of the tank as 40 ft, with other dimensions as shown in the figure

$\therefore$ Volume of tank $= \frac{\pi}{4} \times 40^2 \times 4 + \frac{1}{3} \times \frac{\pi}{4} \times 40^2 \times 10$

$= 9240$ ft.^{3}

Adopting two tanks for ultimate conditions

$\therefore$ Volume of tanks $= 2 \times 9240$

$= 18480$ ft.^{3}

Total daily flow $= \frac{2.98 \times 10^6}{7.48}$

$= 40,000$ ft.^{3}

Let the detention time $= t$ hrs.

Then

$$t \times \frac{40,000}{24} = 18480$$

$= 1.11$ hrs. (satisfactory)

Using one tank for the present flow

$$= \frac{3.3}{2}$$

$= 1.65$ m.g.d.

Tank capacity $= 9240$ ft.

$$\therefore t_p \times \frac{1.65}{7.48} \times 10^6 \times \frac{1}{24} = 9240$$

$$\text{or } t_p = \frac{7.48 \times 24}{1.65 \times 10^6} \times 9240$$

$= 1.00$ hr (satisfactory)

Tank surface area $= \frac{\pi}{4} \times 40 \times 40$

$= 1260$ ft^2

Ultimate over flow rate (two tanks)

$$= \frac{2.98 \times 10^6}{1260 \times 2}$$

$= 1180$ gal/ft^2/day

(which is O.K.)

Ultimate weir flow rate $= \frac{2.98 \times 10^6}{2 \times \pi \times 40}$

$= 11800$ gal/ft/day

(which is O.K.)

Surface loading for present conditions

$$= \frac{3.3}{2} \times \frac{10^6}{1260}$$

$$= 1320 \text{ gal/ft}^2\text{/day}$$

which is O.K.

Weir over flow rate for present condition

$$= \frac{3.3}{2} \times 10^6 \times \frac{1}{\pi \times 40}$$

$$= 13200 \text{ gal/ft/day}$$

which is O.K.

7.4.6 Sludge Treatment

The amount of suspended solids in normal sewage is usually assumed to be 0.20 lbs per capita per day, but a moderate amount of industrial waste may rise this figure to about 0.22 lbs. The value of 0.22 lbs per capita per day is adopted in this report due to the presence of the industrial waste.

Solids in normal sewage are shown in table 7.2 while table 7.3 shows the derived values from table 7.2 on the basis of $\frac{0.22}{0.20}$ ratio.

TABLE 7.2

SOLIDS IN NORMAL SEWAGE

State of solids	Mineral (gms)	Organic (gms)	Total (gms)
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

TABLE 7.3

SOLIDS IN THE SEWAGE

Solids	Settleable (lbs)	% of the total	Non Settleable (lbs)	Total
Volatile	$0.086 \times \frac{0.22}{0.195}$	73.5	0.0645	0.1615
	$= 0.097$			
Fixed	$0.031 \times \frac{0.22}{0.195}$	26.5	0.0245	0.0585
	$= 0.034$			
Total	0.131	100	0.0890	0.220

The settleable solids are removed in the primary sedimentation tank while non settleable pass to the secondary treatment units.

Assuming that 8% of the volatile solids are oxidized biologically during filteration and the rest of the solids pass into the final sedimentation tank.

$\therefore$ Volatile solid in the final sedimentation tank

$= 0.92 \times 0.0645$

$= 0.0594$ lbs.

Fixed solid in final sedimentation tank

$= 0.0245$

Total weight of solids per capita per day in the final sedimentation tank $= 0.0839$ lbs.

The total solids in sludge from the primary and final settling tanks are as under

TABLE 7.4

PRIMARY AND SECONDARY SEDIMENTATION TANK SOLIDS

	Primary Sed. tank (lbs)	Final Sed. tank (lbs)	Total (lbs)	%age
Volatile solids	0.097	0.0594	0.1564	72.4%
Fixed solids	0.034	0.0245	0.0585	27.6%
Total	0.131	0.0839	0.2149	100.0%

The volume of wet sludge is expressed by the following formula:

$$V = \frac{W}{S\rho} + \frac{W\,(1 - x)/x}{\rho}$$

Where

W = Weight of daily added dry solids.

S = Specific gravity of solids.

x = Weight fraction of solids in sludge

ρ = Density of water.

The specific gravity "S" is obtainable from the following formula:

$$S = \frac{1}{P/S_v + (1 - P)/S_f}$$

where:

S_v = Specific gravity of volatile solids and is taken as 1.

S_f = Specific gravity of fixed solids taken as 2.5.

So $$S = \frac{1}{0.724/1 + 0.276/2.5}$$

$$= 1.20$$

Taking moisture content of sludge as 95%.

$\therefore$ Volume of sludge per capita per day from the aforementioned expression

$$= \frac{0.2149}{1.2 \times 62.4} + \frac{0.2149\,(1 - 0.05)/0.05}{62.4}$$

$$= 0.0684 \text{ ft}^3.$$

Taking mean temperature in winter = $60^{\circ}F$

From figure 26-1[5], for a temperature of $60^{\circ}F$ the time of digestion is 48 days.

From figure 7-2[5], the reduction in volatile solids = 60.

Considering that 30% of the remain 40% volatile solids is changed into fixed solids, then:

The remaining volatile solids

$$= 0.40 \times 0.1564$$

$$= 0.0625 \text{ ---- } 42\%$$

The remaining fixed solids

$$= 0.0585 + 0.30\ (0.1564 - 0.06256)$$

$$= 0.0867 \text{ ---- } 58\%$$

Total solids $= 0.1492$

Specific gravity of dry solids in the digested sludge

$$= \frac{1}{0.42/1 + 0.58/2.5}$$

$$= 1.53$$

Taking the water content of the digested sludge

$$= 92\%$$

The volume of the wet digested sludge per capita per day

$$= \frac{0.1492}{1.53 \times 62.5} + \frac{0.1492(1-0.08)/0.08}{62.5}$$

$$= 0.0289 \text{ ft.}$$

The required volume of digestion tank per capita is expressed by the following expression:

$$V = \left[V_d + 1/3\,(V_o - V_d)\right] t$$

where:

V = Tank volume per capita in ft^3.

V_d = The daily digested volume of sludge per capita in ft^3.

V_o = The daily volume of raw sludge in ft^3.

t = Time of digestion in days.

$$V = \left[0.0289 + 1/3\,(0.0684 - 0.0289)\right] 48$$

$$= 2.02\ ft^3$$

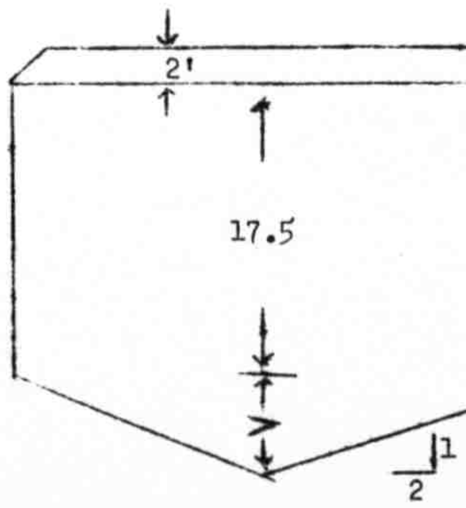
Capacity of tanks $= 2.02 \times 49{,}000$

$= 99{,}000\ ft^3$

Using 4 tanks two for primary and two for secondary stage.

Capacity of one tank $= \frac{49000}{4}$

$= 12250\ ft^3$



Taking tank diameter = 28 ft, then

$$\frac{\pi}{4} \times 28^2 \times x + \frac{1}{3}\ \frac{\pi}{4} \times 28^2 = 12250$$

or

$$x = 17.50 \text{ ft.}$$

7.4.7 Design of Sludge Drying Beds

Ultimate population = 44,000

Industrial population equivalent

= 5,000

Total population = 49,000

Land would be provided at the rate of 1.00 ft^2 per capita, which is = 49,000 x 1.00

= 49000

Providing beds with the dimension of 20' x 50'

$\therefore$ Number of beds $= \frac{49000}{1000}$

= 49

= 50 (say)

Twenty five drying beds would be provided for the present, while ultimately their number will be increased to fifty.

7.4.8 Supplementary Work

a) Distribution Boxes

As the flow through the treatment plant would be by gravity therefore distribution boxes would be necessary in order to have the required pressure and uniform distribution of the sewage to the different units. The boxes would be circular in plan and would have sufficient capacity to accomodate the sewage flow for 30 seconds.

Design of distribution

Average flow = 3.57 c.f.s.

Flow through units = 2.5 x 3.57

= 8.92 c.f.s.

Detention period = 30 seconds.

Volume = 8.92 x 30

Adopting diameter of 8 ft.

$$\therefore \text{ Depth} = \frac{8.92 \times 30}{\pi/4 \, .8 \times 8}$$

= 5 ft.

b) Sludge Pumping Station

A small pumping station will be installed for pumping the sludge to the digestion tanks. One pump would be provided for the ultimate amount of sludge while a 2nd one of similar capacity would be used as a stand-by.

Sludge withdrawal from the sedimentation tanks would be by gravity and would be done after every 2 hrs. The sludge will be collected in the sump of the pumping station from where it will be pumped to the digestion tanks after every 6 hrs.

Capacity of Sludge Pump:

Pumping would be done every 6 hrs.

Assuming the period of operation as 20 minutes for each batch of sludge.

Volume of sludge = 0.0684 ft/capita/day

Ultimate volume of wet sludge

$$= 49000 \times 0.0684$$

$$= 3350 \text{ ft.}$$

$$= \frac{3350 \times 7.48}{4 \times 20}$$

$$= 313 \text{ g.p.m.}$$

c) Laboratory

Laboratory would be provided in the treatment plant to facilitate the following tests:

(i) B.O.D.

(ii) Suspended solids.

(iii) Chlorine residual.

(iv) pH.

(v) Coliform and other essential tests.

The size of the laboratory should be 20' x 20' with a room for the chemist as 10 x 10 ft. In addition to this an office for superintendent, a store and a workshop would be provided.

CHAPTER 8

HYDRAULICS OF THE TREATMENT PLANT

8.1. Derivation of Head Losses

Initial flow "Q" = 1.32 c.f.s.

Ultimate flow "Q" = 3.57 c.f.s.

Flow to units "Q" min.

= 1.32 x 2.5

= 3.30 c.f.s.

Flow to units "Q" max.

= 3.57 x 2.5

= 8.92 c.f.s.

8.1.1 From Primary Sedimentation Tank Outlet Upto the Distribution Box of the Primary Sedimentation Tanks

Q min. = 3.30 c.f.s.

Using two tanks, therefore flow/tank

$$= \frac{3.30}{2}$$

= 1.60 c.f.s.

Q max. = 8.92 c.f.s.

Using all the four tanks, therefore flow/tank

$$= \frac{8.92}{4}$$

$$= 2.23$$

Adopting 12" Q c.i. pipes.

From nomogram[5], the following are obtained.

For Q max., h_f = 0.004 and V = 2.88 ft/sec ---(O.K.)

For Q min., h_f = 0.0021 and V = 2.10 ft/sec ---(O.K.)

(i) Length of pipe = 48 ft.

(ii) Equivalent length for two number - 90° bends[19]

= 2 x 15

= 30 ft.

(iii) Equivalent length for entrance[19]

= 18 ft.

(iv) Equivalent length for exit[19] to D.B.

= 32 ft.

Total effective length = 128 ft.

$\therefore$ $h_{f.max.}$ = 0.004 x 128

= 0.512 ft.

$h_{f.min.}$ = 0.0021 x 128

= 0.270 ft.

8.1.2 From the Distribution Tower of Primary Sedimentation Tanks to the D.B. of Filters

Q min. = 3.30 c.f.s.

and Q max. = 8.92 c.f.s.

Adopting 2 - 15" dia. c.l. pipes for the ultimate condition and 1-15" dia. pipe for the present.

From nomogram(5) the following are obtained:

for Q max., h_f = 0.0045 and V = 3.60 ft/sec -----(Satisfactory)

for Q min., h_f = 0.0026 and V = 2.70 ft/sec -----(Satisfactory)

(i) Length of pipe = 145 ft.

(ii) Equivalent length for entrance = 22 ft.

(iii) Equivalent length for exit = 40 ft.

(iv) Equivalent length for gate valve = 10

Total effective length = 217 ft.

$\therefore$ $h_{f.max.}$ = 0.0045 x 217 = 0.975 ft.

and $h_{f.min.}$ = 0.0026 x 217 = 0.565 ft.

8.1.3 From the Distribution Box of Filters Upto the Top of the Central Column Pipe of the Filters

Q max. $=$ 8.92 c.f.s.

Q min. $=$ 3.30 c.f.s.

Q min per filter $= \frac{3.30}{2}$

$=$ 1.60 c.f.s.

Q max. per filter $= \frac{8.92}{4}$

$=$ 2.23 c.f.s.

Adopting 12" diameter c.l. pipe.

From the nomogram(5) following values for velocities and head losses are obtained:

for Q max., h_f $=$ 0.004 and V = 2.88 ft/sec --(O.K.)

for Q min., h_f $=$ 0.0021 and V = 2.10 ft/sec --(O.K.)

(i) Length of pipe $=$ 49 ft.

(ii) Equivalent length for entrance(19)

$=$ 18 ft.

(iii) Equivalent length for exit

$=$ 32 ft.

(iv) Equivalent length for 1-90° bend

$=$ 15 ft.

(v) Equivalent length for gate valve

$=$ 7

Total effective lengths $=$ 121 ft.

$\therefore\ h_{f.max.}$ = 0.004 x 121

= 0.484 ft.

and $h_{f.min.}$ = 0.0021 x 121

= 0.252 ft.

Vertical pipe length = 6.5 ft.

Residual head on the nozzles

= 4

Therefore max. head loss = 0.456 + 6.5 + 4

= 10.956 ft.

and min. head loss = 0.240 + 6.5 + 4

= 10.740 ft.

8.1.4 From Filters to the Filters Distribution Tower

Adopting 12" diameter C.I. pipe.

Max.flow/filter = 2.23 c.f.s.

Min. flow/filter = 1.60

From the nomogram(5), following values for head losses and velocities are obtained:

for Q max., h_f = 0.004, V= 2.88 ft/sec

---- (Satisfactory)

for Q min., h_f = 0.0021, V= 2.10 ft/sec

---- (Satisfactory)

(i) Length of pipe = 41.5 ft.

(ii) Equivalent length of pipe for entrance

= 18

(iii) Equivalent length of pipe for exit

= 32 ft.

Total length = 91.50 ft.

$\therefore$ $h_{f.max.}$ = 0.004 x 91.50

= 0.366 ft.

and $h_{f.min.}$ = 0.0021 x 91.50

= 0.192 ft.

8.1.5 From Filters Outlet Box Upto the Distribution Box of Final Sedimentation Tanks

Q max. = 8.92 c.f.s.

and Q min. = 3.30 c.f.s.

Adopting 2 - 15 diameter C.l. pipes for the max. development and 1 - 15 diameter C.l. pipe for the present condition:

From the nomogram[5], following values for head losses and velocities are obtained:

for Q max., h_f = 0.0045 and V= 3.60 ft/sec --(O.K.)

for Q min., h_f = 0.0026 and V= 2.70 ft/sec --(O.K.)

(i) Length of pipe = 113 ft.

(ii) Equivalent length of pipe for entrance loss

= 22 ft.

(iii)Equivalent length for gate valve

= 10

(iv) Equivalent length of pipe for exit loss

= 40 ft.

Total effective length of pipe

= 185 ft.

$\therefore$ $h_{f.max.}$ = 0.004 x 185

= 0.74 ft.

and $h_{f.min.}$ = 0.0021 x 185

= 0.388 ft.

8.1.6 From the Distribution Box of Humus Tanks Upto the Centre of the Humus Tanks

Maximum flow/tank = $1.25 \times \frac{3.57}{2}$

= 2.23 c.f.s.

Minimum flow/tank = 1.25 x 1.32

= 1.54 c.f.s.

Adopting 12" diameter C.I. pipes.

From nomogram:-

when Q = 2.23 c.f.s.,

then h_f = 0.004,

and V = 2.88 ft/sec. --- (Satisfactory)

When Q = 1.54 c.f.s,

then h_f = 0.002

and V = 2.05 ft/sec. ---(Satisfactory)

(i) Length of pipe = 45 ft.

(ii) Equivalent length for entrance loss

= 18 ft.

(iii) Equivalent length for exit loss

= 32 ft.

(iv) Equivalent length for 1-90° sweep elbow[19]

= 15 ft.

(v) Equivalent length for gate valve

= 7 ft.

Total effective length

= 117 ft.

$\therefore$ $h_{f.max.}$ = 0.004 x 117

= 0.468 ft.

and $h_{f.min.}$ = 0.002 x 117

= 0.256.

8.2. Effluent Channel Size

Q max. = 2.23 c.f.s.

Q min. = 1.54 c.f.s.

According to Chezy the flow in open channels can be represented by the following expression:

$$Q = A.C\sqrt{mi}$$

where

A = Sectional area of the channel.

m = A/p

= Hydraulic mean depth.

i = Slope of the channel.

C = A constant.

$$= \frac{157.5}{1 + \frac{k}{\sqrt{m}}}$$ (17)

, in which k is also a constant depending on the surface of the channel and has the following values: ( )

Clean sides of wood, brick, stone etc.

$k = 0.29$

Dirty sides of wood, brick, stone etc.

$k = 0.50$

Sides of natural earth = 2.35

As the value of m is not known, therefore a value 100 is assumed.

The value for "i" is taken as 1/80 which is the natural available slope in the ground.

$\therefore$ Q max. $= C\,A\sqrt{mi}$

or 2.23 $= 100 \times \frac{b^2}{2}\sqrt{\frac{b^2}{2\ (b + b)}} \times \frac{1}{80}$

(where "b" is the width of the rectangular section).

or b $= 5\sqrt{31.8}$

$= 2$ ft.

$\therefore d_{max.}$ $= 1$ ft.

and $d_{min.}$ $= 0.69$ ft.

Providing free board $= 8$ inch

$\therefore$ Channel section $= 2' \times 1.67'$

8.3. Derivation of Levels

8.3.1 Levels in the Final Humus Tanks

(i) Levels in the peripherial channel of the tanks:-

Ground elevation at the outlets from the tanks

= 1266.00

Maximum water depth in the outlet channel

= 1.00 ft.

Free board of the channel

Corresponding to the maximum flow in the channel

= 0.67 ft.

Minimum water depth in the channel

= 0.69 ft.

Free board of the channel corresponding to the

minimum flow $= 0.98$ ft.

$\therefore$ Maximum water level in the peripherial channel of

the tank $= 1266 -$ free board.

$= 1266 - 0.67$

$= 1265.33$ ft.

and minimum water level $= 1265.02$ ft.

(ii) Levels in the centre of the tanks:-

Flow over a weir, whose length equals the widtth of the approaching channel, can be represented by the following formula:-

$$Q = 3.33\ L\ H^{3/2} \quad (17)$$

$$\text{or} \quad H = \left(\frac{Q}{3.33 \times L}\right)^{0.67}$$

$$\therefore H_{max.} = \left(\frac{2.23}{3.33 \times \pi \times D}\right)^{0.67}$$

$$= \left(\frac{2.23}{3.33 \times \pi \times 40}\right)^{0.67}$$

$$= 0.066 \text{ ft.}$$

$$\text{and } H_{min.} = \left(\frac{1.54}{3.33 \times \pi \times D}\right)^{0.67}$$

$$= 0.05 \text{ ft.}$$

Allowing drop over the weir

$= 0.5$ ft.

(measured from the sill of the weir to the max. water level in the channel).

$\therefore$ Maximum water level in the tank

= Maximum water level in the peripherial channel + $H_{max.}$ + 0.5

= 1265.33 + 0.066 + 0.5

= 1265.896

Similarly minimum water level

= 1265.02 + $H_{min.}$ + 0.5 + 0.31

= 1265.02 + 0.05 + 0.5 + 0.31

= 1265.88 ft.

8.3.2 Levels in the D.B. of the Humus Tanks

Max. head loss upto the centre of the tank

= 0.44 ft.

Min. head loss upto the centre of the tank

= 0.22 ft.

$\therefore$ Max. water elevation in the distribution box

= 1265.896 + 0.468

= 1266.368

and Min. water elevation in the distribution box

= 1265.88 + 0.256

= 1266.136

8.3.3 Levels in the D.B. of the Filters

Max. head loss from the (outer) D.B. of the filters

to the distribution box of humus tank

= 0.74 ft.

Minimum head loss = 0.388 ft.

$\therefore$ Max. water level in the (outer) D.B. of the filters

= 1266.368 + 0.74

= 1267.108 ft.

and Minimum water level in the (outer) D.B. of the filters

= 1266.136 + 0.388

= 1266.524 ft.

8.3.4 Levels in the Filters

Maximum head loss upto the (outer) D.B. of the filters

= 0.366 ft.

and Minimum head loss = 0.192 ft.

$\therefore$ Maximum level = 1267.108 + 0.366

= 1267.474 ft.

and Minimum level = 1266.524 + 0.192

= 1266.716 ft.

8.3.5 Levels in the (Inner) D.B. to the Filters

Max. head loss upto the filter

= 10.984

Minimum head loss upto the filters

= 10.752

.'. Maximum level in the D.B.

= 1278.458 ft.

and Minimum level = 1277.468 ft.

8.3.6 Water Levels in the (Outer) D.B. of the Primary Sedimentation Tanks

Max. head loss upto the filters D.B.

= 0.975 ft.

Min. head loss upto the filters D.B.

= 0.565 ft.

.'. Max. water level in the D.B.

= Max. water level in the (inner) D.B. of the filters + Max. head loss upto the box of the filters

= 1279.433 ft.

and similarly minimum water level

= 1278.033 ft.

8.3.7 Water Levels in the Primary Tanks

(i) Water levels in the peripherial channel of the primary tanks:-

Max. head loss = 0.512 ft.

Min. head loss = 0.270 ft.

$\therefore$ Max. water level = 1279.433 + 0.512

= 1279.945 ft.

and Min. water level = 1278.033 + 0.270

= 1278.303 ft.

(ii) Water levels at the centre of the tanks:

Allowing drop over the weir

= 0.5 ft.

Now $Q = 3.33\ L\ H^{3/2}$ (17)

$$\therefore H_{max.} = \left(\frac{Q\ max.}{3.33 \times \pi . D}\right)^{0.67}$$

$$= \left(\frac{2.23}{3.33 \times \pi \times 46}\right)^{0.67}$$

= 0.06 ft.

$$\text{and } H_{min.} = \left(\frac{Q\ min.}{3.33 \times \pi \times D}\right)^{0.67}$$

$$= \left(\frac{1.60}{3.33 \times \pi \times 46}\right)^{0.67}$$

= 0.046

Weir sill elevation = Max. water level in the Peripherial channel + drop

= 1279.945 + 0.5 ft.

= 1280.445 ft.

$\therefore$ Max. Water level in the tank

= 1280.445 + 0.06 = 1280.505 ft.

and Min. water level in the tank

= 1280.445 + 0.046 = 1280.491 ft.

CHAPTER 9

LIFTING CUM RECIRCULATION PUMPING STATION

9.1 General

After careful consideration, it is concluded that a pumping station near the treatment works is necessary due to the following reasons.

(1) The trunk, which is to deliver sewage to the treatment plant, is deep enough at the site of the plant due to which it cannot deliver the sewage to the plant by gravity and hence necessitates lifting of the sewage, which in addition would also provide certain amount of operational head for the plant.

(2) The design of the treatment plant is such that it necessitates recirculation of the treated effluent to the treatment plant through the primary sedimentation tank.

Therefore a pumping station of such size and capacity would be provided which would serve the dual purpose of lifting the raw sewage as well as the recirculation of the treated effluent.

Recirculation rate, as discussed earlier would not be constant but vary with the raw sewage at such a rate that when the rate of flow of raw sewage is max. i.e. 2.5 x average flow, the recirculation would be zero and when the raw sewage flow approaches zero then the recirculation would be max. and vice versa. In other words the flow through the treatment plant would be at a constant rate of 2.5 x average flow irrespective of the rates of variation in both. The quality of the effluent,however,would not be effected because these peaks would be attained for a very short duration.

Sewage pumping stations vary from the installation of a pump in the manhole to a ground building. Subsurface structures are undesirable because of the difficulties in operation and maintenance and in corrosion of equipment, therefore a ground building with generous ventilation would be provided. It would contain the following:

(i) Receiving or Wet Well.

(ii) Dry Well.

(iii)Control Room.

(iv) Pumping units and Pressure Sewer.

9.1.1 Receiving or Wet Well

The function of a wet well, at a sewage pumping station, is to act as an equalising basin to minimise the fluctuations of load on the pumps and in addition to act as a suction pit from which the pumps draw sewage. It also facilitates automatic operation. In this report it is provided to act a suction pit in addition to receive the recirculated flow from the hopper of the final sedimentation tank.

The following design criteria would be followed:

(1) It would be partly under ground and partly above the ground.

(2) The size and shape would be such as not to permit the accumulation of deposits that may become septic.

(3) Dead spaces and flate bottoms are to be avoided.

(4) A slope of 1:1 would be provided in the bottom towards the suctions of the pumps.

(5) The well would be divided into two portions for the purpose of cleaning etc.

(6) An emergency outlet would be provided for the purpose of catering with the raw sewage in case all the pumping units fail to work due to one or another reason.

(7) The length of the well would be taken equal to the length of the dry well which in turn would be fixed from pumping units accomodation point of view, while the width would be taken as 5 feet in order to facilitate mannual working in the well if and when required.

9.1.2 Dry Well

Dry well would be provided adjacent to the wet well for the purpose of accomodating the pumping units. Its advantages are summarized as under.

(1) Self priming of the pumps can be achieved.

(2) The possibility of corrosion would be reduced.

(3) Operation and maintenance would be easier.

(4) Danger of flooding of the machinery and equipment would be avoided.

The size of the well is usually fixed from the accomodation point of view of the pumping units and hence depends upon the number of the units.

9.1.3 Control Room

The control room would be provided just above the dry well for the purpose of accomodating the electric motors which would be coupled to the pumps in the dry well by means of vertical shafts. The principal advantages of a vertical shaft installation are the protection of the motor against dampness, ease of accessibility and the saving of building space below ground. However there are certain disadvantages also like: misalignment of the shaft; inaccessibility to the pumps for maintenance and repairs etc., but they are not of a serious nature. Shafts could be kept aligned by using shaft aligners. The size of the control room will be similar to the dry well.

9.1.4 Pumping Units and Pressure Sewer

Pumps are most often installed in a row with all pumps discharging into the same header. However installation of this type necessitates the provision of pumps having approximately the same shut-off head and a rising characteristic from rated capacity to shut-off. Otherwise the pumps which discharge at the highest pressure will do the work even to the extent of shutting down the other pumps.

Non clog, vertical spindle centrifugal pumps coupled with vertical shafts electric motors would be provided with their delivery pipes connected to a common header.

As the flow through the treatment plant would remain constant, therefore the sewage would be pumped at a constant rate. The pumping arrangement would be such that would be installed as and when needed with sufficient provision for stand by units. The design of the units would be done for ultimate conditions and vacant space in the dry well would receive the unit as per requirement.

Pressure sewer would have to be installed to carry the recirculated as well as the raw sewage to the treatment plant through the primary sedimentation tanks. Cast iron pipes would be used becuase of their characteristics which make them suitable for pressure lines.

(Note) Screens and grit channels would be installed in the pumping station.

9.2. Design

9.2.1 Head Discharge Curve for the Pressure Sewer

Present flow "Q"min. = 3.3 c.f.s.

= 1480 g.p.m.

Future flow "Q" max. = 8.92 c.f.s.

= 4,000 g.p.m.

Elevation to which the sewage is to be pumped

= 1280.505

Sewage elevation in the sump = Invert elevation of the trunk + diameter of the trunk - d max. - Z - N - clearance between the bottom of the parshal flume and water surface in the sump = 1256.30

Static head = 1280.505 - 1256.30

= 24.20 ft.

(i) Size of the Sewer

Adopting 18 inches diameter pipe.

$$V_{max.} = \frac{8.92 \times 4 \times 12 \times 12}{\pi \times 18 \times 18}$$

= 5.64 ft/sec. (Satisfactory)

$$V_{min.} = \frac{3.30 \times 4 \times 12 \times 12}{\pi \times 18 \times 18}$$

= 1.87 ft/sec. (Low)

Adopting 15 inches diameter pipe

$$V_{max.} = \frac{8.92 \times 4 \times 12 \times 12}{\pi \times 15 \times 15}$$

= 7.27 ft/sec. (Satisfactory)

$$V_{min.} = \frac{3.30 \times 4 \times 12 \times 12}{\pi \times 15 \times 15}$$

= 2.90 ft/sec. (Satisfactory)

(ii) Water Hammer

Water hammer $= \frac{V_o C}{g}$ (18)

where V_o = Initial velocity

C = Velocity of pressure wave

$$= \frac{4720}{\sqrt{1 + \frac{kD}{Et}}} \quad (18)$$

k = Bulk Modulus

$= 3 \times 10^5$

E = Modulus of Elasticity

D = Diameter of Sewer

g = Acceleration due to gravity

t = Wall thickness of pipe

$$\therefore \quad C = \frac{4720}{\sqrt{1 - \frac{3 \times 10^6 \times 1.25 \times 24}{15 \times 10^6 \times 1}}}$$

$= 3740$

$$h = \frac{V_o C}{g}$$

$$= \frac{7.27 \times 3740}{32.2}$$

$= 850$ ft.

Total head $= 850 + 30$

$= 880$ ft.

$= 382$ p.s.i.

(iii) Losses:

(i) Equivalent length for entrance

= 22 ft.

(ii) Equivalent length for exit

= 40 ft.

(iii) Equivalent length for $4-90^{\circ}$ bends

$= 72$

(iv) Equivalent length for 2 No. Gate and check valves $= 20$

Total equivalent length

$= 154$ ft.

Length of the rising main

$= 346$ ft.

$\therefore$ Total effective length

$= 500$ ft.

Assume a discharge $= 2,000$ g.p.m.

$$Q = \frac{2000}{7.48 \times 60}$$

$$= 4.47 \text{ c.f.s.}$$

$$V = \frac{4.47 \times 4 \times 12 \times 12}{\pi \times 15 \times 15}$$

$$= 3.64 \text{ ft./sec.}$$

$$VD = 3.64 \times 15$$

$$= 54.60$$

$$\frac{E}{D}^{(20)} = 0.0007$$

$$f^{(20)} = 0.0175$$

$$h_f = f\,l\,\frac{V^2}{2gd}$$

$$= \frac{0.0175 \times 500 \times 3.64 \times 3.64}{2 \times 32 \times 1.25}$$

$$= 1.45 \text{ ft.}$$

This loss of head is not high and hence the size of pipe is reasonable.

Assume:

(i) $Q = 0$ cfs, then

$h_f = 0$ ft.

(ii) $Q = 250$ gpm, then

$$V = \frac{250 \times 4}{7.48 \times 60 \times \pi (1.25)^2}$$

$= 0.455$ ft/sec.

$$h_f = f\, l\, \frac{V^2}{2gd} \quad (18)$$

$$= 0.0175 \times 500 \times \frac{0.455 \times 0.455}{2 \times 32 \times 1.25}$$

$= 0.0226$ ft.

(iii) $Q = 500$

$V = 0.91$ ft/sec.

$h_f = 0.0905$ ft.

(iv) $Q = 750$ gpm.

$V = 1.365$ ft/sec.

$h_f = 0.204$ ft.

(v) $Q = 1{,}000$ gpm.

$V = 1.82$ ft/sec.

$h_f = 0.362$ ft.

(vi) $Q = 1250$ gpm.

$$V = \frac{1250 \times 4}{7.48 \times 60 \times \pi\ (1.25)^2}$$

$$= 2.26 \text{ ft.}$$

$$h_f = 0.0175 \times 500 \times \frac{2.26 \times 2.26}{2 \times 32 \times 1.25}$$

$$= 0.54 \text{ ft.}$$

(vii) $Q = 1500$ gpm.

$V = 2.73$ ft/sec.

$h_f = 0.81$ ft.

(viii) $Q = 2000$ gpm.

$V = 3.64$ ft/sec.

$h_f = 1.45$ ft.

(ix) $Q = 2500$ gpm

$V = 4.55$ ft/sec.

$h_f = 2.24$ ft.

(x) $Q = 3000$ gpm.

then

$V = 5.46$ ft/sec.

and

$h_f = 3.24$ ft.

(xi) Q = 3500 gpm.

then

V = 6.36 ft.

and

h_f = 4.42 ft.

(xii) Q = 4000 gpm.

then

V = 7.27 ft/sec.

and

h_f = 5.77 ft/sec.

TABLE 9.1

SUMMARY OF DISCHARGES AND HEAD LOSSES IN

A 15 INCHES DIAMETER PIPE

Discharge (gpm)	Head loss (ft)	Static head (ft)	Total head (ft)
250	0.023	24.20	24.223
500	0.091	24.20	24.291
750	0.204	24.20	24.404
1000	0.362	24.20	24.562
1250	0.540	24.20	24.740
1500	0.810	24.20	25.010
2000	1.450	24.20	25.650
2500	2.240	24.20	26.440
3000	3.240	24.20	27.440
3500	4.420	24.20	28.620
4000	5.770	24.20	30.970

9.2.2 Characteristic of Pumps

The characteristic curves for the pumps are shown in figure 9.1 and 9.2.

Curve No. 4305-4 is for the smaller pump, whose details are as under:-

(i) Size 6 x 8 x 4M and 8 x 8 x 4½ .

(ii) Impeller diameter	=	14½ inches.
(iii) Efficiency	=	70%
(iv) R.P.M.	=	730
(v) Discharge	=	1480 gpm at 24.9 ft and
(vi) H.P.	=	9.5
N_s	=	$\frac{N \sqrt{Q}}{H^{0.75}}$
	=	2500 r.p.m.

For the present two pumps P_1 would be installed. One of them will work regularly while the 2nd would be used as a stand by. Afterwards when the demand is increased two more P_1 and a larger pump P_2 (figure 8.2) would be installed. Ultimately P_2 would work regularly while the 4-P_1 pumps would be utilized as standby.

P_1 working alone would pump 1480 gpm.

2-P_1 working in parallel would pump = 2600 gpm

3-P_1 working in parallel would pump = 3350 gpm

$4\text{-}P_1$ working in parallel would pump = 4000 gpm

P_2 working alone would pump = 4000 gpm.

Similar pumping arrangement would be utilized for the recirculating flow into the sump. These pumps would be comparatively of lower heads and their operation would be controlled through the float arrangement in the sump.

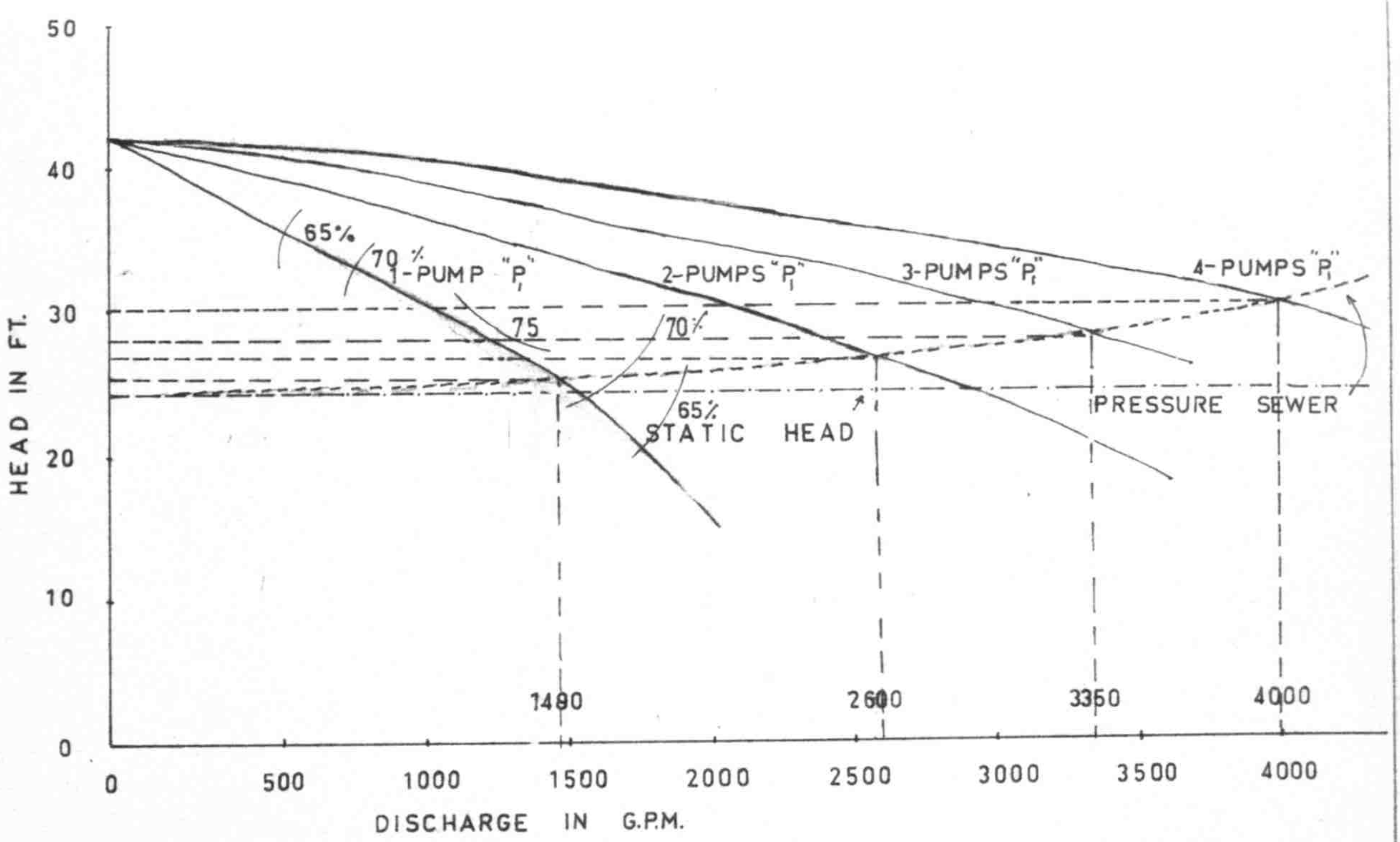


FIG. 9·1	CHARACTERISTIC CURVES OF "P_1" PUMPS AND CAPACITY HEAD CURVE OF PR. SEWER.

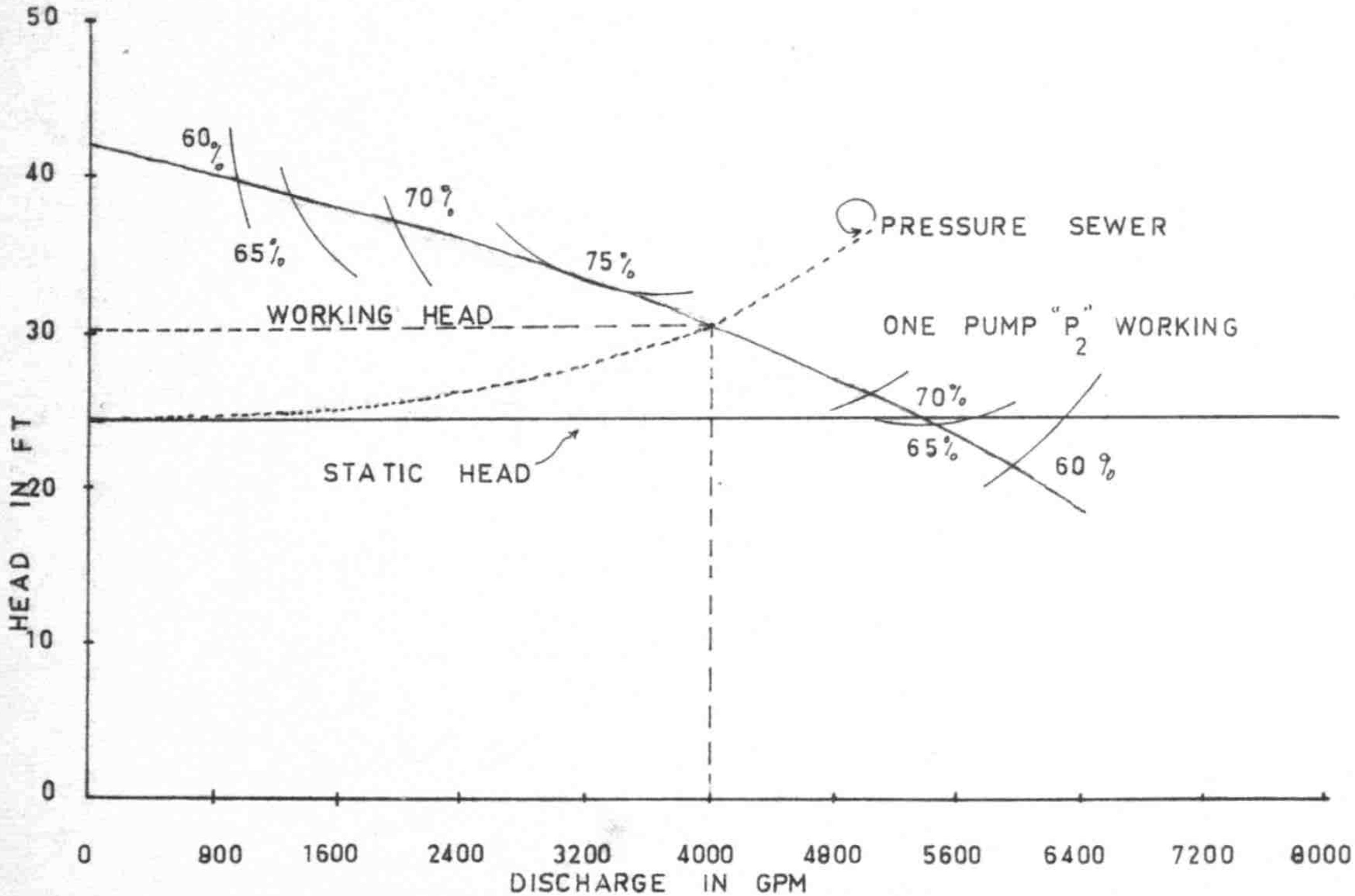


FIG 9.2	CHARACTERISTIC CURVE OF PUMP "P_2" & CAPACITY HEAD CURVE OF PRESSURE SEWER

CHAPTER 10

COST ESTIMATE

It is necessary to have an approximate cost estimate of the project in order to frame an idea regarding the financial aspect involved in the execution. This will also help the authorities to allocate funds and invite tenders in advance.

The estimate of quantities and the cost of the various items of the project is therefore given below.

The scheme will be completed in two phases of 25 years each, consequently the costs will be estimated for the two phases separately and finally combined under abstract of cost.

10.1. First Stage

10.1.1 Sewerage Network

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
1	(a) Provision and lay-out of 8" concrete pipe sewer at invert depth upto 6 feet.	5900	Rft	15/-	88,500/-

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
	(b) Provision and lay-out of 8" concrete pipe sewer at invert depth above 6 feet.	6750	Rft	18/-	121,500/-
2	Provision and layout of 10" concrete pipe sewer at invert depth over 6 feet.	9300	Rft	20/-	186,000/-
3	Provision and layout of 12" concrete pipe sewer at invert depth over 6 feet.	2400	Rft	24/-	57.600/-
4	Provision and layout of 15" R.c.c. pipe sewer at invert depth above 6 feet.	1950	Rft	28/-	54,600/-
5	Provision and layout for 21" R.c.c. pipe sewer at invert depth above 6 feet.	4725	Rft	35/-	157,375/-
6	(a) Construction of man-holes over 8" pipe sewer upto 6 feet depth.	23	each	300/-	6,900/-
	(b) Construction of man-holes over 8" pipe sewer above 6 feet depth.	30	each	375/-	11,250/-
7	Construction of manholes over 10 in pipe sewer above 6 feet depth.	39	each	450/-	17,550/-
8	Construction of manholes over 12 in pipe sewer above 6 feet depth.	15	each	500/-	7,500/-

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
9	Construction of man-holes over 15 in pipe sewer above 6 feet depth.	8	each	550/-	4,400/-
10	Construction of man-holes over 21" pipe sewer above 6 feet depth.	16	each	700/-	11,200/-
11	General Services	-	lump sum	50,000/-	50,000/-
			Total		774,375/-

10.1.2 Treatment Plant

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
1	Construction of Primary sedimentation tanks i/c Provision of all mecha-nical equipment and all other appurtenances.	2	each	40,000/-	80,000/-
2	Construction of trick-ling filters i/c provi-sion of filter media, rotary distributors and all other appurtenances.	2	each	70,000/-	140,000/-
3	Construction of final sedi-mentation tanks i/c provi-sion of all mechanical equipment and all other appurtenances.	1	each	25,000/-	25.000/-

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
4	(a) Construction of primary sludge digestion tanks i/c provision of mixing mechanism and all other appurtenances.	1	each	30,000/-	30,000/-
	(b) Construction of secondary sludge digestion tanks with all required appurtenances.	1	each	20,000/-	20,000/-
5	Provision of recirculation/lifting pumping sets complete with electric motors pressure sewer and all other appurtenances.	2	each	7,500/-	15,000/-
6	Construction of sludge pumping station including cost of 2 pumping sets complete with electric motors, pressure sewer, and all other required appurtenances.	1	each	25,000/-	25,000/-
7	Construction of sludge drying beds including the cost of all piping and required appurtenances.	25	each	1,000/-	25,000/-
8	Construction of distribution boxes for primary and final sedimentation tanks and trickling filters i/c cost of connecting pipes.	3	each	3,000/-	9,000/-

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak. Rupee
9	Construction of wet well, dry well, control room, screens, grit channels, partial flume, stair cases, gates and all other appurtenances.	1	each	10,000/-	10,000/-
10	Construction of chlorination plant with all equipment.	1	each	15,000/-	15,000/-
11	Construction of Laboratory with allthe necessary equipment.	1	each	7,500/-	7,500/-
12	Construction of store.	1	each	6,000/-	6,000/-
13	Provision of wire fencing around the treatment plant site i/c cost of gates etc.	-	lump sum	7,500/-	7,500/-
14	Cost of land.	5	acre	2,000/-	10,000/-
15	Development of land i/c construction of internal roads.	-	lump sum	15,000/-	15,000/-
16	General Services.	-	lump sum	10,000/-	10,000/-
			Total		450,000/-

10.2. Second Stage

10.2.1 Treatment Plant

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak.Rupee
1	Construction of primary sedimentation tanks i/c provision of all mechanical equipment and all other appurtenances.	2	each	40,000/-	80,000/-
2	Construction of trickling filters i/c provision of filter media, rotary distributors and all other appurtenances.	2	each	70,000/-	140,000/-
3	Construction of final sedimentation tanks i/c provision of all mechanical equipment and all other appurtenances.	1	each	25,000/-	25,000/-
4	(a) Construction of primary sludge digestion tanks i/c provision of mixing mechanism and all other appurtenances.	1	each	30,000/-	30,000/-
	(b) Construction of secondary sludge digestion tanks with all required appurtenances.	1	each	20,000/-	20,000/-
5	Provision of recirculation/ lifting pumping sets complete with electric motors, piping and other appurtenances.	3	each	6,000/-	18,000/-

Item No.	Description	Estimated quantities	Unit	Rate Pak. Rupee	Total Cost Pak.Rupee
6	Provision of sludge pumping sets including cost of electric motors complete with all piping and other required appurtenances.	1	each	6,000/-	6,000/-
7	Construction of sludge drying beds including the cost of all piping and required appurtenances.	25	each	1,000/-	1,000/-
8	Construction of chlorination plant with all equipment.	1	each	15,000/-	15,000/-
9	General Services.	-	lump sum	20,000/-	20,000/-
			Total		364,000/-

10.3. Abstract of Costs

Item	Cost in Pak. Rupee
Sewerage Network	774,375/-
Treatment plant, 1st Stage	450,000/-
Treatment plant, 2nd Stage	364,000/-
	1,588,375/-
	say RS 1,589,000/-

Therefore the cost per capita comes to be Rs 36/- which is equivalent to approximately $ 8.0 per person.

SUMMARY

The idea behind this report is to present a preliminary design of a sewerage scheme for Bannu, W. Pakistan. Since the waste at present is disposed off in an insanitary manner, therefore it is necessary to provide the city with a sewerage system in order to do away with the possibilities of spreading of disease etc.

The design period of the project has been taken equal to 50 years and would be effective from 1967 till 2017. The present estimated population of the city is 26,000 persons, while that for the ultimate condition it would be 44,000. A conventional type of treatment has been adopted after careful consideration of all the possible methods of treatment.

The units of the plant would be constructed in two phases. The first phase would be effective for the first 25 years, while the 2nd phase would be for the remaining 25 years of the design period.

Sewers would be laid for the maximum expected development of the city within the design period for the

reasons discussed earlier. Concrete pipes would be used for all sewers other than those under pressure. Efforts have been to avoid pumping of the sewage as far as possible and practicable at the cost of deep excavation for sewers at some places. However, complete elimination of pumping was impossible and therefore a pumping station has been provided near the treatment plant which would not only facilitate lifting of the sewage but would also control the recirculation of the sewage from the hopper of the final sedimentation tank i.e. recirculation pumping units would also be installed in the sewage lifting station. Pumping units of the station would be installed as and when they would be required.

The effluent of the plant, after chlorination, would be utilized for broad irrigation.

The total cost of the project is Rs 1,589,000/- which is approximately equal to $ 8 per capita and seems to be reasonable on account of the availability of cheap labor.

1/3rd of the cost of the project would be met by the municipality from its own resources while the remaining 2/3rd will be given as grant in-aid to the municipality by the Government of W. Pakistan.

REFERENCES

(1) Rainfall Data Collected by Irrigation Dept. Bannu Division (unpublished).

(2) Fair and Geyer, Water Supply and Waste Water Disposal Second Printing. New York: John Willey & Sons Inc., 1952.

(3) Census Report for Bannu, 1961.

(4) Frederick, E.M. "Population Forecasting by Sanitary Engineers" Journal of the Sanitary Engineering Division, A.S.C.E. Vol. 90, August 1964.

(5) Steel Water Supply and Sewerage, Fourth Edition, New York. McGraw-Hill Book Company, Inc.

(6) Prof. S. El-Khuri, Lectures compiled by Prof. Khuri on the subject of Waste Disposal; School of Engineering, A.U.B., 1960.

(7) Public Health Engineering Dept. Feasibility Report on Bahawalpur Sewerage and Drainage Project.

(8) Nelson Leonard Nemerow, Theories and Practices of Industrial Waste Treatment. Addison-Wesley Publishing Company, Inc.

(9) Willem Rudolf, Industrial Wastes, Their Disposal and Treatment.

(10) Edmund B. Besselievve, Industrial Waste Treatment, McGraw-Hill Book Company, Inc.

(11) Metcalf, Sewerage and Sewage Disposal, New York. McGraw-Hill Book Company, 1930.

(12) Sawyer, Chemistry for Sanitary Engineers, McGraw Hill Book Company, 1960.

(13) Babbit, Sewerage and Sewage Treatment. New York: John Wiley & Sons, Inc., 1952.

(14) A.S.C.E. & I.N.P.C.F. Manual on Design and Construction of Sanitary and Storm Water Sewer, (Washington D.C. 1960).

(15) McKee and Wolf, Water Quality Criteria, State Water Quality Control Board. Publication No. 3A.

(16) Catalogue on How to Get Top Performance from your Sewage Treatment Plant.

(17) Lewitt, Hydraulics and Fluid Mechanics. 10th edition. London, Sir Isaac Pitman & Sons, Ltd.

(18) R.L. Daugherty and A.C. Ingersoll, Fluid Mechanics with Engineering Applications.

(19) Seelye Elwyn E. Data Book for Civil Engineers.

(20) Victor Streeter, Fluid Mechanics, 3rd edition, New York, McGraw-Hill Book Company.

BANNU SEWERAGE SCHEME	
TOPOGRAPHY	
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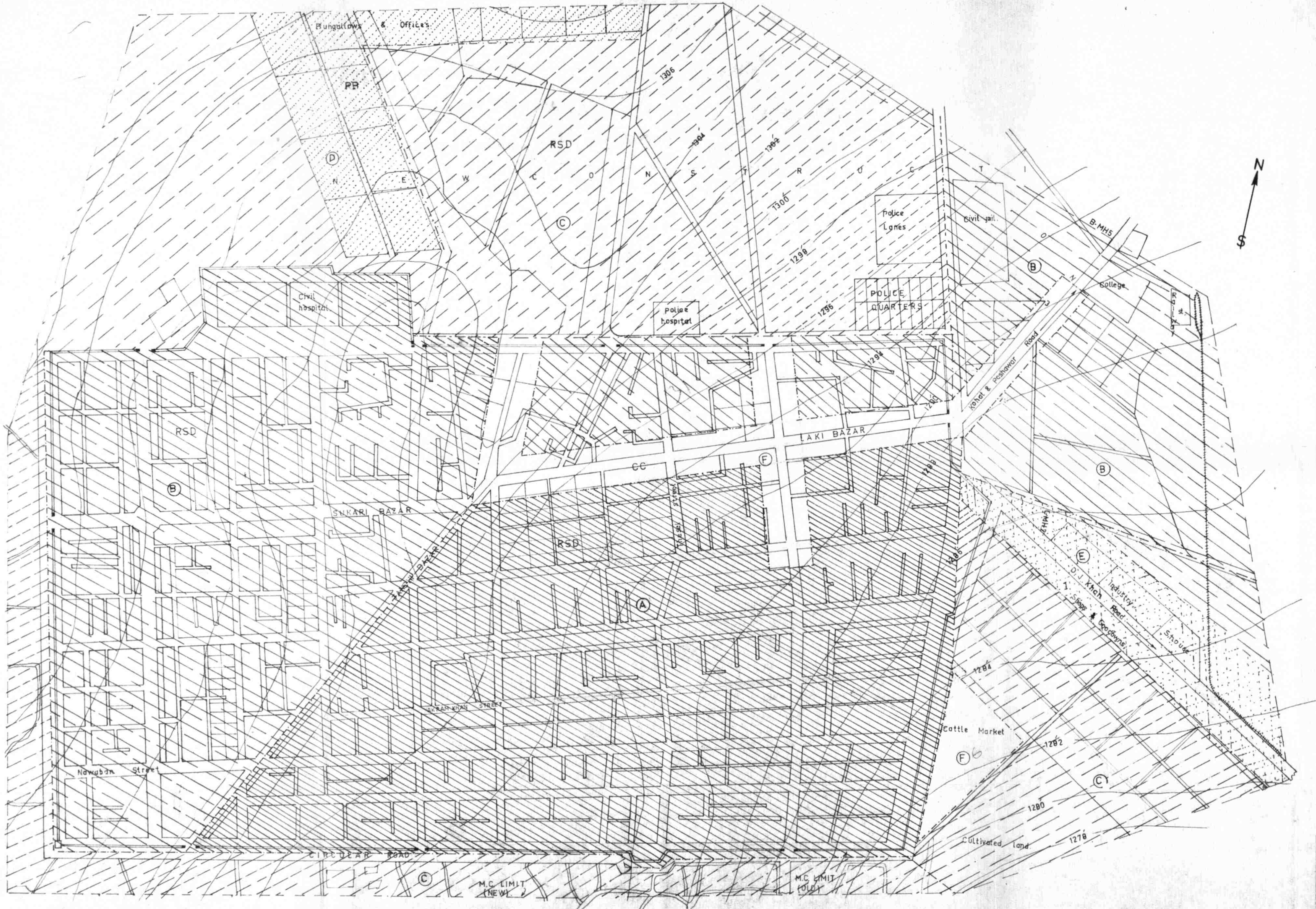
ENGINEER N.H. AFRIDI.	DATE MAY 1967



LEGEND

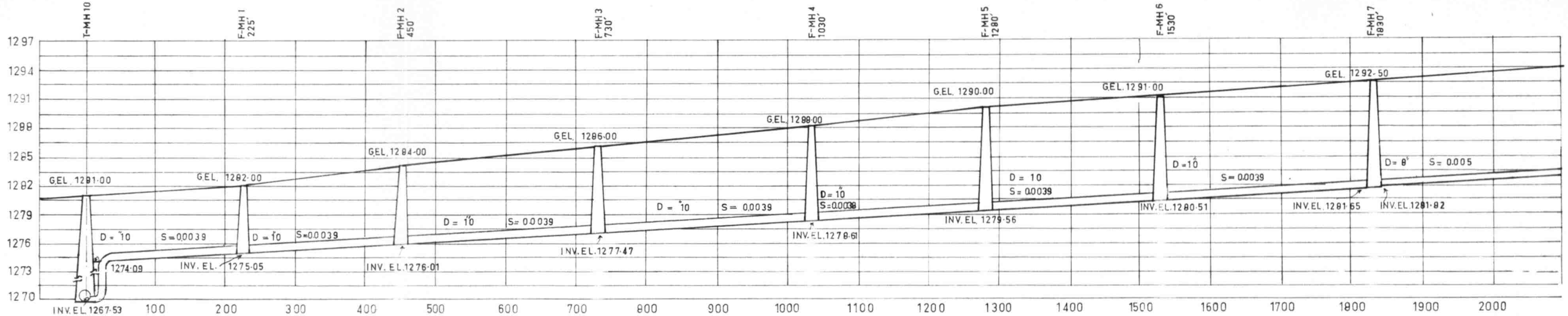
	PRESENT POP. PERSONS/ACRE	FUTURE POP. PERSONS/ACRE
(A)	51	70
(B)	40	65
(C)	19	45

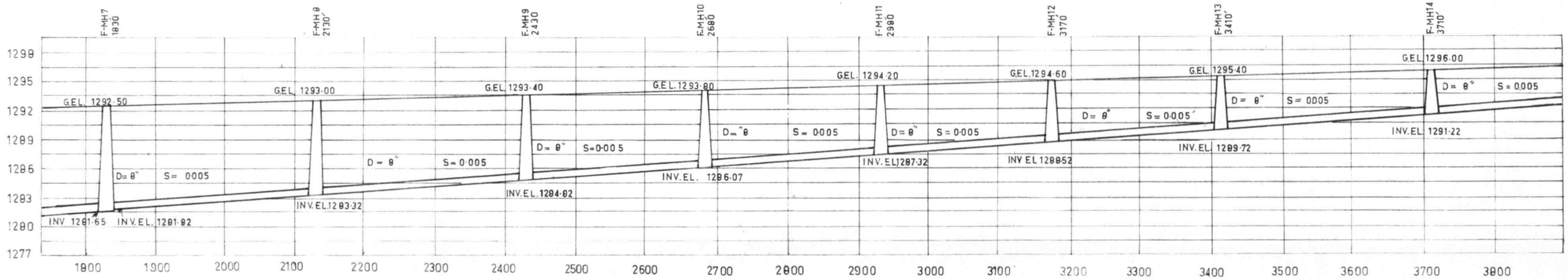
BANNU SEWERAGE SCHEME

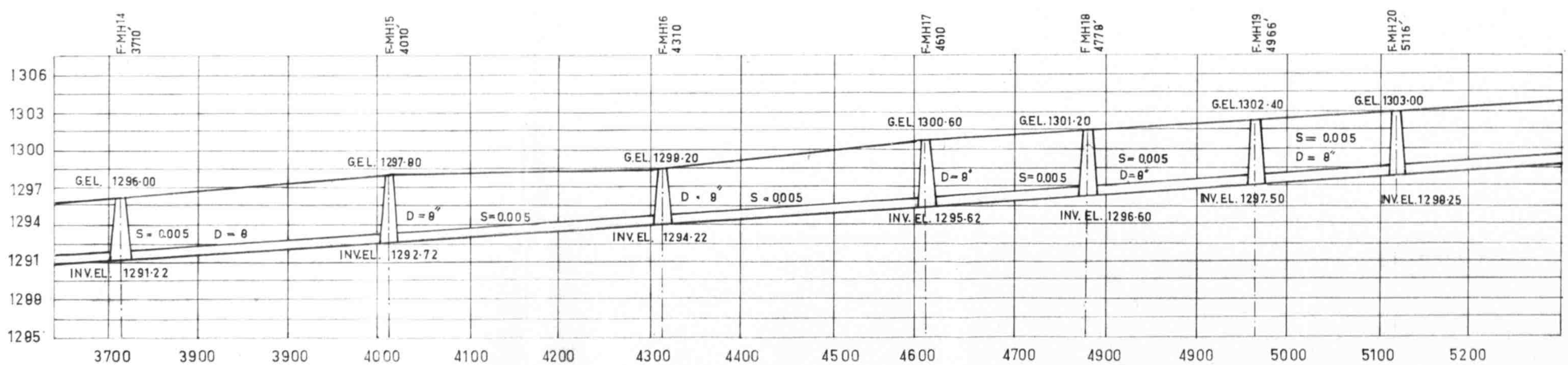
PRESENT & FUTURE POPULATION DENSITIES

SCALE :- 1" = 300'	DRAWING NO. 2
ENGINEER N.H. AFRIDI.	DATE MAY 1967

BANNU SEWERAGE SCHEME	
MAIN SEWERS & TRUNK LAYOUT	
SCALE :- 1" = 300'	DRAWING NO. 3
ENGINEER N.H.AFRIDI.	DATE MAY 1967







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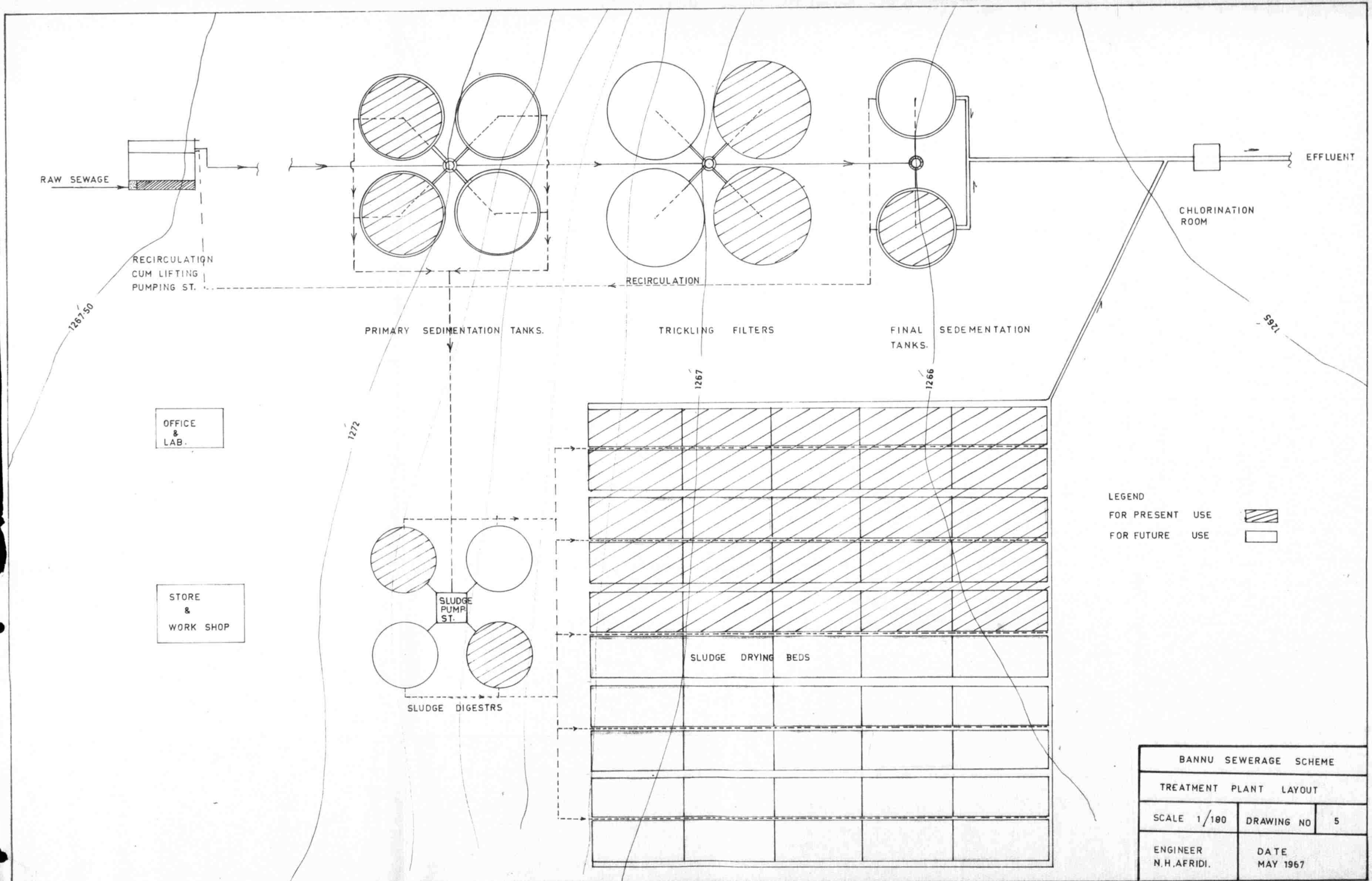
ENGINEER N·H·AFRIDI.	DATE MAY 1967



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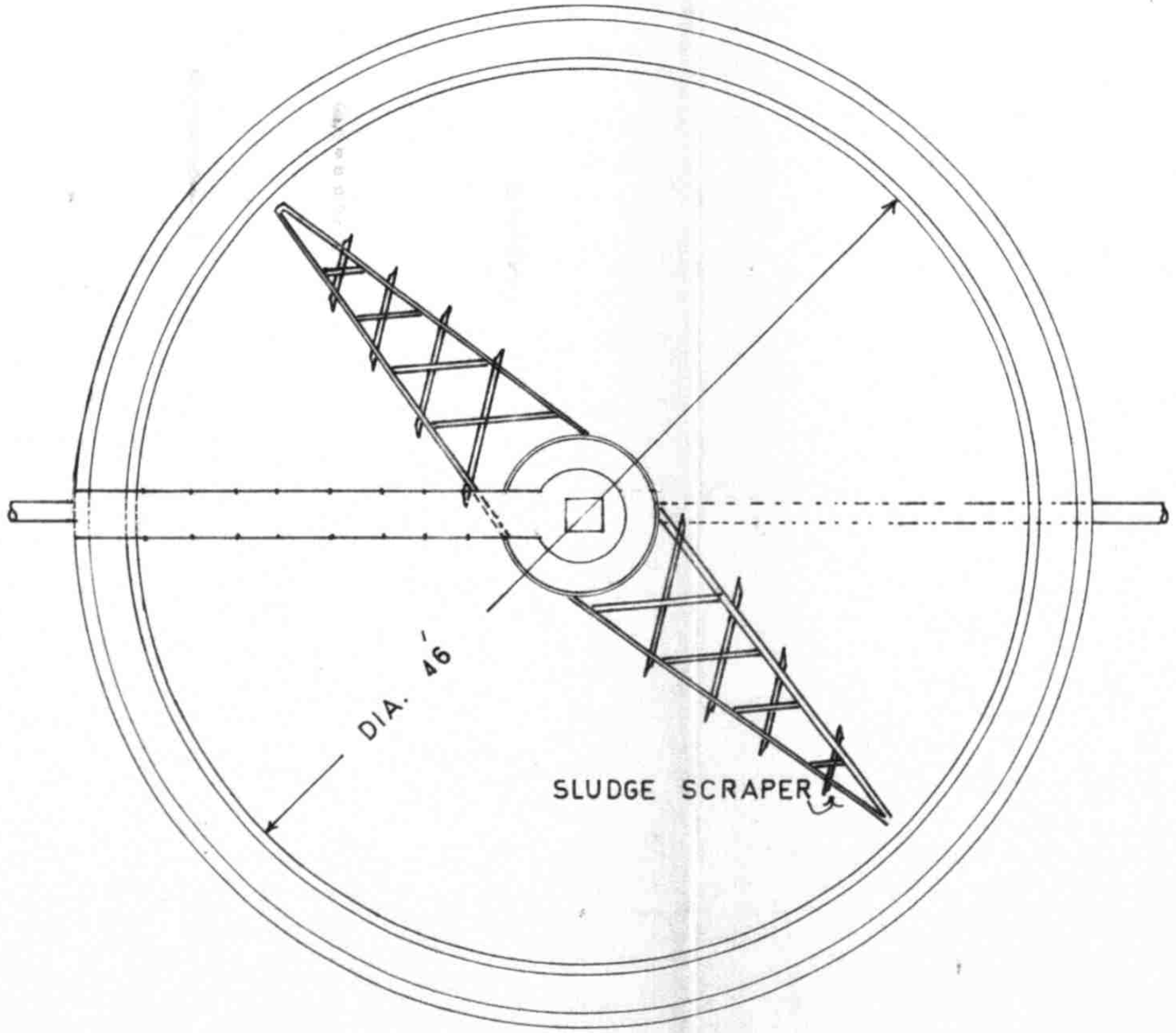
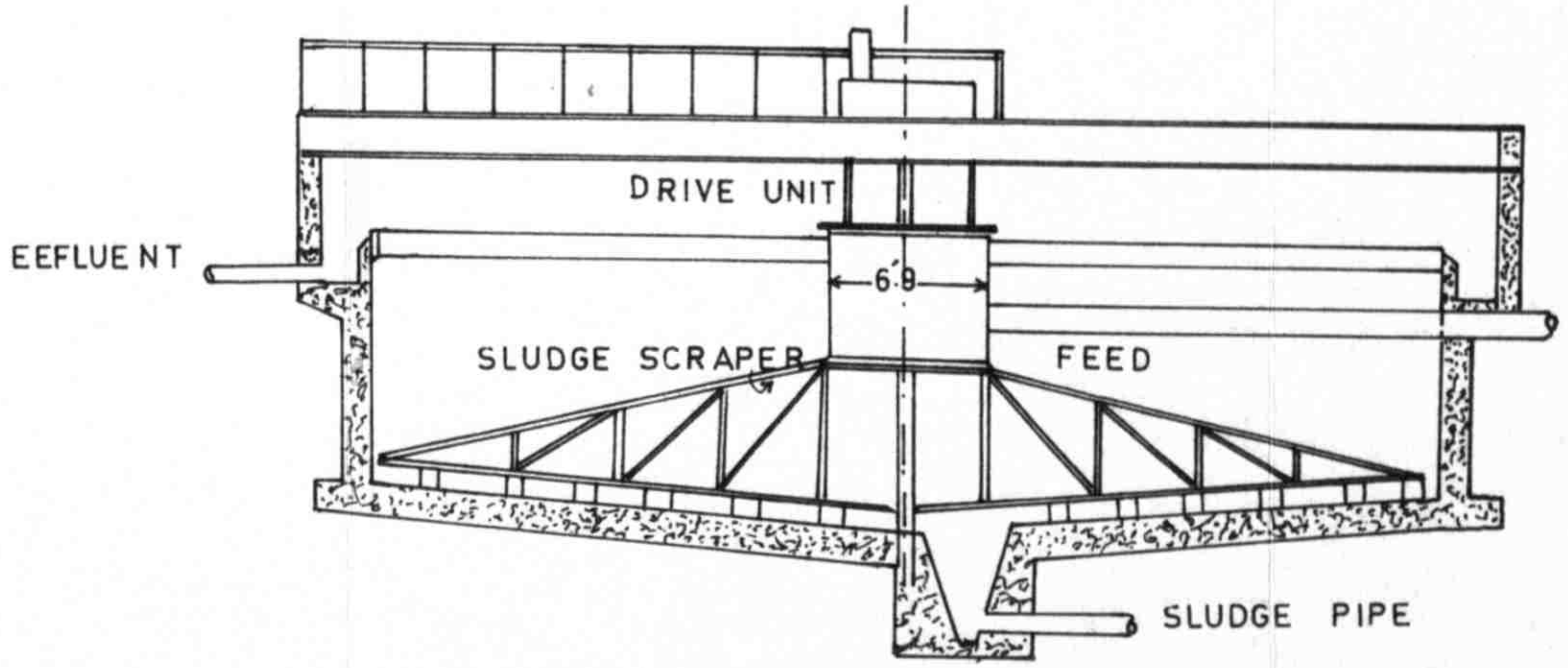
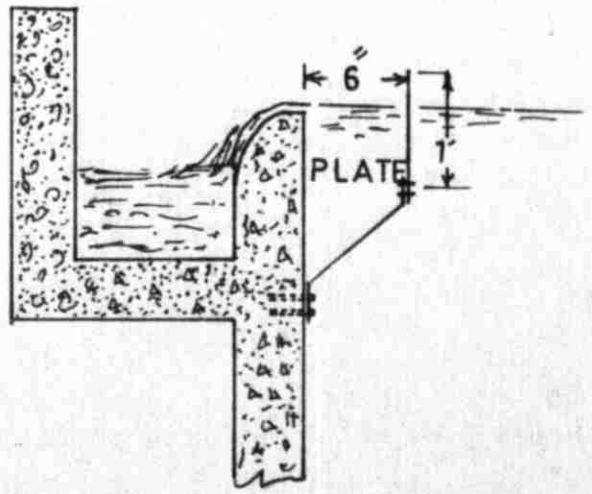


PLAN

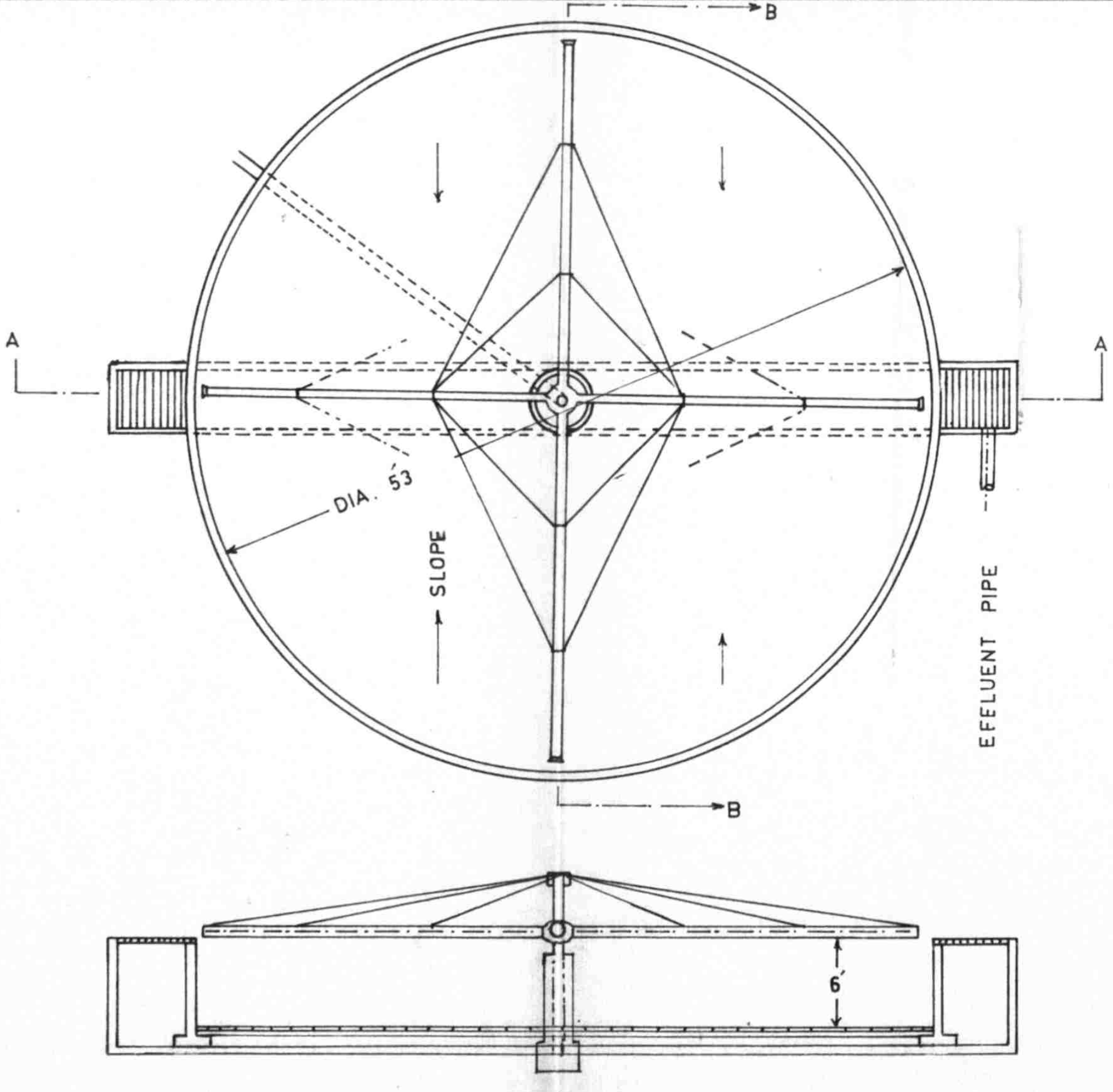


SECTIONAL ELEVATION

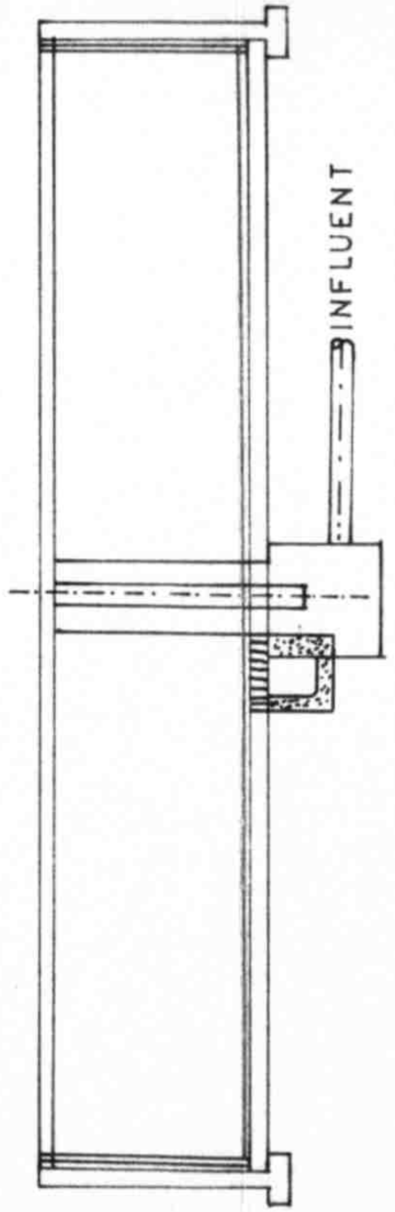


DETAIL OF SCUM BAFFLE
AND WEIR

BANNU SEWERAGE SCHEME	
PRIMARY SEDEMENTATION TANK.	
SCALE 1/100	DRAWING NO 6
ENGINEER N H AFRIDI	DATE MAY 1967



SECTION A-A



SECTION B-B

BANNU SEWERAGE SCHEME		
TRICKLING FILTER		
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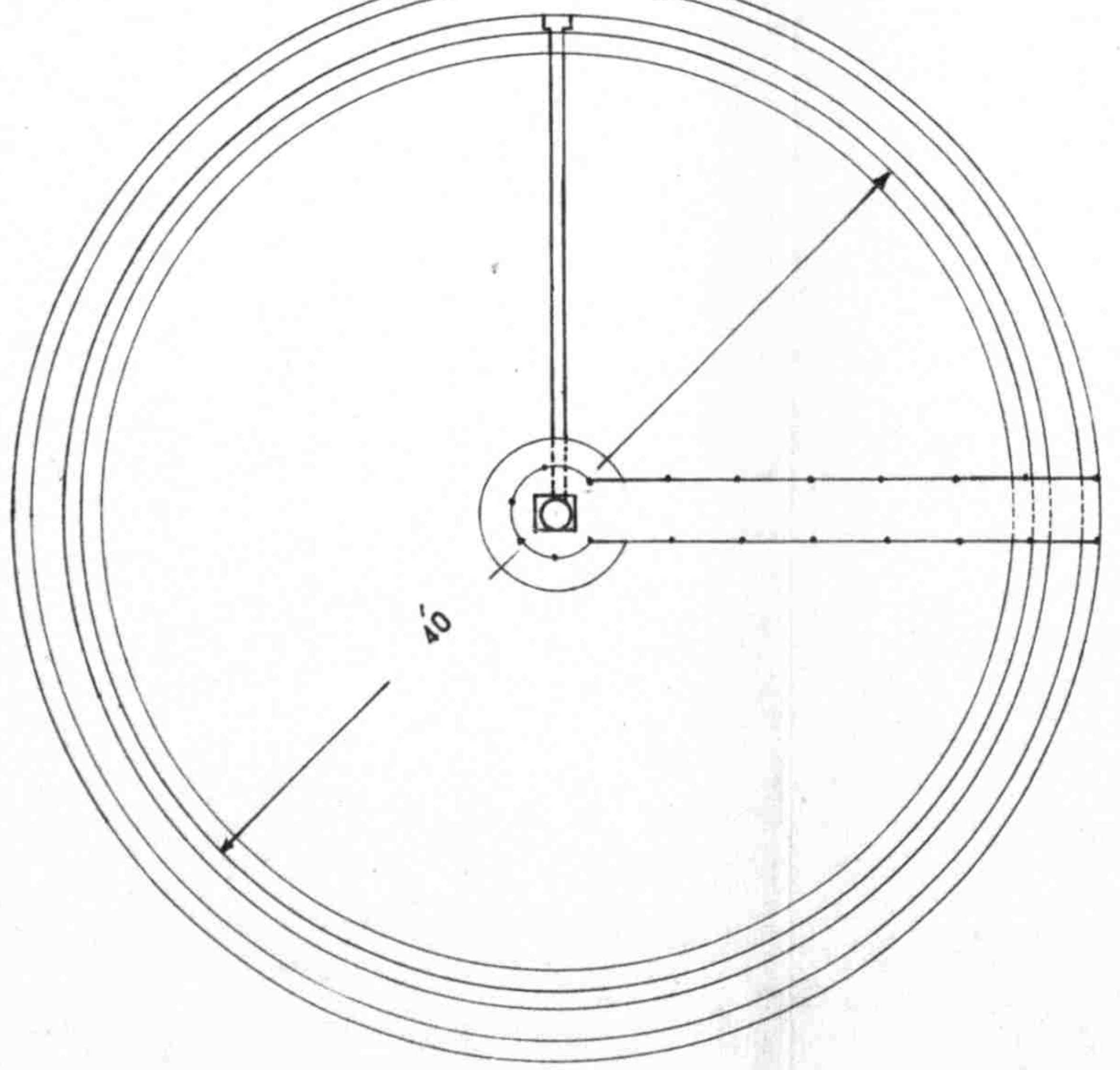
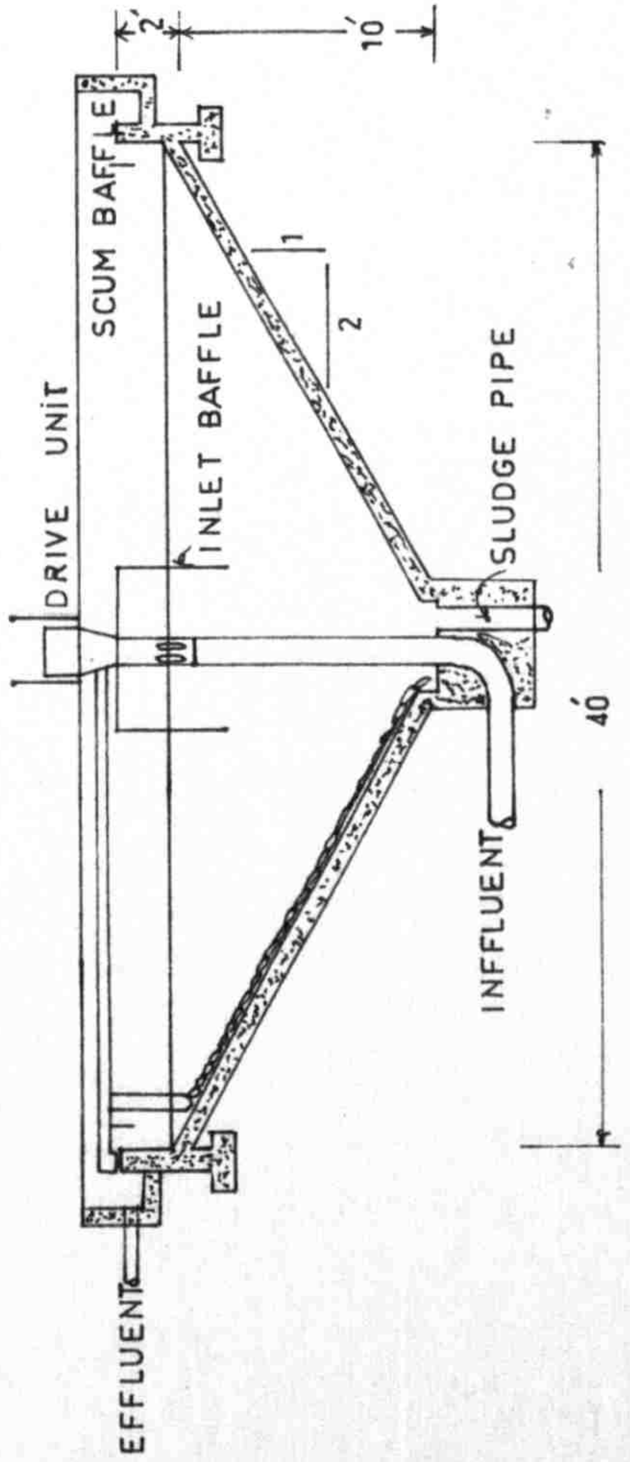
ENGINEER N.H. AFRIDI.	MAY 1967	



PLAN



SECTIONAL ELEVATION

BANNU SEWERAGE SCHEME		
FINAL SEDEMENTATION TANK.		
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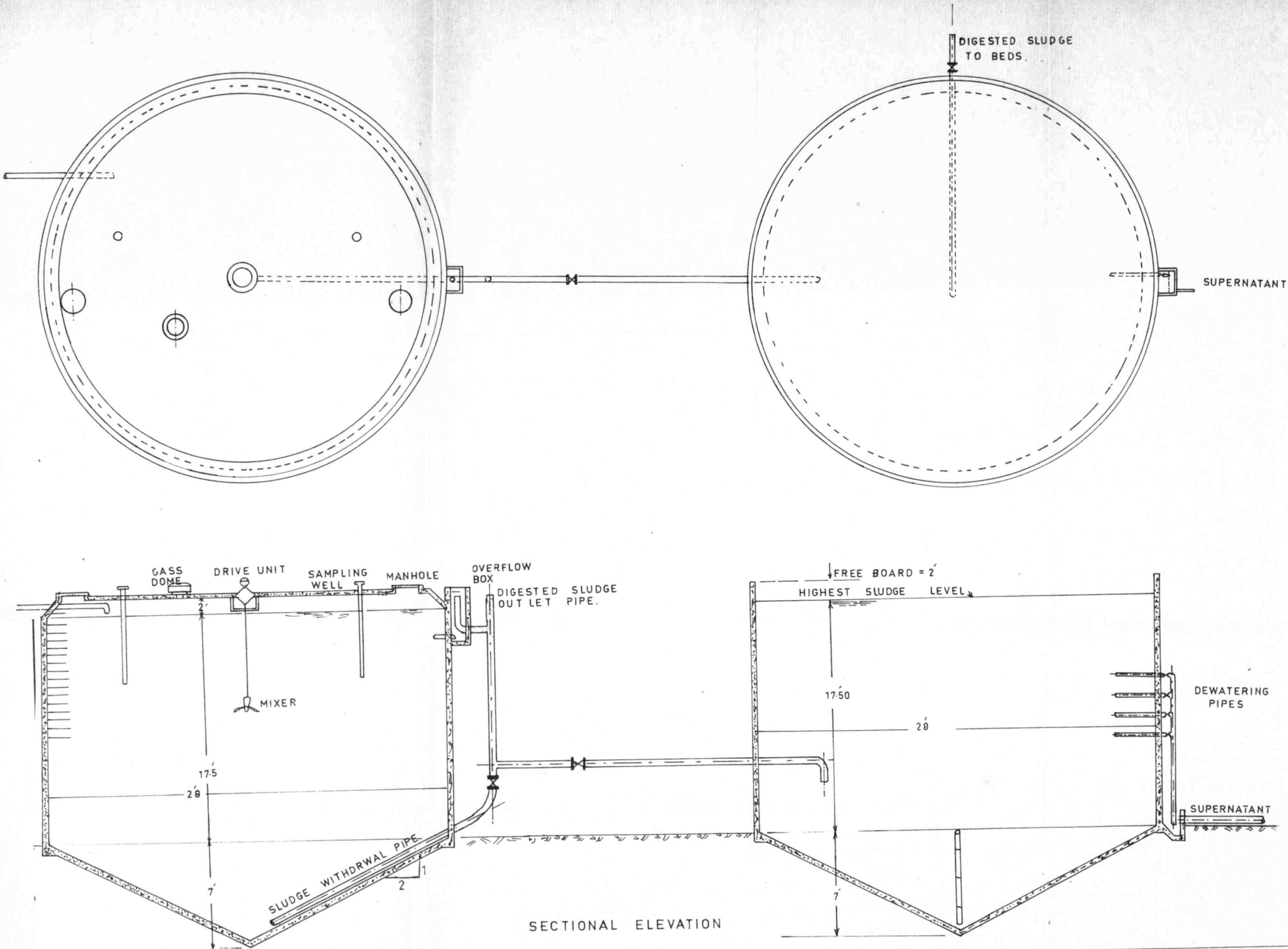

SECTIONAL ELEVATION

BANNU SEWERAGE SCHEME		
SLUDGE DIGESTION TANKS		
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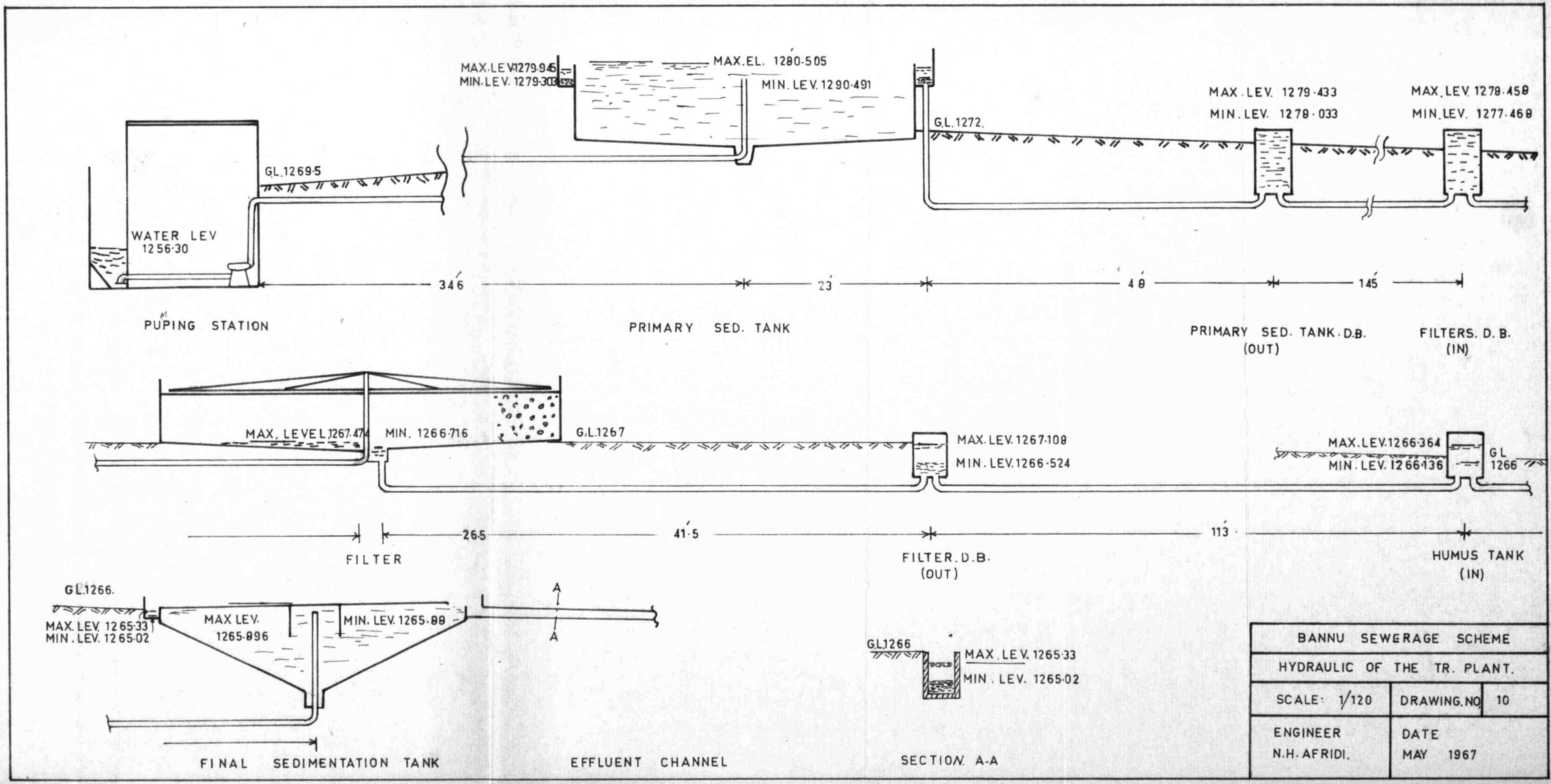
ENGINEER N. H. AFRIDI	DATE MAY 1967	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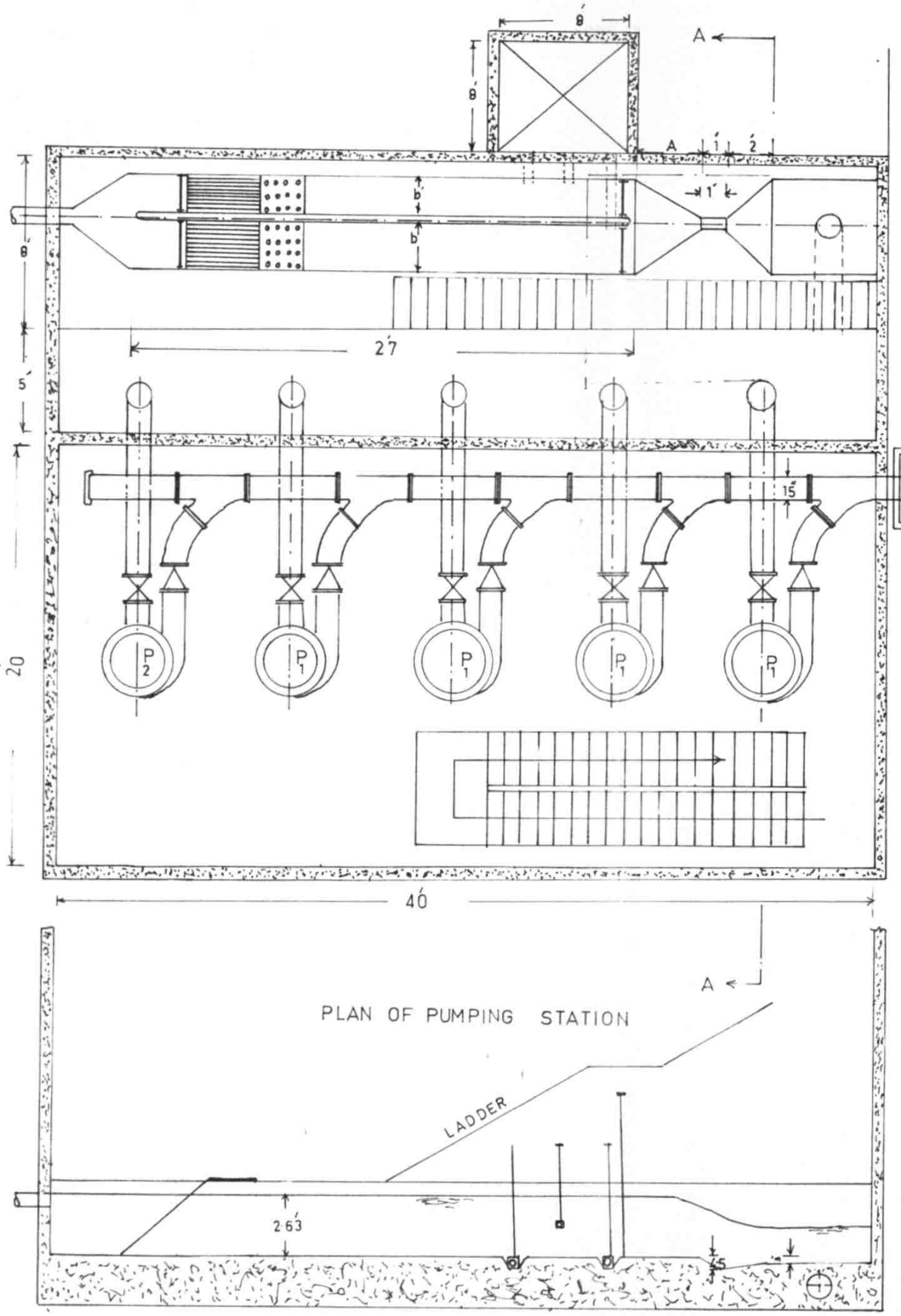
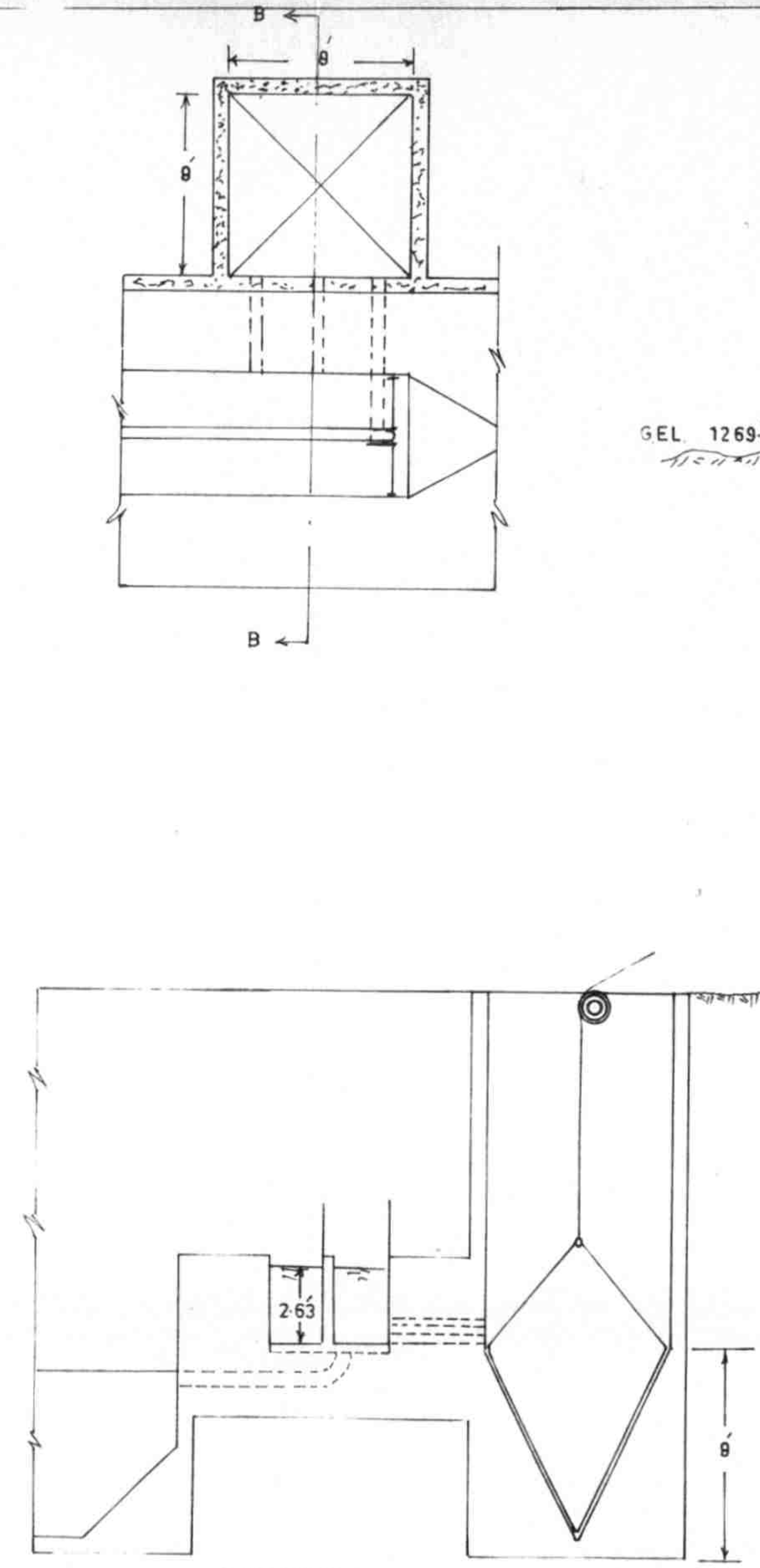
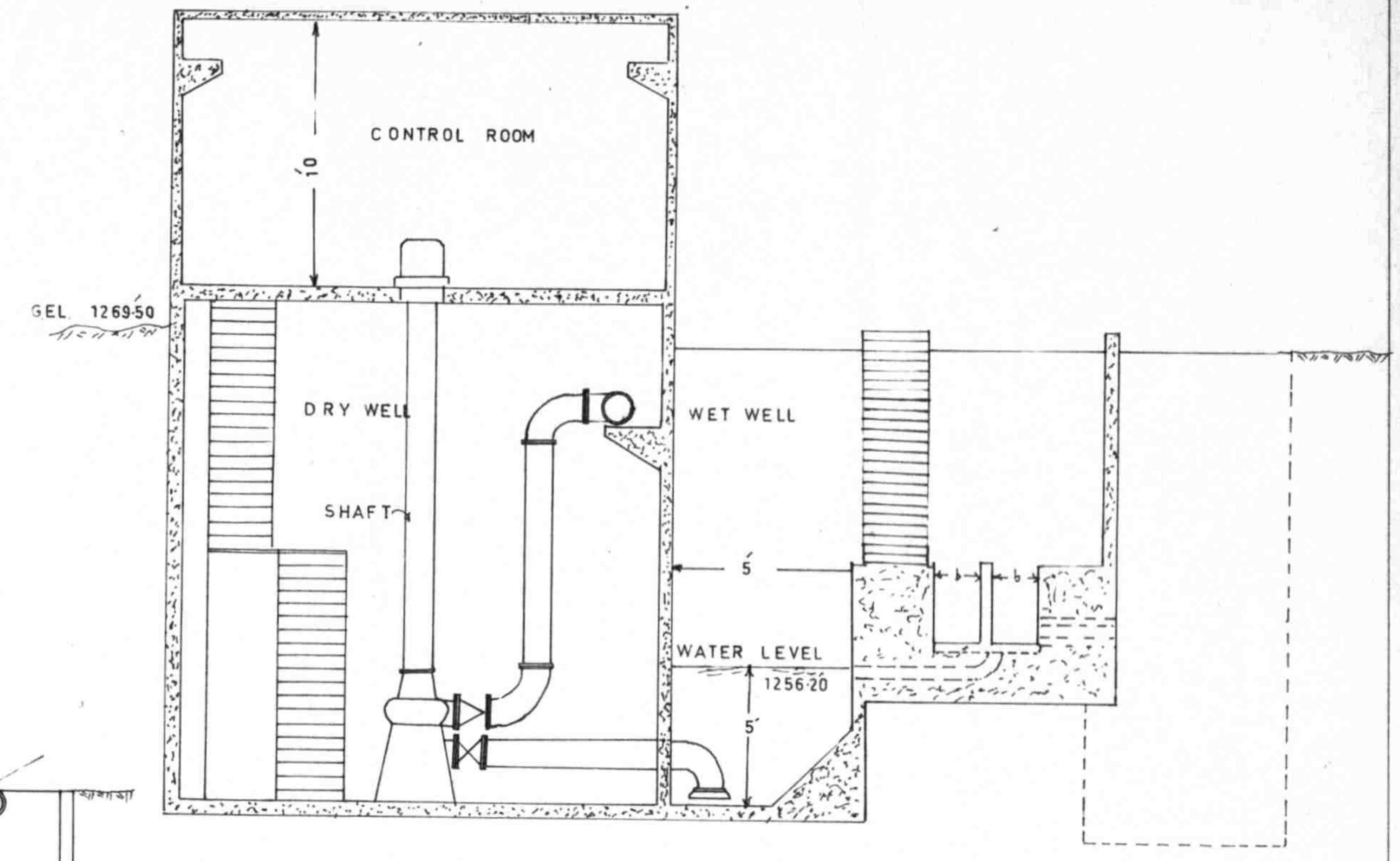
MIN.LEV. 1279.303
MAX.EL. 1280.505
MIN. LEV. 1290.491
G.L.1272.
MAX. LEV. 1279.433
MIN. LEV. 1278.033
MAX. LEV. 1278.458
MIN. LEV. 1277.468
GL.1269.5
WATER LEV
1256.30
346
23
48
145
PUPING STATION
PRIMARY SED. TANK
PRIMARY SED. TANK. D.B.
(OUT)
FILTERS. D. B.
(IN)
MAX. LEVEL 1267.474
MIN. 1266.716
G.L.1267
MAX. LEV. 1267.108
MIN. LEV. 1266.524
MAX. LEV. 1266.364
MIN. LEV. 1266.136
GL
1266
26.5
41.5
113
FILTER
FILTER. D.B.
(OUT)
HUMUS TANK
(IN)
GL.1266.
MAX. LEV. 1265.33
MIN. LEV. 1265.02
MAX LEV.
1265.896
MIN. LEV. 1265.88
A
A
G.L.1266
MAX. LEV. 1265.33
MIN. LEV. 1265.02
FINAL SEDIMENTATION TANK
EFFLUENT CHANNEL
SECTION A-A
BANNU SEWERAGE SCHEME
HYDRAULIC OF THE TR. PLANT.
SCALE: 1/120
DRAWING. NO 10
ENGINEER
N.H. AFRIDI.
DATE
MAY 1967



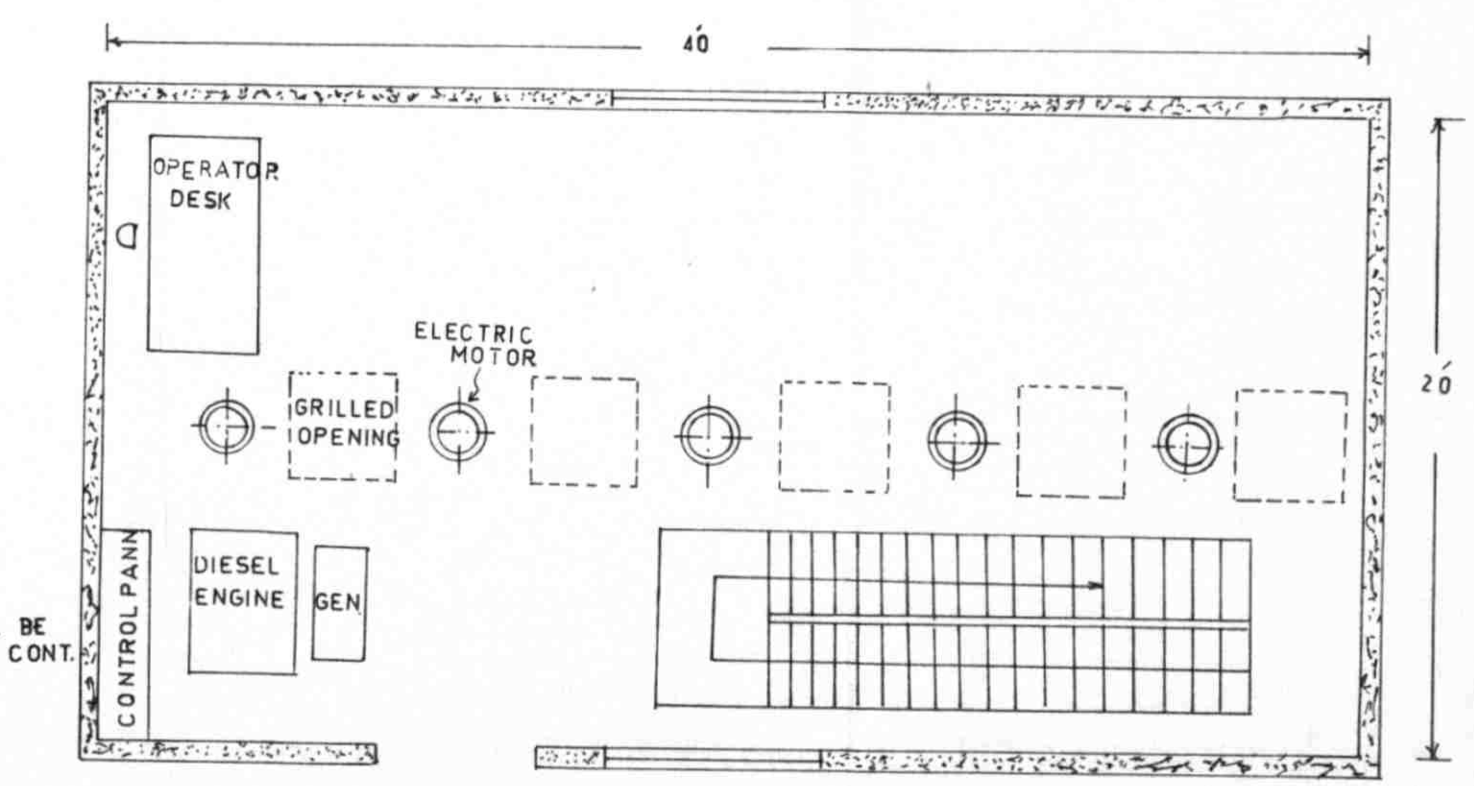
PLAN OF PUMPING STATION



SECTION B-B



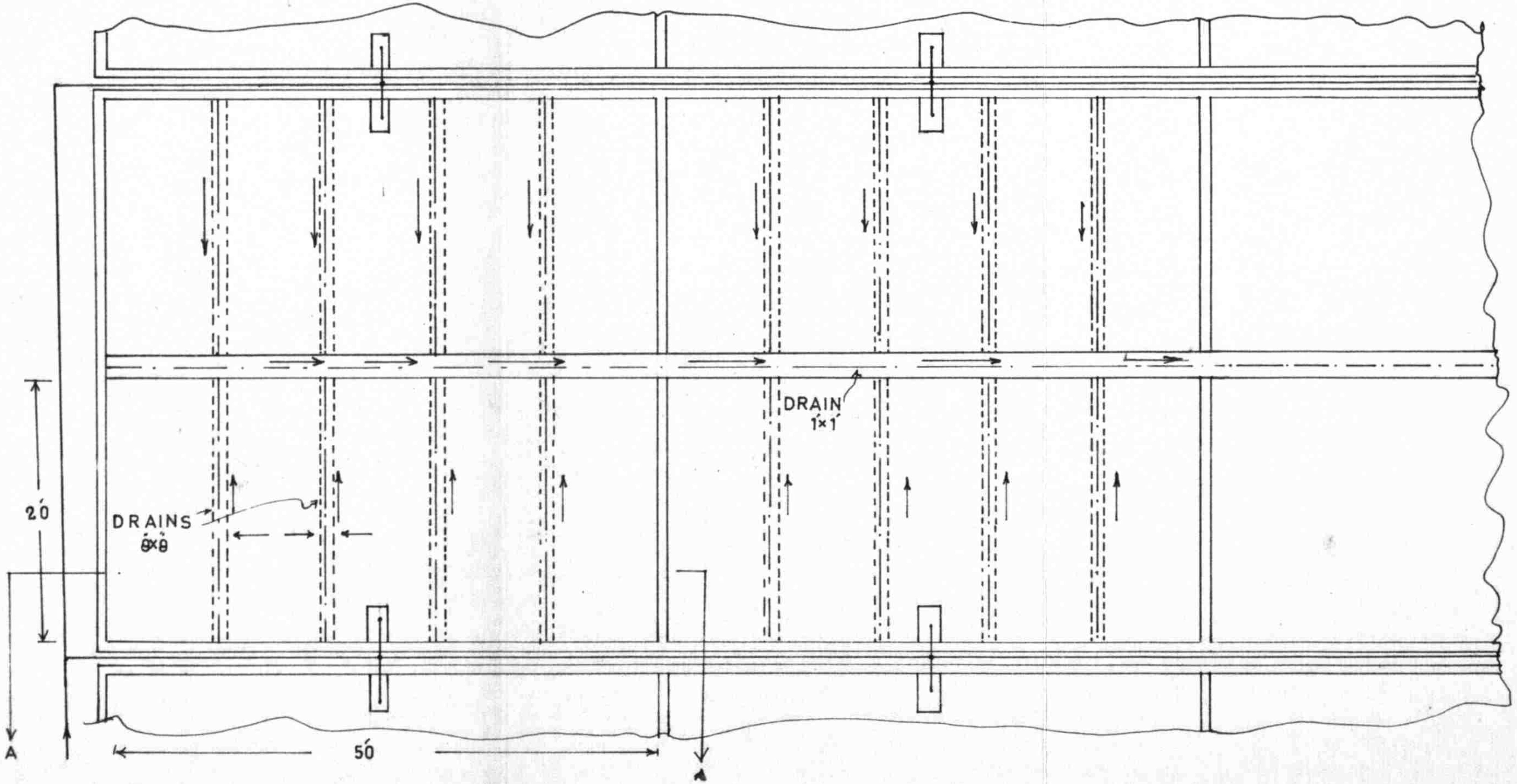
SECTION A-A

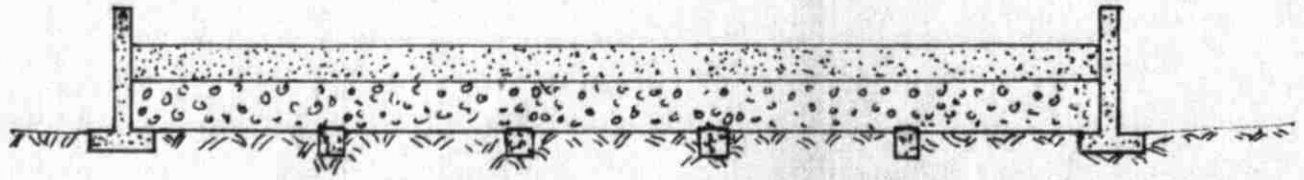


NOTE
RECPUMPS WILL BE INSTALLED IN CONT. ROOM.

PLAN OF CONTROL ROOM

BANNU SEWERAGE SCHEME	
PUMPING STATION	
SCALE 1/53	DRAWING NO. 11
ENGINEER N.H.AFRIDI.	DATE MAY. 1967





SECTION A-A

BANNU SEWERAGE SCHEME	
SLUDGE DRYING BEDS	
SCALE 1/120	DRAWING NO 12
ENGINEER N.H. AFRIDI	MAY 1967