PRELIMINARY DESIGN

OF

SEWERAGE SYSTEM FOR SAIHAT, SAUDI ARABIA

By

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SYNOPSIS

The design for a sewerage system for the city of Saihat in Saudi Arabia has been undertaken as partial fulfillment for the requirements of the degree of Master of Engineering, with major in Sanitary Engineering.

Saihat possesses peculiar problems of flat ground, high subsoil water levels with high salt contents and lack of adequate financial resources for the execution as well as maintenance of the sewerage system. The nearness to the sea and availability of cheap land, however, provide the city with the advantage of treatment of sewage by oxidation lagoon and ultimate disposal of lagoon effluent into the sea. The climate is also helpful for lagoons. The existence of a natural soil, known as marl in the city is an added advantage whereby lagoons can be made impervious satisfactorily at a quite low cost.

The available data has been studied, and design has been made for sewers network, pumping station, treatment works, and supplementary works. Drawings have been prepared for the above works. Brief specifications have been written and approximate cost estimates have been prepared.

Theestimated cost of the total project comes to about U.S. $ 464,500.00 or $24.80 per head of the total designed population.

CHAPTER I

INTRODUCTION

A. Location and History

The city of Saihat is located on the Persian Gulf in the Eastern Province of Saudi Arabia. It is situated at a distance of 6.5 kilometers North of Dammam, the Capital city of Eastern Province, and about 9.5 kilometers South of Qatif, another important city. It lies at an approximate latitude of 26° 28′North and longtitude of 50° 03′East.[1]

Saihat was only a small town few decades ago, inhabited mainly by fishermen and farmers feeding the capital of Province, Dammam. The development of ARAMCO (Arabian American Oil Company) in Dhahran area, at a distance of about 15 kilometers from Dammam in the South, caused heavy influx of population in Dhahran area and the neighbouring areas. This influx resulted in increase in the prices of land and property in the adjoining cities of Dammam, and Khobar. Saihat had the following advantages which promoted and still promote its rapid growth.[2]

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[1] The Associated Consulting Engineers, Unpublished Report on Town Planning of Saihat, Saudi Arabia, submitted to the Government of Saudi Arabia (Beirut: 1964), p. 2.

[2] Ibid., p. 5.

(a) Location and proximity to Damman and Dhahran.

(b) The relative reasonable cost of land in comparison to land prices in the adjoining cities.

(c) Availability of electricity.

(d) The city is surrounded by palm tree plantation and fertile lands. It is expected that there will be improvement in agricultural production.

(e) Construction of a modern agricultural experimental station by the Ministry of Agriculture.

The growth of the city continued on the northern and eastern side under control by the local authorities. Proper planning of the city has however lately been felt necessary for guiding the development and improving the existing conditions of the old city. The problems and requirements of sewage disposal will be dealt with in the subsequent chapter. The local authorities and the Arab Industrial Development Department of ARAMCO made arrangements with the Associated Consulting Engineers, Beirut for the preparation of a master plan for the development of the city incorporating the developments already made by the municipality. The master plan has since been prepared and proper developments are under way.

The increased population and development has caused the existing facilities of water supply, sewerage and other facilities to be felt very much inadequate making it essential for the authorities concerned to produce a master plan and construct adequate public facilities for the present, as well as future increase of population.

B. Area, Topography and Soil

The old city is located on an area, elevated from the surroundings by few meters as shown on the contours on drawing No. 2, gradually rising from the adjacent plains around, over an area of about 100 acres. Further developments were made on the surrounding plains and are still being continued. The master plan covers a total area of about 635 acres including the land under the old town. The plain has a gentle slope towards the sea as well as towards the south.

The soil consists of sandy and silty clay in the old city, while in the surrounding plain this soil is underlaid by a thin layer of sand in varying depths.

The water table is very close to the natural ground surface specially in the region adjacent to the shore and southern part of the city which is almost flat. The depth of water table near the seashore and in the plain varies from surface level at the seashore to about three feet at the far end of the plain. The depth of water table in the old city is greater and varies according to the elevation of the particular place.

The high tides play a predominant part in the variation of ground water levels. On the coastal areas the slope is very gentle and hence the tides go far inland. The water-table is very much influenced by the tides and is highest during the period of high tides.

C. Climate

The climate of Saihat has similar characteristics of the other port cities of Arabian Gulf. There is no weather recording or meteorological station at Saihat, but the weather records of Dhahran shows the following:[3]

(a) Average temperature in summer 97^{o}

(b) Average temperature in Winter 59^{o}

(c) Extreme temperatures recorded are:

Maximum	;	120^{o} F
Minimum	:	35^{o} F

(d) Relative humidity 6% to 100%

(e) Summer starts in mid-May and continues upto the end of September. The winter is pleasant with cool nights and sunny days in general.

(f) Rainfall is very rare. During winter, very few heavy storms of short duration occur. The maximum rainfall recorded during the last twenty years, which occured in the winter 1953-54 totalled about 7" and available records show maximum intensity of of 1.2" in one day. The annual rainfall varies from less than 1" to about 4".

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[3]Associated Consulting Engineers, Unpublished Preliminary Report on Water Supply & Sewerage of Al-Khobar, Saudi Arabia (Beirut: 1963), submitted to the Government of Saudi Arabia.

D. Scope of Work

The city of Saihat requires development of all public facilities including roads, water supply, sewerage and others. While the authorities are actively busy on all phases of improvements, the present study is confined to the design of a sewerage system for the city in partial fulfillment for the Master's requirements of the writer.

The design shall indicate the material, location and sizes of sewers, mains, trunks, also location of collecting chambers, pumping units, establishment of all basic criteria, feasibility of methods of sewage disposal and design of various components of treatment works, and supplementary works.

The preparation of detailed working drawings, estimates and specifications will increase the amount of work very much and will be beyond the scope of this work. Hence, the preparation of detailed drawings, estimates and specification will not be attempted in the work. The estimates will be prepared in rough to give an idea of the cost of the project for allocation of funds.

CHAPTER II

PRESENT PROBLEMS AND REQUIREMENTS OF SEWAGE DISPOSAL

The area is divided into two parts in order to explain the present problems:

(a) Old city

(b) Newly constructed areas.

The old city is made up of small mud houses amidst irregular and narrow streets without proper sewers.[1] The sewage is absorbed in cesspools within the city in the dwellings where running water is available and plumbing system is installed. Very few houses have been provided with septic tanks. The sub-soil water level being high, the sites of cesspools and septic tanks are the sore spots causing insanitary and unhygeinic conditions around. For those dewllings where no running water is available the pit privy serves the purpose.

The newly constructed areas have been provided with few haphazardly laid sewers. These individual sewers flow directly towards the sea and discharge on the sea shore. This produces insanitary condition prohibiting the use of seashore in the area for useful purposes and making it a potential source of

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[1] Associated Consulting Engineers, Report on Town Planning of Saihat, op. cit., p. 6.

danger by the action of tides, when the tides push and spread the sewage polluted water on the ground around the inhabited areas.

It is therefore essential that the sewage of the entire city should be collected by a network of sewers and disposed of by some suitable means to achieve the following objectives:-

(i) The cesspools and septic tanks now producing insanitary conditions are eliminated.

(ii) No sewer falls directly on the sea coast so that the sea water and coastal areas are free from any contamination and can be used for all suitable and useful purpose without any danger of hazards from sewage pollution.

CHAPTER III

POPULATION FORECASTING & DESIGN PERIOD

The design of a sewerage system is based upon the amount of sewage, which depends on the population served. As the whole sewerage sysem has to serve for a certain period of time, in which it is required to give satisfactory performance, it is necessary to determine this design period for the system and forecast as accurately as possible the expected population increase by the end of the design period.

A. Present Population and Past Records

The Government in the country does not carry out any census program and the population records are not existing. The only available source of present population is the census carried out by the Malaria Control Department in 1963.[1] The figures given were 7565 persons in 1168 houses.

During the year 1958, the ARAMCO estimated the population of Saihat as 4000 on the basis of aerial photograph, counting the houses and test census of a small area.

No other population data is available.

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[1] Associated Consulting Engineers, Report on Town Planning of Saihat, op. cit., p. 7.

B. Design Period and Expected Population Increase

The design period depends upon the following factors:

(a) Anticipated rate of growth of population.

(b) The cost of extension or increasing the capacity.

(c) Useful life of structures and equipment employed.

(d) Financial position of the sponsoring authority.

(e) Performance of work during the early years when they are not loaded to capacity.

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

The population data indicate the following:

Population in 1963	7565
Population in 1958	4000
Population increase in the last five years	3565
Probable percentage increase	89.2 %

The areas covered in the master plan indicate the following :

Area under old city	= 101.11 acres
Net area newly developed under previous planning	= 138.45 acres
Net additional area to be developed under the master plan	= 318.4 acres
Open spaces	= 77.15 acres
Total area of city	= 635.11 acres

The probable percentage increase in population during the last five years i.e. 89.2% is very high. This high rate of increase had been due to the reasons already explained earlier in chapter I. Big building programs are underway in the adjoining cities of Khobar and Dammam where the projects of water supply, sewerage, roads and electricity are already under advanced stages of completion and the building boom has already been shifted to those places. Thus the high rate of development in Saihat will be very much reduced.

Before starting with the discussion on the expected future increase in population, the following basic assumptions are made :-

(1) The form of political, social and economic organization will remain substantially unchanged.

(2) No war, internal revolution, nationwide devastation, or epidemic will occur.

(3) No large scale 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.

No provision can be made for the above factors as those are entirely unpredictable and cannot be accounted for in the projects.

Due consideration has been made for the population increase in the master plan. All the established methods of population increase were considered and it was concluded that a population increase within a design period of fifty years will require this much development as indicated in the master plan.

In order to justify the above conclusion, the established methods of population forecasting as generally considered by the sanitary engineers are discussed in brief in appendix I. Forecast is made on that basis. As this method also requires a sufficient past record, it cannot be applied in this case.

It will be seen from the foregoing discussion of all the established methods of population forecasting usually employed in the sanitary works, that the absence of sufficient record of past population is a big hurdle in determining the future expected population. We are therefore left with judgment of population within a period of next fifty years on some other basis.

The approach can be changed from the population data to the extent of development easily attainable in the suburb of the city. The study of the surrounding area of the city indicate that development is most economical towards the southern side of the city. The previous planned development is already touching the sea coast on eastern side. The northern side and western side are flanked by palm tree plantation and agricultural land, which are costly. There is plenty of area available on the southern side which can be developed. An area of about 318 acres had been selected out of this as suitable for development at minimum cost. This area has been included in the master plan as an additional area to be developed and converted into lots of various sizes to the requirements of people of different financial resources. Adequate provision had been made for all modern amenities of roads, parks, schools, commercial centers, and sport spaces.

The population calculated on the basis of these lots is expected to be a more representative figure of the anticipated population increase.

The anticipated population at the end of fifty years according to the master plan is calculated as under:-

Population as per 1963 census	= 7565
Houses as per 1963 census	= 1168
Average number of persons per house	$= \frac{7565}{1168} = 6.5$
Number of houses in the newly developed area as counted from the plan	= 235 houses
Number of persons inhabited in the newly developed areaassuming that population density remains the same	= 235 x 6.5 = 1530 persons.
Number of persons in the old city	= 7565 - 1530 = = 6035 persons.
Number of houses that are still to be built in the newly developed area as counted from the plan	= 498
Number of houses now proposed in the master plan as counted from the master plan	= 1539 houses

Total number of houses to be built within the city under the master plan = 498 + 1539

= 2037 houses

In the foregoing calculation of persons in the city, we got a figure of 6.5 persons per house. But in the future development this figure is most likely to be reduced for the reason that formerly the system of tribes was prevalent and is still continuing to a certain extent in Saudi Arabia. Usually people live in combined families for safety and economy. Now, with the development and rising standard of living, the families in the tribal form are expected to be segregated and thus the population intensity per house will go down. It is therefore most probable that population per house will come down to 5.5 on an average. Further calculations will be made on this figure for expected population.

Therefore the expected population increase will be

= 2037 x 5.5 = 11,195 persons

The total expected population at the end of 50 years will be = 7565 + 11195

= 18,760 persons

This figure of population appears to be reasonable.

The population intensities are calculated as follows:

Population intensity in the old town $= \frac{6035}{101.11} = 61$ persons per acre.

Population intensity in the newly developed areas:

$$= \frac{(235 \times 6.5 + 498 \times 5.5)}{138.45}$$

= 31 persons per acre.

Population intensity in the proposed developing area

$$= \frac{1539 \times 5.5}{318.40}$$

= 26 persons per acre.

Although few multi-storied buildings have been constructed so far, in the area previously devèloped , it is not expected that the new development will be covered by a large number of such buildings due to high cost of construction, weak sub-soil strata, high water table, lack of financial resources of the people in the area and bleak prospects of good return of investments. It is therefore assumed that most of the houses will be single storied and each of the plot of land will have one residence only.

The anticipated population of 18,760 person appears to be justified.

The design of the sewerage system will be based on this population. The lateral and main sewers will be designed and constructed on the basis of full population growth, but efforts will be made that the trunk sewers have not to run under loaded pending development which is likely to take a long time to occur. The whole project is recommended tobe constructed in stages. The laterals and main sewers will also be laid where population exists. Further laterals and mains sewers will be added as the population development goes further and further.

The population is likely to continue even after fifty years of presently planned period, but by that time the whole sewerage system may require redesign, depending upon the actual population growth utilizing the existing works constructed now with or without addition and replacement.

CHAPTER IV

WATER RESOURCES AND AMOUNT OF SEWAGE

The study of existing water supply is necessary as the amount of sewage depends largely upon the quantity of water consumed by the community. Due consideration has to be given for the proposed water supply system and provision has got to be made in the planned sewerage system so that no renewal of the sewerage system is necessary with the augmentation of water supply system.

A. Water Consumption Per Capita

The present water supply of the city comes from thirteen dug wells each about four hundred feet deep. These wells run under artesian conditions and water comes out under a pressure of few feet. This water comes from the acquifer known as Khobar acquifer which exists at a depth below three hundred and twenty feet. The discharge from this acquifer varies from place to place in a wide range from 10 g.p.m. to 400 g.p.m. Most of the houses are directly connected with the pipes to the wells and water flows in these pipes under artesian pressure. There is no check on the quantity of water flowing into the individual houses. The test reports indicate that the discharge from the individual well is about 150 g.p.m.[1]

In the circumstances it is impossible to estimate the present average rate of per capita consumption accurately. Taking into account the general plumbing system and sanitary fittings and wastage of water, it is probable that the per capita water consumption is about 30 to 40 gallons per day.

The design of a comprehensive water supply scheme is actively under way. The basis of this design is the master plan already discussed in the preceding chapter. The new development is likely to be continued under proper planning and it is expected that the house construction will be in accordance with the modern public health standard, fitted with modern sanitary fittings and with water carrieage system of sewage disposal. Also it is expected that with the anticipated rise in the standard of living of community, the old houses willbe developed and installed with modern sanitary fittings. Taking into account all these factors, it is probable that the future water consumption per capita will be fifty gallons per capita per day and it is most likely that this figure will be adopted in the design for sewerage system in the following study.

B. Amount of Sewage

The basic factor involved in determining the amount of

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[1] Obtained from the data collected from ARAMCO, Dhahran, by the Associated Consulting Engineers, Beirut.

sewage is the type of sewerage system adopted either separate sanitary sewers or combined with storm water.

A separate system of disposal of sanitary sewage is proposed for the following reasons :-

(a) The city is situated on the sea coast having a general slope of land towards the sea. The rain water will flow directly over land from the city to the sea and is not likely to constitute any hazard. Ponding may be expected to occur in certain areas. Arrangement will be made to pump this ponded water, or outlets will be provided in certain manholes for disposal of this water.

(b) The area has very little rainfall, and that too in the form of very short duration storms. The combined sewers and treatment works if designed will run underloaded for the whole year except for a short duration of storm under constant danger of clogging and choking.

(c) The combined system of sewers will necessitate heavy cost of pumping the storm water along with the sewage as the area is very flat and pumping of sewage will be essential.

(d) There is no objection of discharging of storm water into the sea.

(e) The mixture of storm and sanitary sewage would necessitate the treatment of both and would thus increase the cost of treatment in comparison to separate system where the cost of maintenance is restricted to the sanitary sewage only.

(f) Domestic sewage has to be taken to a sufficient distance from the habitation for treatment or disposal. In a combined system, the storm water will have to be carried to the same distance at extra cost.

(g) The using of separate system reduces the size of the pipes as well as first cost in installation.

In view of all the advantages of separate system and disadvantages of combined system for this particular city, the separate system for sanitary sewage is fully justified and will be adopted.

No system of disposal of storm water is proposed for the city. The storm water will flow over the roads and paths and will ultimately go into the sea.

The rainfall records of about twenty years is available with the ARAMCO. This record is enough for design on a twenty years cycle of storm. The records indicate that maximum rainfall occurred in 1953-54 and the total seasonal rainfall was 7 inches. The maximum rainfall as per details available is 1.2. inches in one day. The details of storm and maximum rainfall intensity is not available. The soil in the city is sandy and a large area is at present unpaved and is likely to remain unpaved due to limited financial resources of the local municipality. A considerable portion of water will thus be absorbed in the surface soil.

As the storms in the area are of flash nature a very high intensity of rainfall usually occurs and is expected. This high intensity of rainfall will cause ponding in the areas which are very flat.

As the financial resources are limited, no storm water sewers can be recommended, because even if the storm water sewers are constructed, pumping will have to be resorted to as the area is almost flat, and it will not be possible for the municipality to maintain this system properly.

It is therefore proposed that the areas subjected to ponding will be provided with inlets to the sanitary sewers. This is likely to overload the pumping station, but it can be allowed because such a condition will occur only once in a few years. The street pavements and roads will be provided with enough camber and slope to reduce the effects of pondage.

The amount of sewage flow depends upon the following:

(a) Domestic Sewage

(b) Commercial Sewage

(c) Ground Water infiltration

(d) Industrial Waste

Each of the above is dealt with briefly as follows :-

(a) Domestic Sewage. The amount of domestic sewage depends upon the water consumption per capita per day. About

70 percent to 80 percent of domestic water consumption usually flows into the sewers.[2] The balance does not reach the sewers due to leakage, lawn sprinkling and manufacturing process. The rate of domestic flow is not constant. It varies with the season of the year and hour of the day. The maximum and minimum rates of sewage flow are the controlling factors in the design of sewers.

(b) <u>Commercial Sewage.</u> Commercial establishments like hotels, restaurants, offices, hospitals and schools also contribute towards the sewage flow. As the city in question is small, the sewage flow from such commercial establishments is expected to be small and is usually considered as included in the domestic flow. For the purpose of design, the same density of population has been adopted for the commercial and recreational areas. This is likely to increase the total population by a small amount which is not likely to effect the design.

(c) <u>Industrial Flow.</u> Saihat is not an industrial town and there are no industries which can contribute appreciably towards increasing the sewage flow. No provision is being

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[2] Water Pollution Control Federation Manual of Practice No. 8, (American Society of Civil Engineers, Manual of Engineering Practice No. 38) (A Joint Committee), <u>Sewage Treatment Plant Design</u> Third printing, (Washington: 1963), p. 5.

made for industrial flow as there is no likelihood of this city becoming and industrial town in near future.

(d) <u>Ground Water Infiltration.</u> There is always possibility of ground water entering into the sewerage system through joints and manholes that are not watertight or through cracks in the pipes. This amount of seepage depends upon the following factors :-

(i) Ground water levels above the invert of sewers.

(ii) Permeability of soil.

(iii) Structural defect in pipes

(iv) Poor jointing material

(v) Improper foundation provision

(vi) Poor workmanship during installation of sewers.

(vii) Number of joints

Sanitary sewers have to be designed to carry the unavoidable amounts of infilteration of ground water in addition to the peak sanitary flows. No commonly acceptable method has so far been developed for evaluating rates of infiltration to be used in design or to be allowed in specification. The allowance for ground water infilteration is based on a determination of conditions effecting infilteration and the exercise of judgment in evaluating their effects. Various bases and allowances have been used including allowances per mile, per inch diameter mile of sewer, per capita and per acre of sewered area.

For the city of Saihat water level is very high, and the subsoil consists of silty and sandy clay with high permeability. Taking into account both these factors it is probable that the infiltration will not exceed thirty gallons per capita per day.[3] The work will be adequately designed and supervised in order to render the infiltration absolutely minimum.

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

(a) Domestic sewage	=	40 gpcpd
(b) Infiltration	=	30 gpcpd
Total flow	=	70 gpcpd

There is a girls school towards the west of the city at a distance of about 3500 feet from the western edge of the old city. The sewage of this school cannot be included in the city sewerage system as the distance is very great and will necessitate abnormal depths of sewers. It is therefore proposed that this school may be provided with a septic tank for disposal of its sewage. This school has therefore not been included in the design of the system.

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[3] Ibid.

CHAPTER V

SEWERS NETWORK

A. Design Criteria

Domestic sewage containing harmful substances and pathogenic bacteria, is very dangerous for human health. It must therefore be removed quickly through a network of sewers and disposed of so as not to cause any harm to life or property.

The design of sewers is based upon the amount of sewage flow. The amount of sewage flow is a variable factor and depends upon the time of the day, the day of the week and the season of the year. The hourly variation in the consumption of water reaching the sewers may result in the flow upto 200 percent of the average rate during peak water consumption hours while the minimum flow may go down to 50 percent of the average or even less during the hours of minimum water consumption. The seasonal variation of flow may result in the increase upto about 150 percent of the yearly average in summer and decrease upto about 75 percent during winter.

The maximum flow in the sewer will thus be about three times the average flow. This peak is normally attainable in the case of laterals. But in the case of mains and trunk sewers the position is somewhat different. The laterals serve a small area where the residents usually belong to one group or profession for example office workers, business men and others with different habits and times of maximum water consumption. The peaks in all laterals feeding the mains and trunks never coincide. The intensity of the peak in the mains and trunks will therefore be less. But this amount cannot be worked out accurately as it involves lot of uncertainty. Similarly uncertainty is involved in determining various depths and dimensions. So in order to facilitate the job of the designing engineer, various authorities have standardized design criteria based on long years of study and experience. For the purpose of this design work, the New York State Department of Health 1947 design criteria has been adopted and the same is reproduced as follows[1]:-

1. Capacity of laterals and sub-main	4 times average daily flow
2. Capacity of mains, trunks, intercepting and outfall sewers.	2.5 times average daily flow
3. Absolute minimum flow	0.5 times average daily flow.
4. Minimum velocity flowing full	2 ft/sec.
5. Maximum velocity	10 ft/sec.

[1]The Associated Consulting Engineers, Preliminary Report on Water Supply and Sewerage of Al-Khobar, Saudi Arabia, p. 25.

6. Minimum depth of sewage in pipe for minimum estimated flow	2 inches
7. Absolute minimum diameter of lateral sewer pipe	8 inches
8. Minimum depth below street surface to the top of lateral sewer	3 feet
9. Minimum depth below street surface to the top of main/ trunk	5 feet

10. The invert of laterals at a junction should be above those of main by at least 1/2 the difference in diameter of small pipe and 1/4 times the difference of large pipe.

11. Sewers provided within 50 feet or less of every plot.

12. Manholes provided at all angles for bends, changes in grade, at all junctions of street sewers, all the upper ends of all laterals and at intermediate points where the distance exceeds 250 feet.

Exception to the above criteria has been made in case of item 8. As the area is very flat and subsoil water is very high, the cost of execution and construction below water table increases more than the proportional depth. The depth of the top of the sewer pipe has therefore been adopted as 2.5 feet below the finished road level. The present contours indicate level of grounds. It is assumed that the hard crust of the roads to be constructed will make the level of road surface about 6 inches higher than ground level. The calculations have been made on this assumption. The city being very small a very high intensity of traffic and loading is not expected on the roads in the town. The pipe sewers laid at this level will be thus safe from the thrust on the road. The maximum velocity indicated is high, but in this case such a velocity will never be achieved as the area is mostly flat.

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

(i) Circular shape has the advantage of giving maximum cross-sectional area for the amount of material in the wall.

(ii) Convenience in manufacture.

(iii) Good hydraulic properties

(iv) Fairly stable in place.

B. Design of Network

A system of sewers which provide sewage flow through sewers and treatment plant until final disposal by gravity is the most desirable for reasons of economy in the initial cost as well as in maintenance. But for the city of Saihat, this goal is not achievable as the area is very flat and pumping is not avoidable. As the full development of the area is likely to take about fifty years, the aim of the sewerage system is to serve the present area immediately and to be later extended gradually to serve the whole planned area.

It is apparent from the contours of the city as per drawing No. 2 that the old city is situated over an elevated ground with peak located on the northwest corner sloping all around. There is a general slope of the surrounding plain towards the sea. This topography poses a serious problem for a sewerage system of the area. There is very little population on the western side with no development planned for future. A small development had been planned in the north and eastern side while a major planning has been done on the southern side which is almost flat.

The coastal road on the northern part is located on quite a low ground which is flooded by high tide. It is therefore necessary that the level of the ground will be raised all along the coast line to an elevation of two feet above sea level so that the road is not flooded by the high tide. The ground level of two feet has therefore been assumed on the coastal road.

There are three alternatives available for a sewerage network :-

(a) System having one pumping station

(b) System having two pumping stations

(c) System having three pumping stations.

The above systems are briefly discussed in order to determine the most suitable one :-

(a) System Having one Pumping Station

In this system, the sewers will direct towards one pumping station suitably located within the city. This system will surely save the cost of constructing more than one pumping station, but it will increase the depth of not only the pumping station very much but will increase the excavation of the trunk sewers and main sewers abnormally. As the area is flat and the sub-soil water is very near the surface, this system would be too expensive to be adopted. Moreover, it will cause unnecessary fall in the sewage flow, thus increasing pumping head which can be easily avoided by pumping from a shallow depth. The maintenance expenses will also be high making this system unsuitable for this city.

(b) System Having Two Pumping Stations

In this system the whole city will be divided into two sewerage districts each having independent sewers feeding two pumping stations. This system has the advantages of reducing the depths of wet wells, pumping stations and sewers as well as reduction in the pumping head. Thus this method is superior to the first method. But in this case also the depth of trunk sewers, wet wells and pumping stations will be quite excessive as the new developments on the north, east and mainly south are very flat and the distances are quite long.

(c) System Having Three Pumping Stations

In this system the whole city will be divided into three sewerage districts each having trunk sewer, main and laterals feeding three pumping stations each located in a district. This system will greatly eliminate the disadvantages of the above two systems. The lengths and diameters of the mains and trunks will be much less and thus the depth of wet wells, trunk sewers and mains will be much reduced and will fall within reasonable limits. The shallow depths of wet wells will reduce the pumping cost by reducing the pumping head. This system will however have the disadvantage of having many units of wet wells, pumping plants and equipment. But this can be compared well with the savings in constructing deep underground with subsoil water very near to the surface and reduced cost of maintenance by low pumping heads. Thus this system will be most economical in first cost as well as maintenance.

A sewers network has been designed on the basis of the last system having three pumping stations, one for the northern areas, second for the old city and eastern portion and the third for the southern development. Location for the pumping stations and wet wells have been selected by trial and error as the one which is giving almost equal depth of excavation for all sewers reaching to that place from the surroundings. This will give maximum economy in construction by reducing the depth of sewers and consequently wet wells.

The sewer districts have been marked on the city plan. The main sewers, trunk sewers and the site of the wet wells and pumping stations have been shown on the plan.

The areas, population, discharges, slopes and levels have been calculated and tabulated as per sheet numbers 1, 2, 3, 4. attached as per appendix IV.The calculation for velocities and slopes have been based on the monogram based on Mannings Formulae for circular pipes flowing full with $n = 0.013$[2].

According to the design criteria adopted the minimum depth of flow during lowest discharge period, should not be less than 2 inches. Calculations of partial flow as per graph of hydraulic element of a circular pipe[3] indicate the following minimum discharges for the sizes of pipes use.

For 8" dia. minimum discharge = 0.098 c.f.s.

For 10" dia. minimum discharge = 0.176 c.f.s.

For 12" dia. minimum discharge = 0.204 c.f.s.

Although these limiting discharges have been kept in view while designing the network in all diameters, in case of 8 inch pipe the minimum discharge in main sewer is usually much less than this amount. This cannot be rectified as the only solution is the reduction of diameter of pipe, which we have assumed 8 inches as absolute minimum. However, it is most probable that the sedimentation if any during lowest discharge period will be washed away by flushing the sewers and there will be no

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[2] E.W. Steel, <u>Water Supply & Sewerage</u>, (New York: McGraw-Hill Book Co., Inc., 1960), p. 371.

[3] <u>Ibid.</u>, p. 375.

trouble. The diameter of pipe sewer cannot be reduced as it will cause difficulty in cleaning if there is serious clogging taking place.

The design has been made on the basis of maximum population at the end of designed period, which will take a long time to attain and it is probable that the velocity will be much less during the initial period. Periodic cleaning of sewers will be done by flushing with a hose in all main and trunks.

Longitudinal section of the main sewers and trunk sewers have also been drawn and attached as per appendix V.

In the design of sewers network it has been assumed that no basements will be constructed. The subsoil water being very high, no basement can be efficiently constructed for ordinary purposes. No provision has therefore been made in the design for basements. If however some body does construct a basement, he will have to provide for lifting of sewage to a level above the invert of the lateral running along the street.

C. Pipe Material

For the old city the sewers will be laid of asbestos cement pipes or concrete pipes or vitrified clay pipes whichever is economical. For newly developed area vitrified clay pipe will be used. A brief discussion of pipe material has been given in Appendix II.

D. Embedment of Sewers

The sewers normally have to be bur-ied under the ground and are therefore subject to loads of backfill and external loads on the road. For large sewers and for those cases where the external loads are heavy, the actual loads and stresses developed in pipes due to that are calculated. The pipe material and strength are then designed accordingly. The ordinary pipes of vitrified clay,[5] asbestos cement and cement concrete pipes are sufficiently strong to bear the pressures resulting from the average backfill and street loads.

As the load transmitted to the pipe is proportionate to the width of the trench, the width should not exceed one and a half times the outside diameters of pipe[6] or the minimum space which allows workability for laying and jointing of pipes.

A suitable bed is essential for placing the pipe sewer in the trenches. It has been experienced that a well compacted bed of soil usually serves the purpose very well. It is therefore recommended that class B bedding may be provided which states as under :[7]

> Compacted granular bedding with tamped backfill. The pipe shall be bedded in compacted granular material placed on a flat trench bottom. The granular material shall be crushed stone or pea gravel which will pass a 1/2-inch sieve but will be retained on a No. 4

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[5] Hardenberg W.A., Sewerage and Sewage Treatment (Scranton, Pennsylvania: International Book Co., 1950), p. 35.

[6] *Ibid.*, p. 55.

[7] Water Pollution Control Federal Manual of Practice No. 9 American Society of Civil Engineers Manual of Engineering Practice

> sieve. The granular bedding shall have a minimum thickness of one-fourth the outside pipe diameter and shall extend halfway up the pipe barrel at the sides. The remainder of the side fills and a minimum depth of 12 in. over the top of the pipe shall be filled with carefully compacted material.

This will equally distribute the load on the pipe sewer to the bed of the trench. This will serve the purpose satisfactorily without causing any trouble.

E. Manholes

[8] Manholes are the most common of all sewer appurtenances required for the proper function of sewerage system. Their principal function is to permit the inspection and clearing of sewers.

As already stated in the discussion of design criteria, manholes will be provided at all angles for bends, changes in grades, at all junctions of street sewers, at the upper end of all laterals and at intermediate points where distance exceeds 250 feet.

Circular manholes are proposed for reasons of greater stability and economy in construction. The size of the manhole will depend upon the size of the sewer on which it is constructed. A minimum inside diameter of 4 feet will be adopted for laterals and mains, while a diameter of 5 feet will be adopted for trunk

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Contd. from p. 33.

No. 37 (A Joint Committee) (Third Printing 1963), Design and Construction of Sanitary and Storm Sewers (Washington: 1960), pp. 202-203.

sewers.

The manholes will be constructed of concrete. Concrete has been selected for reasons of strength, flexibility and economy in comparison to other material. The thickness of the wall will depend upon the depth of the manhole. The minimum thickness adopted will be four inches for laterals and mains while it will be six inches for trunk inold city. In the new city, however, greater thickness will be required to withstand water pressures and thickness of six inches for laterals and main sewers and eight inches for trunks is recommended. The manholes will be allowed to be constructed as cast-in-place or reinforced precast rings as suitable to the contractor doing the job. It is recommended that the exposed surfaces of the concrete will be plastered while the outer surface of concrete will be applied with bituminous paint coating to prevent corrosion due to salts present in the subsoil water. This coating will be done for those manholes which lie in the area where the water table is high. Iron steps will be provided on side to facilitate coming up and going down for sewer men.

A clear opening of 21 inches is recommended in the manhole at the road surface level. This opening will be placed either in the centre or side of the manhole. Cast iron manhole covers and frames are proposed. The covers will be rough at the top to prevent slipperness. The system being designed for domestic sewage only, the covers will not be perforated. The top surface of the cover will be constructed flush with the finished road so that there is no difficulty to traffic.

A base slab of concrete eight inches thick will be provided on lateral sewers to support the wall of the manhole and to prevent the entrance of ground water. This thickness will be increased in case of mains and trunk sewers according to the depth of sewer and depth of ground water table. The thickness and reinforcement will be calculated to be safe under the hydrostatic pressure of subsoil water for the walls as well as the bottom.

All the sanitary flow shall be carried in smoothly constructed U-shaped channel which may be formed integrally with the concrete base or may be constructed separately. The side height of the channel will be 1/2 to 3/4 of the diameter of the sewer. The adjacent floor area will be sloped to drain to the channel with slope of one inch per foot. When more than one sewer enters a manhole the flow channel may be curved smoothly and will have enough capacity to carry the maximum flow.

Drop manholes will be provided at the junction of the laterals with the main or trunk. Outside connection or back drop will be provided for protection of the man entering the manhole. The entire outside drop will be encased in concrete to protect it against damage during back fill. The opening on the side of the manhole will serve for inspection and clearing of sewer joining the manhole.

CHAPTER VI

SEWAGE TREATMENT

A. Design Criteria

The satisfactory disposal of town sewage is most important from the point of view of health and general well being of the public, as the sewage in addition to pathogenic bacteria contains highly putrescible matter like feces, bits of garbage, decaying fruits and other useless and discarded material. This matter decomposes quickly specially in warm weather with the production of abnoxious odors. In coastal areas like Saihat, where sewage is not disposed of carefully in sea water, objectional deposits may be formed and decaying solids may strand on the shore, spoiling recreational spots and swimming beaches. Grease and soap contained in sewage may rise to surface and cause froth and foaming. A well planned disposal system is therefore essential for the town. The existing condition necessitates the following criteria to be kept in mind while designing the sewage disposal system:

(1) The disposal system should give satisfactory results. There should be no damage or pollution of surface or ground water and no damage to health or property.

(2) As it will take a long time about fifty years when the area is expected to be fully developed, the disposal work therefore should be such that the final capacity can be built up in stages. The individual units should be such that they can be operated efficiently under partial loading or slightly overloading.

(3) The plant should be simple, efficient and economical.

(4) As the area is small, the maintenance cost of plant should be low. The plant should not require the service of skilled operator or specialist and should require only little maintenance.

(5) The plant design should be based on the maximum use of local condition and local available material.

(6) As the country as a whole is very much under-developed and this city being small, it will be very difficult to maintain mechanical and electrical equipment. The repairs and replacement of mechanical and electrical parts machinery will be very expensive. The use of machinery therefore will be kept as minimum. Unskilled labour is quite cheap in the area. Therefore, efforts will be made to use maximum of unskilled and semi-skilled labour.

(7) The design of the treatment work will be based on the average flow of sewage expected taking into account the present population, and expected increase during the design

period. The pumping units will be designed for maximum and minimum flow at any time. Several units willbe proposed to cope with the requirements. The initial installation will be confined to the present requirement with facilities of extension as the need arises with the increase in population.

B. Methods of Disposal

Oxidation lagoon has been considered to be the best suited method of sewage treatment for this city. A brief discussion of different available methods of treatment has been given in Appendix III.

C. Available Financial Resources

The city of Saihat has a small municipality to look after the usual essential services in the city. It has a very limited income and is not in a position to contribute any substantial amount towards the execution of a sewerage scheme. The sewerage schemes are nonproductive and cannot be expected to repay the cost or depreciation. The people of the city are not well off financially and cannot be expected to pay heavy amount of taxes. A small amount of rent will be charged from the individual houses but that will have to be spent in the maintenance of the system. The maintenance of this system will be expensive as it will involve a great amount of pumping. It is therefore not expected that any appreciable amount can be saved out of the tax income to repay the amount of initial cost if obtained on loan or of depreciation.

The Arab Industrial Development Department of the ARAMCO has agreed to provide the Government of Saudi Arabia with the required loan free of interest to execute the development schemes of this city including water supply sewerage and roads for the newly planned city.[1] No provision will therefore be made for repayment of the capital involved or interest thereon.

However, it cannot be over emphasized that the whole Saudi Arabia is very much under developed and requires great amount of money for development. The local municipalities have very limited income and it is beyond their capacities to contribute towards big schemes of water supply and sewage. All efforts have therefore been made to keep the cost of the scheme to absolute minimum, consistent with efficient performance of the system.

In case of sewers, however, no provision has been cut short for economy so that the life of the sewer may not be reduced and troubles of choking and silting may not arise reflecting badly upon the designing and execution.

D. Site for Treatment Works

The oxidation lagoons will be constructed outside the city limits at a distance of about quarter a mile from the end of the city along the coastal road on its southern extension as shown on drawing No. 15. The location on the southern por-

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[1]Information collected by Associated Consulting Engineers, Beirut.

tion has been selected because of the general slope of the area towards the south, and smaller length of effluent channel required for ultimate disposal of lagoon effluent into the sea. This location will be well isolated from the residential and commercial areas and will not cause any trouble even if shock loads produce abnormal condition of odors and nuisance under extraordinary circumstances.

This location will have another advantage that the effluent from the lagoons will be discharged into the sea at a distance of about a quarter mile from the city and in case the quality of effluent is reduced due to any reason, it will not have adverse effects on the coast along the city.

The site will be located at a distance of about 500 feet to the west of coastal road in order to save lagoons from the possible maximum high tides. It will give an added advantage that the effluent will flow by gravity into the sea.

Although the subsoil water level is high at this location, it will not interfere with the efficient functioning of the lagoon as no heavy structures are required to be constructed.

CHAPTER VII

DESIGN OF PUMPING STATION

A sewerage system without pumping is most desirable as it requires least cost for the maintenance of the system. The greater portion of Saihat on the eastern and southern parts is almost flat. This will require greater depths of sewers. The pumping will have to be resorted to, and will not be avoidable.

The design of the sewers network described in chapter V necessitated the construction of pumping stations at three places for most economical maintenance. The location of each of these pumping stations has been marked in Drawing No. 2 and 3 as PSI, PSII, and PSIII. It will not be economical to lay a separate pressure sewer from each pumping station to the oxidation lagoon, for the following reasons :-

(1) Increase in initial cost of pipe and heavy pumps.

(2) Increase in maintenance cost for the long pipes.

(3) Extra power to pump sewage against friction in long length of pipes.

It will therefore be economical to work the two pumping stations PSI and PSII as lift stations. The sewage from the PSI will be pumped into the PSII and from there the combined sewage will be pumped into PSIII. This pumping station PSIII will pump the sewage directly into the lagoons. Further saving will be effected in the length of pressure mains by utilizing the main-sewers of PS II and PS III for carrying discharges from the PS I and PS II. The pressure sewer from PSI will discharge its sewage into the main sewer M_1 of PS II at the manhole M_1H_5 from where sewage will flow by gravityto PS II. The pressure sewer from PS II will discharge its sewage into the main sewer S_1 of PS III at the manhole S_1H_2 from where sewage will flow by gravity to PS III.

At each of these pumping stations the following structures are necessary and will be provided :

(1) Screens,

(2) Grit channel,

(3) Wet well, dry well and control room,

(4) Pumping units, and pressure sewer.

(1) Screens. Screens are essential to remove large suspended or floating solids which may clog the pumps and pipes. The following design criteria will be adopted in the design of screens :-

(i) The screens will consist of galvanized steel racks having a cross-section of 2" x 3/8" with 2" side parallel to sewage flow.

(ii) The racks will be inclined to the horizontal at an angle of 60^o.

(iii) The velocity of flow through the racks shall be a minimum of 1 1/2 ft. per second.

(iv) Effective depth of rack will be the depth of the rack from the invert of the sewer to the maximum level of sewage or diameter of sewer.

(v) The distance between the racks will be 2 inches.

The screenings will be removed manually from the racks at convenient intervals. Manual operation is adopted in view of the cheap unskilled labour available. The screenings will be disposed of along with the general refuse of the city or burnt.

The invert of the bottom of screen will be kept 6 inches below the invert of the sewer to allow space for collection of large particles which otherwise may cause stagnation and clogging in the screens as well as in the sewers.

(2) Grit Channel. A grit channel will be provided at each pumping station to remove grit, sand and other inorganic settleable matter in order to reduce abrassion and wear of the equipments and pipes and to eliminate possibility of filling of treatment plant and reduction of its capacity. The horizontal velocity of flow will be kept at 1 ft/sec. in the channel. At this velocity all the inorganic matter, grit and silt usually

settle down. The variable flows of sewage will produce variable velocities in the grit channel. This difficulty will be overcome by providing two channels in parallel and placing them in or out of operation according to the rate of flow. Parshall flume will also be provided to control the velocity in the channel. The design will be made on the basis of peak flows so that there is no overflooding.

The depths of sewers reaching the pumping station will be quite considerable, so it will be economical to construct the grit channel on the longer side of the wet well to save cost of construction under water in a separate excavation. This will also facilitate operation and maintenance.

The cost of maintenance of mechanical equipment for removing grit will be very high in comparison to cheap available unskilled labour. The grit will therefore be removed manually. The two grit channels will facilitate the grit removal. One channel will be made empty during low flows and will be cleared of grit. The grit will be washed in a separate chamber as shown in drawing No. 12A. The grit shall be removed by a perforated bucket which will be pulled up from above.

A drain open jointed pipes will be embeded in the bed of grit channel to drain away the sewage collected before the removal of grit. This sewage will be led into the wet well.

The grit removed will be utilized for filling of low areas around the pumping station or else where as required.

The following assumptions are made in the design of grit channel:-

(i) Horizontal velocity of flow in the channel as 1 ft/sec.

(ii) Vertical velocity of settlement of particles having a size of 0.2 m.m. as 1 inch/sec. This size of particles is usually met within sewage.

(iii) Effective depth of grit channel as 12 inches minimum.

The length of grit channel required will then be 12 feet as a particle will take 12 seconds and 12 feet to reach a depth of 12 inches at the rate of 1 inch per second.

(3) <u>Wet Well, Dry Well and Control Room.</u> The flow of sewage is never constant and discharge varies from time to time. A wet well is therefore necessary to act as an equilizing basin to minimize the fluctuations of load on the pump and tomake the automatic operation possible with simple controls.

The capacity of the wet well should be as small as possible to prevent septic action from taking place during periods of very low flows. However the wet well must be large enough to allow 4 to 5 minutes lapse between successive starting of pumps.

The time required for successive starts of pumps is determined by the formula

$$t = \frac{V}{Q} + \frac{V}{R-Q}$$

When t = time between successive start of pump

V = Volume of wet well

R = Pumping rate

Q = Discharge of sewage.

Differentiating the above equation shows that the critical point will be arrived at, and the economical size of the wet well will be one minute flow of the pump.

In this particular case, as thescreens and grit channel are being proposed to be installed inside the wet well for the sake of economy, the design basis for such a flow will give a very small capacity of wet well for practical purposes. A larger capacity is therefore adopted with a detention period of 3 to 10 minuted depending upon the size of stations.[1]

The bottom of the wet well will be sloped to reduce the possibility of sewage solids settling out and becoming septic.

The low level of sewage in the wet well will be kept at the level of the start of first pump in order to maintain the prime. The minimum difference between the high level and low level of sewage in the wet well will be kept two feet for proper operation of the liquid level controller.

An emergency by-pass will be provided in the wet well above the level of sea so that under extremely worse conditions when all the pumps including standby and diesel sets are put out

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[1] WPCF Manual of Practice No. 8, <u>op. cit.</u>, p. 46.

of operation, the sewage can flow by gravity into the sea without flooding the pumping stations thus saving the electric motors and other equipment .

It is proposed to install pumps in a dry well adjacent to the wet well. This will have the following advantages :-

(i) Corrosion of the pump is reduced.

(ii) Servicing and maintenance is simple.

The wet well is proposed to be kept open for the following reasons :-

(i) To keep the sewage in contact with atmosphere and prevent production of anaerobic condition.

(ii) To provide workability for the removal of screenings and grit and cleaning of grit channel manually.

The pumps are proposed to be vertical spindle type. The electric motors will be placed inside a control room to be constructed above the dry well. This will eliminate the chances of damage due to accidental flooding of dry well and also facilitate the maintenance of the electric motor.

The size of the control room will depend upon the number and sizes of the pumps required to be installed.

(4) Pumping Units and Pressure Sewer. Vertical spindle, nonclog centrifugal pumps coupled with vertical shaft electric motors will be provided. The pumps will be installed inside the dry well to maintain prime in the pump while the motors will

be installed at the ground surface level in the control room. The switches and controls will be installed in the control room.

There is a wide variation of rate of flow of sewage into the wet well making it difficult to determine the proper relationship between the number of pumps or increment of pumping capacity and wet well volume and flow rate. An estimated variation in sewage flow rate is first plotted for a typical day and the capacity of the pumps will be worked out from the estimated hourly flow diagram. Adequate capacity of the pumps will be kept as stand by for emergency. A diesel engine set will be provided at each pumping station for periods of electric failures.

Automatic float controls will be provided for the pump operation so that the pumps are switched on or off according to the level of sewage in the wet well.

The population at present is quite small and it will take a considerable time when the population reaches to the designed level. Till such time the plant is likely to run underloaded and the efficiencies of the various units are likely to be much reduced. In order to overcome this problem, several units will be designed. The design will be based on the estimated maximum population while only such capacity will be installed as will be found initially necessary. However, in the case of wet well, dry well and pump house the reduction of the capacity will not be economical as the cost does not reduce in the same ratio as that of capacity. Therefore full capacity of wet well, dry well and control room will be installed. The wet well will be divided into two parts with gates to avoid long detention times. The initial installation of the pumping units will be confined to present requirements and space shall be left for installation of the additional pump units when the increased population makes it necessary.

Pressure sewers will be installed for carrying the discharge of sewage from the pumping station.

(iii) Cast iron pipes will be used for the pressure sewer. Both concrete asbestospipe will be effected by saline water present in the soil. The protection and maintenance in case of cast iron is cheaper and easier, and as such C.I. pressure sewer will be adopted.

As the pressure sewer will be subject to constant sudden value closures and sudden high pressures due to water hammer, higher class of cast iron pipes, class 250 pipes with a working pressure of 250 psi will be used.

(a) PUMPING STATION 1

Designed population as per calculation sheet No. 1 = 3,506 persons

The discharge of sewage will be as under :-

Average Domestic Sewage at 40 gpcpd = 3506 x 40 = 140,240 g/d

Maximum Domestic sewage at 2 1/2 times average = 350,600 g/d.

Minimum Domestic sewage at 1/2 time average = 70,120 g/d.

Infiltration at 30 gpcpd = 3506 x 30 = 105,180 g/d

Average daily discharge = 140,240 + 105,180 = 245,420 g/d

Maximum daily discharge = 350,600 + 105,180 = 455,780 g/d

= 0.704 cfs

Minimum daily discharge = 70,120 + 105,180 = 175,300 g/d

= 0.27 cfs.

(1) Design of Screens. As the pumping station is small, provide racks 1 1/2 inch apart and of 1/4 inch thickness.

Effective depth of rack 8" (diameter of sewer)

Area of flow per rock $\frac{8}{12} \times \frac{1.5}{12} = \frac{12}{144}$ sft

Area of Screen $= \frac{Q}{V} = \frac{0.704}{1.5} = 0.47$ sft

No. of racks $= 0.47 \times \frac{144}{12} = 5.6$ or 6

Width of rack = 1.75 x 6 = 10.5"

Provide two rack screen of width 6" each, one in each grit channel.

(2) Design of Grit Channel

Q = 0.704 C.f.s.

Two grit channels have been proposed and the discharge per channel will be

= 0.352 c.f.s.

d = 1 ft

v = 1ft/Sec.

Area of flow $= \frac{0.352}{1} = 0.352$ Sft.

Width of channel $\frac{A}{d} = \frac{0.352}{1} = 0.352$ Sft, or 0.5 ft.

Volume of Grit

The volume of grit is usually 4 cft per million gallon.

The volume of grit per day $= \frac{4 \times 245240}{1,000,000} = 1.0$ cft.

For a clearance int rnal of 7 days, volume of grit = 7 cft.

Depth required for grit collection $= \frac{7}{2 \times 12 \times 0.5}$

$= 0.58$ cft.

Total depth of grit chamber will be :-

Depth of flow	1 ft
Depth for grit	0.58 ft
Free board	0.50 ft
Total	2.08

This design has been made on the basis of peak flow. For the periods of low flows only one channel will be used. The depth of flow will be as under for minimum flow :-

Minimum sewage flow = 0.27 cfs

Velocity = 1.00 ft/Sec

Area of X Section $= \frac{0.27}{1} = 0.27$ sft

Width of Channel = .5 ft

Depth of flow $= \frac{0.27}{.5} = 0.54$ ft.

This is OK

The width of channel arrived as 6", is too small and proper operation of the grit removal and cleanliness would not be practical. So adopt a width of grit channel as 12" to facilitate grit removal. The increase in width is not likely to change the design substantially and all other dimensions are kept same.

Design of Parshall Flume

Q max. = 0.704

Q min. = 0.270

Assume flume throat as 3" or 0.25 ft.

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

$$\frac{0.27}{0.704} = \frac{1.1\left(\frac{0.27}{4.1\times 0.25}\right)^{2/3} - Z}{1.1\left(\frac{0.704}{4.1\times 0.25}\right)^{2/3} - Z}$$

$$0.348 = \frac{0.451 - Z}{0.854 - Z}$$

$$0.616\ Z = 0.123$$

$$Z = 0.2$$

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

$$0.854 - 0.2 = 0.654\ \text{ft}$$

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

$$= 0.451 - 0.2$$

$$= 0.251\ ft$$

Already provided depth is 1.0 ft, it is OK.

Width of Channel

$$\text{Width of Channel} = \frac{Q\ max.}{d\ max. \times V}$$

$$= \frac{0.704}{0.654 \times 1}$$

$$= 1.08$$

Already provided is 2.00

The required width is 10 3/16 inches[2] Therefore

1.00 is OK.

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

$$= 1.5\ (0.704)^{1/3} = 1.33\ ft.$$

Required B = 1.5 ft. Therefore adopt B = 1.5 ft.

From table[2] adopt F = 0.5 ft, G = 1.0 ft.

k = 1 inch, N = 2 1/4 inches

[2] H.E. Babbit, Sewerage and Sewage Treatment, seventh edn., (New York, John Wiley & Hall Ltd., 1953), p. 387.

Total length of grit channel will be

= 12 + 2.0 + 1.5 + 0.5 + 1.0 + 1.0

= 18 ft.

Length of Screens = 4 ft.

Total length of screens and grit channel = 22 ft.

(3) Design of Wet Well, Dry Well and Control Room

Average sewage flow = 10,000 gls/hr.

= 170 gpm

Assume capacity of wet well as 5 minutes average flow

$$\text{Capacity of wet well} = \frac{170 \times 5}{7.5} = 113.2 \text{ cft}$$

Assume an effective depth of 2 1/2 ft.

$$\text{Area of wet well} = \frac{113.2}{2.5} = 45.4 \text{ sft}$$

Adopt a wet well of size 22' x 2 1/2' x 2 1/2'

The grit channel can be easily provided on the larger side of this wet well.

The dry well will be constructed adjacent the wet well for accommodating pumps and valves. The area of this will be 22' x 11'. This will provide enough space for installation of about five pumps to be designed under following subhead.

Ladders and stairs will be provided in the dry as well as wet well for the workman to go into the dry well for maintenance, servicing and repairs of pump and removal of screenings and grit from the wet well.

The wet well will be kept open to the atmosphere from the top, while the dry well will be roofed and a control w room will be constructed to accommodate to electric motors diesel engine, switches and controls. The floor area of this control room will be same as that of dry well.

(4) Design of Pumping Units and Pressure Sewer

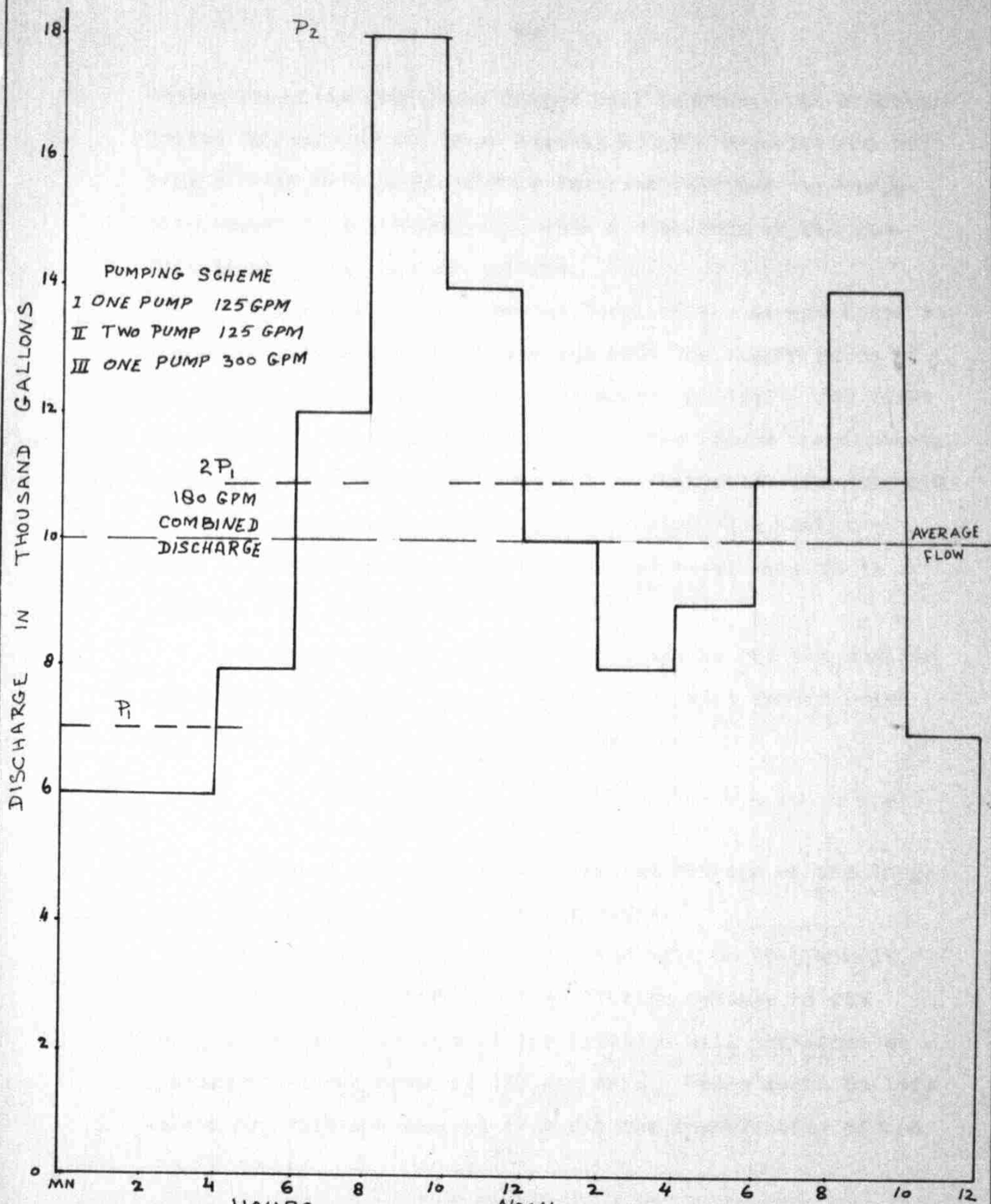
An estimate hourly flow diagram has been prepared as per page 56A. It will be seen from the flow diagram that a suitable arrangement of pumps will be as follows :-

(i) Two pumps of 7500 gph (or 125 gpm)

(ii) One pump of 18000 gph (or 300 gpm)

The operation of pumps will be controlled by automatic float controls.

For low flows the smaller pump of 125 gpm willwork. As this capacity is greater than the minimum flow, the pump will work intermittently by automatic float control in such a way that when the sewage level in the wet well rises to a certain point, float control will operate switch to on position. When the level reaches to the lowest allowable level in wet well, this pump is automatically switched off. During periods of higher flows when the level in the wet well rises to a point, the second pump of 125 gpm will be put into operation by automatic float control, and two pumps of 125 gpm each work together till the level in the sewage falls down to the minimum sewage level. At this point both the pumps are switched off automatically. As the discharge increases with the operation of



ESTIMATED HOURLY SEWAGE FLOW AT P.S. I

second pump, the frictional losses will increase with a proportioned decrease in the total discharge. The calculations of loss of head on page 66A however indicate that the two pumps when operated in parallel will give a discharge of 180 gpm. This is all right for our purpose.

During peak flows when the level of the sewage rises to the highest sewage level in the wet well the bigger pumps of 300 gpm will start operation by automatic control. The float controls will be adjusted such that as the bigger pumps starts operation, the twosmaller pumps will be automatically switched off and the bigger pump will work till such time that the level of sewage goes down to the minimum level when it is automatically shut off.

It is proposed to provide one stand by for the smaller pumps and one for the bigger pumps. The total installation will consist of five pumps as follows :

3 pump sets of 125 gpm each

2 pump sets 300 gpm each

A diesel engine shall be provided for one of the large pumps for emergency during power failure.

The population at the beginning will be smalland it will take some time before the population reaches to its designed value. The initial installation will therefore be confined to three pumps of 125 gpm each. Space shall be left in the dry well and control room for the installation of the remaining two pumps of 300 gpm. It is expected that these three small pumps will provide efficient performance for the first twenty five years, when first larger pump will be felt necessary. While the expected population increase within the next fifteen years will require further installation of one pump of 300 gpm.

One of the three pumps of 125 gpm will be provided with a diesel set. The life of this diesel set will be over within thirty years when a new diesel set will be provided for one of the larger pumps of 300 gpm. The smaller pump will be replaced by another one coupled with electric motor.

Horse Power of Pump and Diameter of Pressure Main

The static head of pumping will be as follows :

Invert of sewer at pumping station = - 8.09

Provide a loss of head of 6" in the s creening and grit channel.

Invert of grit channel at the flume end = - 8.09 - 0.50
- 0.33 = -8.92.

Allow a fall of 6" in the wet well from the invert of grit channel to the high level of sewage in the wet well.

Effective depth of wet well = 2.50 ft

Level of the suction of pump in the wet well will be = - 8.92 - 0.50 - 2.50
= - 11.92

Ground level at the manhole M_1H_5 = 2.00

The pressure sewer will be burried at a depth of about 2' from the surface at a zero level (approx.)

The static pumping head will thus be 11.92 feet.

The length of pressure sewer = 1920 ft.

Maximum discharge = 0.704 cfs.

Adopt a size of pressure sewer as 9"

$$V = \frac{0.704}{\frac{\pi}{4} \times (0.75)^2} = 1.59 \text{ ft/Sec.}$$

This velocity for maximum discharge is too low

Try 8" size

$$V = \frac{0.704}{\frac{\pi}{4} \times (067)^2} = 2.01 \text{ ft/Sec}$$

This is also quite low for maximum discharge

Try next lower size 6"

$$V = \frac{0.704}{\frac{\pi}{4} \times (0.5)^2} = 3.58 \text{ ft/Sec.}$$

This is all right for maximum flow.

Minimum flow is equal to the capacity of small pump

= 125 gpm = 0.278 cfs

Velocity at minimum flow $= \frac{0.278}{\frac{\pi}{2} \times (0.5)^2} = 1.42$ ft/Sec.

This velocity can be allowed. The sediments collected if any during low flows will be washed away during the peak hour flow when the velocity willbe 3.58 ft/Sec

For a lower size of 4" the velocity of $\frac{0.704}{\frac{\pi}{4} \times (1/3)^2}$ =8.05 ft/Sec.

This is too high. Therefore the 6" diameter pipe will be adopted.

Water hammer $= \frac{Vo\ C}{g}$

Vo = Initial velocity

C = Speed of pressure heave

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

K = Bulk Modulus $= 3 \times 10^5$

E = Modulus of Elasticity $= 15\times16^6$

t = Thickness of pipe = 0.5"

D = Diameter pipe = 0.5'

$= \frac{4720}{\sqrt{1+\frac{3\times10^5\times0.5}{15\times10^6\times0.5\times1/12}}}$

$= 4230$

$h = \frac{Vo\ C}{g} = \frac{3.58 \times 4230}{32.2} = 458$ ft.

Total water hammer = 458 + 21.94 (pumping head from page 62)

= 479.94

= 208 psi

Allowable pressure is 250 psi.

Therefore this is OK.

For Larger Pump of 300 gpm

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

$\epsilon = 00085$ From graph 3.

D = 0.5'

$\frac{\epsilon}{D} = .0017$

$V = 3.58$ ft/Sec

$VD'' = 3.58 \times 6 = 21.6$

From graph 3 $f = .0225$

$$hf = .0225 \times 1920 \times \frac{3.58^2}{64.4}$$

$= 8.70$ ft.

Loss of head in Bends and fittings

$$hf = K \frac{V^2}{2g}$$

(i) Bends : 5 bends of 90°

3 bends in 5" pipe $K = 0.3$,[4] $V = 5.16$ ft/Sec

2 bends in 6" pipe $K = 0.28$, $V = 3.58$ ft/Sec

$$hf = 3 \times 0.3 \times \frac{5.16^2}{64.4} + 2 \times 0.28 \times \frac{3.58^2}{64.4}$$

$= 0.37 + 0.11$

$= 0.48$ ft

(ii) Gate value

One gate value 5", $K = 0.10$, $V = 5.16$ ft/Sec

$$hf = 0.10 \times \frac{5.16^2}{64.4}$$

$= 0.04$ ft

(iii) Check value:

One check value 5", $K = 2$, $V = 5.16$ ft/Sec

$$hf = 2 \times \frac{5.16^2}{64.4}$$

$= 0.8$ ft.

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[3] Victor Streeter, Fluid Mechanics, 3rd edn. (New York: McGraw-Hill Book Co.,), p. 220.

[4] Hydraulic Institute, Pipe Friction Manual 1961, 3rd edn. Table 32 (a), (b), p. 28.

Total Loss of head:

Loss of head in pipe = 8.70 ft

Loss of head in bends = 0.48 ft

Loss of head in Gate value = 0.04

Loss of head in Check value = 0.80

Total = 10.02 ft

Total head for pumping:

Static Head = 11.92 ft

Loss of head due to friction = 10.02

Total = 21.94 ft

Performance curves of Peerless pumps shows the following details:

Curve NO; R-1515; size 5 x 5 x 3M; Impellor diameter 10"

Impellor No. V1416, V1417; efficiency 590/0; RPM 860

Sphere 3", Discharge 300 gpm at 21.5 ft with HP 3.0

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

$$Ns = \frac{N\sqrt{Q}}{H^{3/4}} \qquad \frac{860\sqrt{300}}{21.6^{3/4}} = 1490 \text{ r.p.m.}$$

The pump will be radial flow.

For Smaller Pump of 125 gpm

Discharge = 0.278 cfs

Diameter = 6"

$$V = \frac{0.27}{\frac{\pi}{4}(1/2)^2} = 1.42 \text{ ft/Sec.}$$

There is no need to check for water hammer. It is already safe for a velocity of 3 ft/Sec.

Frictional losses are :-

$$hf = fK \times \frac{V^2}{2g}$$

$$\frac{\epsilon}{D} = .0017 \text{ From page 61}$$

$$V = 1.42 \text{ ft/Sec.}$$

$$VD'' = 1.42 \times 6 = 8.52$$

From graph[5] $f = .026$

$$hf = .026 \times 1920 \times \frac{1.42^2}{64.4} = 1.55 \text{ ft.}$$

Loss of head in Bends and Fittings

$$hf = K \frac{V^2}{2g}$$

(i) Bends

6 Bend in all

4 bends 4" and 90° $K = 0.3$[6], $V = 3.19$ ft/Sec

2 bends 6" and 45° $K = 0.18$, $V = 1.42$ ft/Sec.

2 bends 6" and 90° $K = 0.28$, $V = 1.42$ ft/Sec.

$$hf = 4 \times 0.3 \times \frac{3.19^2}{64.4} + 2 \times 0.18 \times \frac{1.42^2}{64.4} + 2 \times 0.28 \times \frac{1.42^2}{64.4}$$

$$= 0.19 + 0.02 + 0.02$$

$$= 0.23 \text{ ft.}$$

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[5] Victor L. Streeter, <u>Fluid Mechanics</u>, <u>op.cit.</u>, p.202.

[6] Hydraulic Institute, <u>op. cit.</u>, p. 28.

(ii) Gate Value

One gate value 4", $K = 0.15$, $\dot{V} = 3.19$ ft/Sec

$$hf = 0.15 \times \frac{3.19^2}{64.4} = 0.02$$

(iii) Check value

One check value 4", $K = 2$, $V = 3.19$ ft/Sec.

$$hf = 2 \times \frac{3.19^2}{64.4} = 0.32$$

Total loss of head

Loss of head in pipe = 1.55

Loss of head in bends = 0.23

Loss of gate value = 0.02

Loss of head in check value = 0.32

Total = 2.12 ft.

Total head of pumping

Static head = 11.92 ft.

Frictional losses = 2.12ft.

Total = 14.04.ft.

Curve V 4291 of Peerless pumps gives following details:

Q = 125 gpm at 13.8 ft at 58% efficiency

RPM 730, size 4 x 4 x 2L

Impeller diameter 9", No. V 2870 and V 2871

Sphere 2 " HP = 3/4

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

<u>For Two Pumps of 125 gpm in Parallel</u>

Plot the characteristic curve of the 6" diameter pipe line in a length of 1920 ft.

(1) for 125 gpm, $H = 14.04$ ft

(2) for 200 gpm, the loss of head is calculated as under:-

$Q = 200$ gpm $= 0.445$ cfs

$$V = \frac{0.445}{\pi/4\,(1/2)^2} = 2.26 \text{ ft/Sec.}$$

$VD" = 2.26 \times 6 = 13.56$

From graph[7] $f = .0245$

$$hf \text{ in pipe} = 0.0245 \times 1920 \times \frac{2.26^2}{64.4} = 3.74 \text{ ft.}$$

Loss of head in bends and other fillings are :-

(i) Bends:-

4 bends 4" diameter, 90°, $K = 0.3$,[8] $V = 5.1$, ft/Sec.

1 bend 4" diameter 45°, $K = 0.18$, $V = 5.1$ ft/Sec.

2 bends 6" diameter 90°, $K = 0.28$, $V = 2.26$ ft/Sec.

$$hf = 4 \times 0.3 \times \frac{5.1^2}{64.4} + 1 \times 0.18 \times \frac{5.1^2}{64.4} + 2 \times 0.28 \times \frac{2.26^2}{64.4}$$

$= 0.48 + .07 + .04$

$= 0.59$

(ii) Gate value

one gate value 4", $K = 0.15$, $V = 5.1$ ft/Sec.

$$hf = 0.15 \times \frac{5.1^2}{64.4}$$

$= 0.06$ ft.

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[7] Victor L. Streeter, Fluid Mechanism, <u>op. cit.</u>, p. 202.

[8] Hydraulic Institute, <u>op. cit.</u>, p. 28.

(iii) Check value   K = 2

$$hf = 2 \times \frac{5.1^2}{64.4} = 0.40$$

Total loss of head :-

Loss of Head in pipe = 3.74

Loss of head in bend = 0.59

Loss of head in Gate value = 0.06

Loss of head in check value = 0.40

Total = 4.79 ft.

The head discharge curve of the above line has been plotted and the characteristic curve of the pump and continuation also plotted in a diagram on page 66A. It appears that two pumps will discharge 180 gpm at 50 percent efficiency. Although the efficiency is quite low the combination may be adopted as the pump sets are quite small and it is not possible to get a pump combination of higher efficiency with low discharge and low heads.

(b) PUMPING STATION II

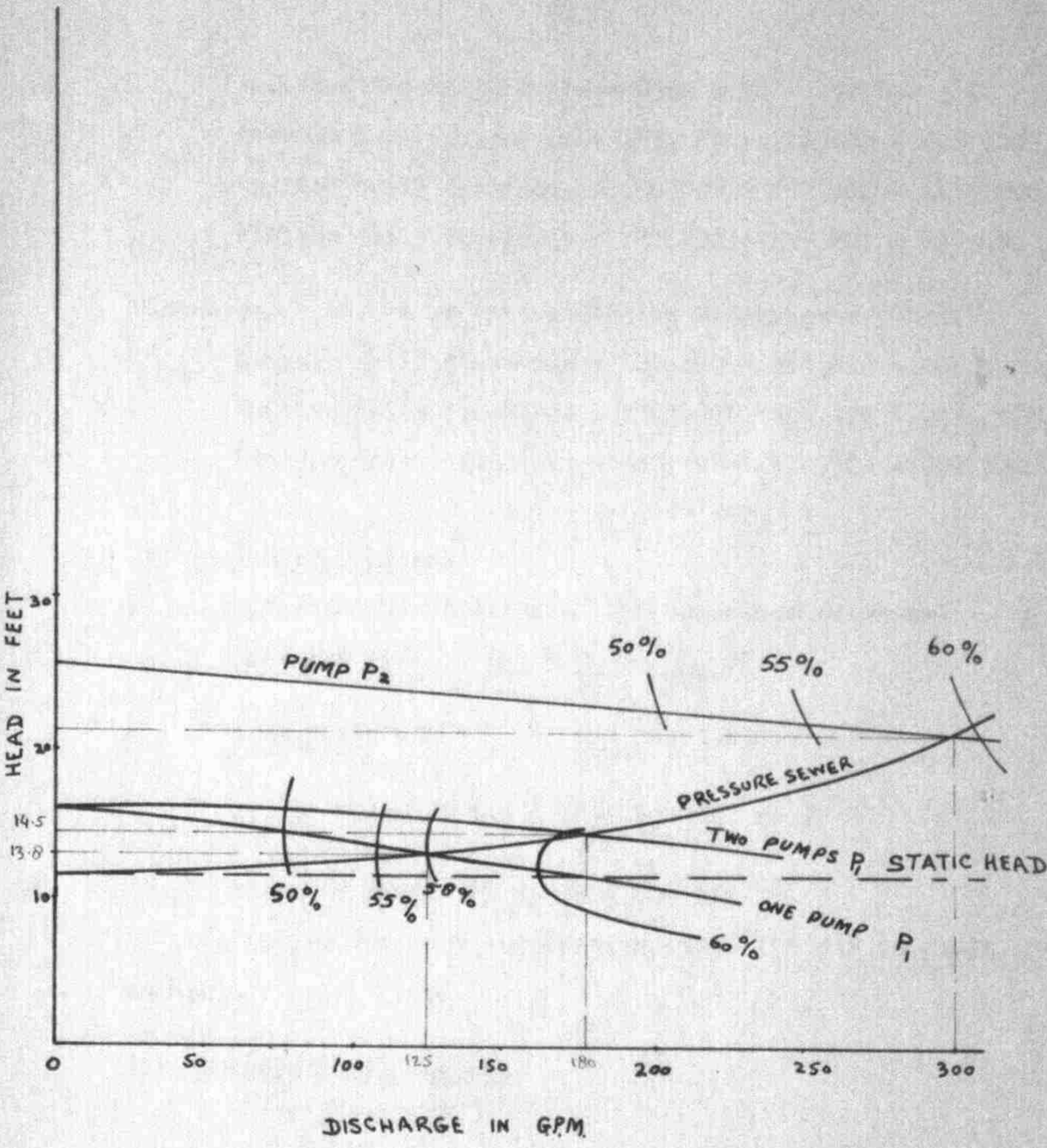
Designed population as per calculation of net work sheet No. 2, page = 8580 persons.

The discharge of sewage at PSII (excluding discharge from PSI) will be as under :-

Domestic sewage at 40 gpcpd = 8580 x 40 = 343,200 g/d

Maximum domestic sewage at 2 1/2 times = 858,000 g/d

Minimum domestic sewage at 1/2 times = 171,600 g/d



CHARACTERSTIC CURVES OF PUMPS AND CAPACITY HEAD CURVE OF PRESSURE SEWER AT PUMPING STATION I

Infiltration at 30 gpcpd = 8580 x 30 = 257,400 g/d

Average daily discharge = 343,200 + 257,400 = 600,600 g/d

Maximum daily discharge = 858,000 + 257,400 = 1115,400 g/d

Minimum daily discharge = 171,600 + 257,400 = 429,000 g/d

Discharge of sewage at PSII including discharge from PSI

Average daily discharge = 600,600 + 245,420 = 846,020 gpd

Maximum daily discharge = 1115,400 +455,780 = 1571,180 gpd

Minimum daily discharge = 429,000 + 175,300 = 604,300 gpd

= 0.93 cft.

(1) Design of Screens

Effective depth of rack 15" (diameter of sewer)

Area per rack $= \frac{15}{12} \times \frac{2}{12} = \frac{30}{144}$ Sft.

Area of Screen $= \frac{Q}{V} = \frac{2.4}{1.5} = 1.6$ Sft

No. of racks $= 1.6 \times \frac{144}{30} = 7.68$ or 8

Width of racks = 2.375 x 8 = 19"

Provide two rack screen each width 12" one for each grit channel.

(2) Design of Grit Channel

$Q = 2.4$ cfs.

Discharge of sewage per grit channel 1.2 cfs

Length = 12 ft

Assume d = 1.25 ft

v = 1 ft/Sec

Area of flow = $\frac{Q}{V} = \frac{1.2}{1} = 1.2$ sft

Width of channel = $\frac{1.2}{1.25} = 0.96$ ft or 1.0 ft.

Volume of Grit

Volume of grit per day at 4 cft per million gallon =

$= \frac{846,020 \times 4}{1000,000} = 3.5$ cft

For a clearance interval of 7 days, volume of grit

$= 3.5 \times 7 = 24.5$ cft.

Depth required for collection of grit = $\frac{24.5}{2 \times 12 \times 1.25}$

$= 0.82$

Total depth of grit chamber :

Depth of flow = 1.00 ft

Depth of grit = 0.82 ft

Free board = 1.00 ft

Total = 2.82 ft.

Design has been made on the basis of peak flow. For the periods of low flows only are channel will be used. The depth of flow for minimum discharge will be as under :=

Minimum sewage flow = 0.93 cfs

Velocity of flow = 1.00 ft/Sec.

Width of channel = 1.00 ft

Depth of flow = 0.93 ft.

This is OK.

Design of Parshall Flume

Q max. = 2.4 cfs

Q min. = 0.93 cfs

Assume flume throat as 6" or 0.5 ft

$$\frac{Q \text{ min.}}{Q \text{ max.}} = \frac{1.1\left(\frac{0.93}{4.1W}\right)^{2/3} - Z}{1.1\left(\frac{2.4}{4.1W}\right)^{2/3} - Z}$$

$$\frac{0.93}{2.4} = 0.388 = \frac{0.648 - Z}{1.22 - Z}$$

$$0.648 - Z = .473 - .388Z$$

$$Z = 0.286$$

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

$$= 1.22 - 0.286$$

$$= 0.934 \text{ ft}$$

Already provided 1.25 ft

It is OK

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

$$= 0.473 - 286$$

$$= 0.187 \text{ ft}$$

It is allowable

Width of Channel $= \dfrac{Q \text{ max.}}{d \text{ max.} \times V.}$

$$= \frac{2.4}{1 \times 1.25} = 1.92 \text{ ft}$$

Already adopted 2.0

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

$$= 1.5\ (2.4)^{1/3} = 2 \text{ ft.}$$

It is OK

Required B as per table[3] = 2.0

Adopt[9] B = 2', F= 1.0 ft, G = 2 ft, K = 3" N = 4 1/2"

Total length of grit channel = 12 + 2.0 + 1.0 + 2.0 + 1

= 18 ft.

Length of Screen and perforated plate-form = 4 ft.

Total length of Screen and grit channel = 22 ft.

(3) Design of Wet Well, Dry Well and Control Room

Average daily discharge of sewer = 35,000 gph.

= 583 gpm.

Assume a capacity of wet well as 3 minutes average flow

Capacity of well well $= \frac{583 \times 3}{7.5} = 233$ cft

Assume an effective depth of sewage as 3 ft.

Area of wet well $= \frac{233}{3} = 78$ sft

Adopt a size of wet well as 22' x 4' x 3'

The grit channel can easily be provided on the larger side

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[9] H.E. Babbit, Sewerage and Sewage Treatment, 1953, (New York: John Wiley & Son, Inc., 1953), p. 387.

of wet well.

The dry well will be constructed adjacent to the wet well to accommodate the pumps. The area of the dry wellwill be 22' x 11'. This will give enough space to provide about five pumps expected to be installed at the station and for servicing and repairs of the pumps. Ladders and stairs will be provided in the wet as well asdry well to remove screening and grit from the wet well and for the maintenance of the pumps in the dry well.

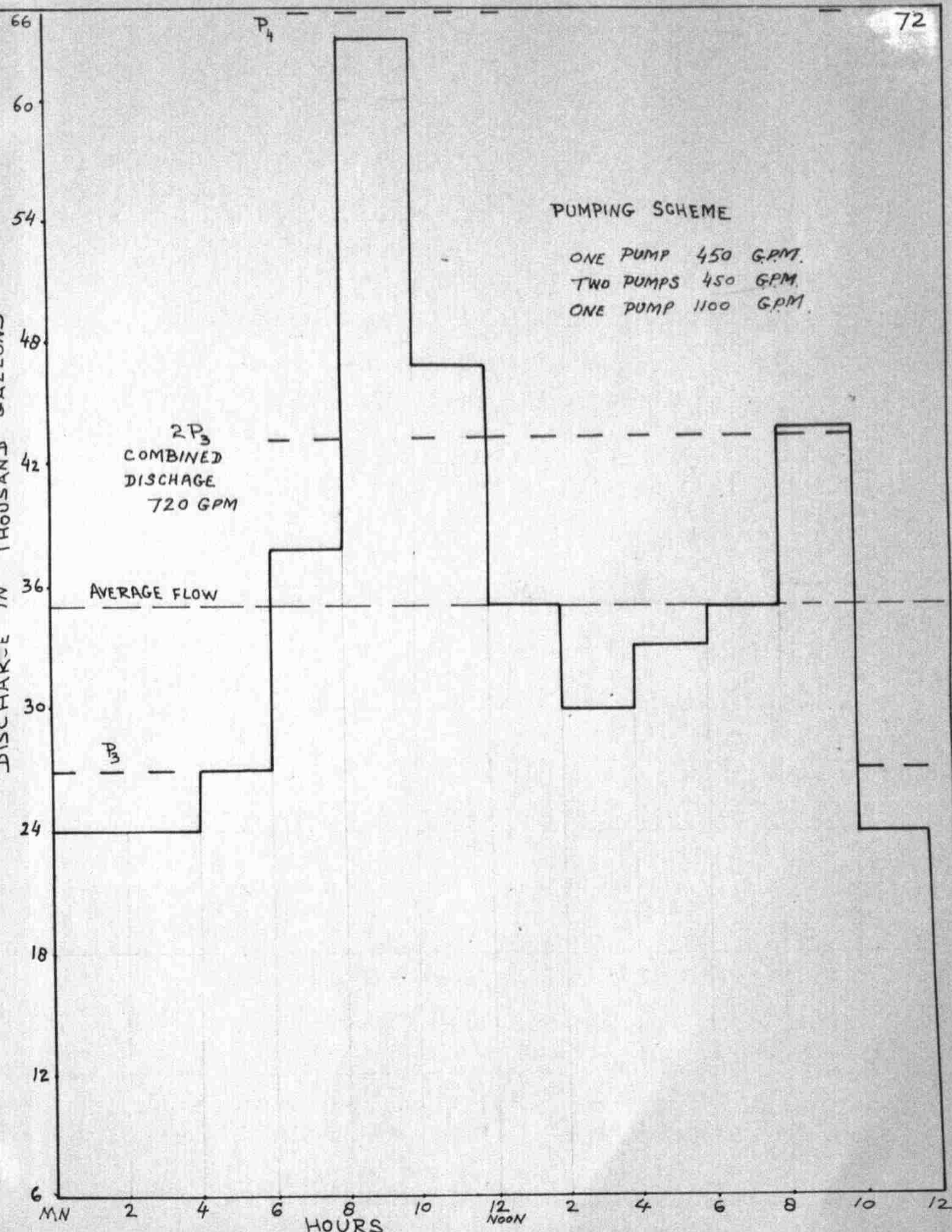
The wet well will be kept open from the top while the dry well will be roofed and a control room will be constructed over it to accommodate the electric motors, diesel engine set, switches and controls.

(4) Design of Pumping Units and Pressure Sewer

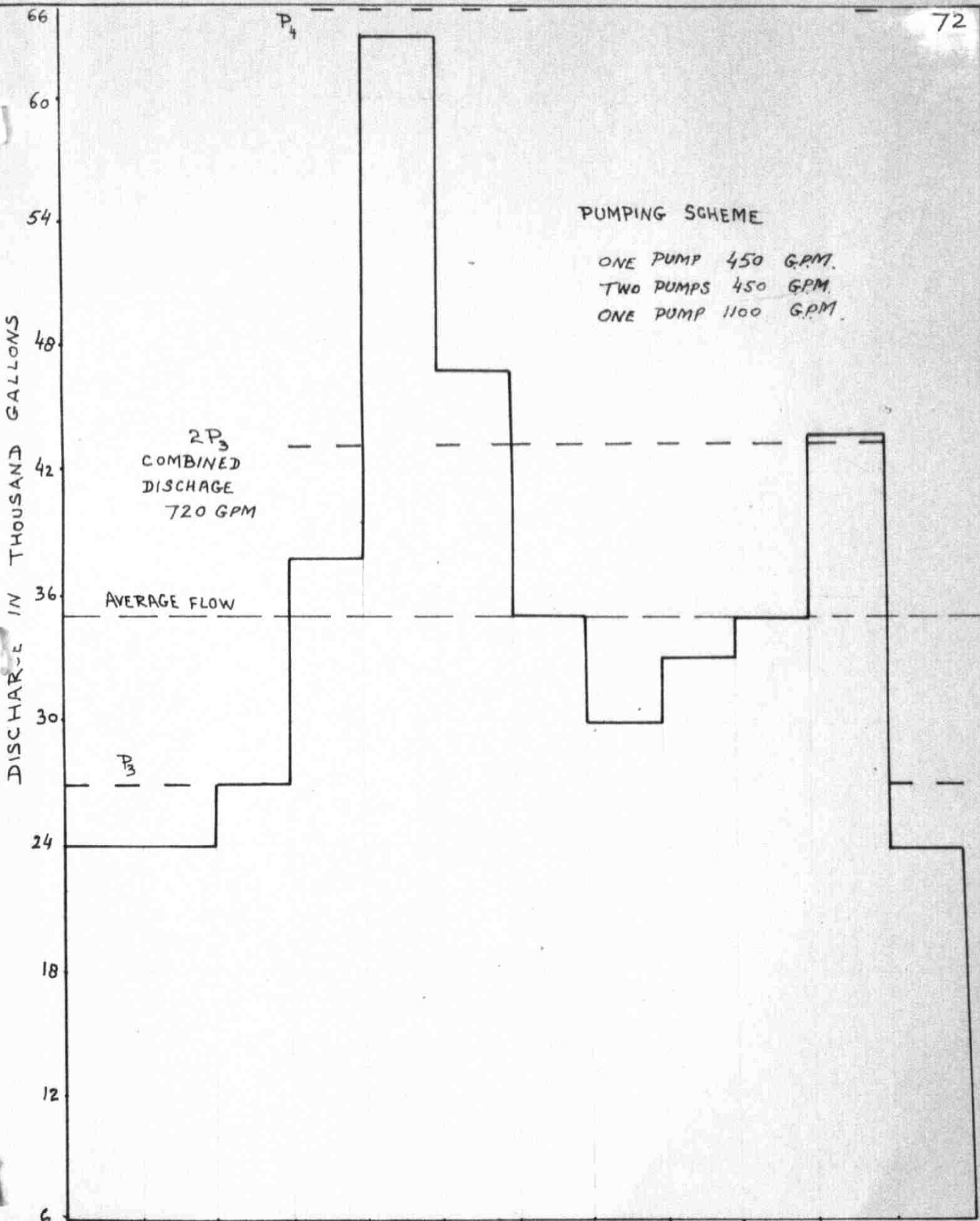
An estimated hourly flow diagram has been prepared as per page 72. It will be seen from the diagram that a suitable arrangement of pumps will be two pumps of 450 gpm each and one pump of 1100 gpm.

The operation of these pumps will be controlled by automatic float controls.

During periods of low flows only one pump of 450 gpm will work. As the level of sewage falls below the minimum allowable level in the wet well, this pump will stop automatically and will start again as the level rise above to a certain point.



ESTIMATED HOURLY SEWAGE FLOW AT P.S. II



ESTIMATED HOURLY SEWAGE FLOW AT P.S. II

As the flow of sewage into the wet well increases beyond the pumping capacity of the first pump, the level will rise and when this level reaches to another point, the float control will cause the second pump of 450 gpm to start operation. As the level goes down to the lowest level of the sewage, both pumps will be automatically switched off by float controls. As the discharge increases with the operation of second pump, the frictional losses will increase with a proportional decrease in total discharge of the two pumps. The calculation of loss of head on page 81 indicate that the combined discharge of the pumps will be 720 gpm.

During peak flows when the level of sewage reaches to the highest designed level of sewage in the wet well, the third pump of 1100 gpm will start operation and will continue to work till the level goes down to the lowest level of sewage in the wet well. The automatic controls will be adjusted such that as the bigger pump starts operation the two small pumps will be switched off automatically. The cycle will thus be continued. It is proposed to provide 3 pumps each of 450 gpm and two pumps 1100 gpm onestand by for emergencies. A diesel engine will be provided with the larger pumps for periods of electric failure.

This pumping station will serve the following areas :-

(i) Old Town. Where the population intensity is already large and no substantial increase in population is expected.

(ii) Developed area under previous development plan where a large portion of construction has been done and there is a greater trend of construction of houses in this area, being just adjacent to the already populated area. Therefore full capacity of the pumping installation is recommended.

The whole city has not been provided at present fully with water supply system and it will take some time before the water supply system is installed and the houses in the old city are provided with sanitary and plumbing installation. Almost same time will be required for the construction of sewerage system as well. It is expected that by the time the sewerage system is completed, the water supply system would have completed and the houses will have been provided with the sanitary and plumbing installation.

Horse Power and Diameter of the Pressure Sewer

The static pumping head will be as follows :-

Invert of sewer at pumping station = -7.01

Provide a loss of head of 6" in the screens and grit channel

Invert of grit channel at the flume end = - 7.01 -0.5 - 0.25

= - 7.76

Allow a fall of 6" in the wet well from the invert of grit channel to the high level of sewage in the wet well

Effective depth of wet well = 3 ft.

Level of suction of pump in the wet well will be

$= -7.76 - 0.5 - 3.0$

$= - 11.26$

Ground level at the manhole $S_1H_2 = 3.10$

The pressure sewer will be burried at a depth of 2'

from the surface at a level of 1.10.

The static head will thus be $11.26 + 1.10$

$= 12.36$ ft.

Length of pressure sewer = 1710 ft.

Maximum discharge = 1100 gpm = 2.44 cfs

(maximum capacity of pump)

Minimum discharge

(Minimum capacity of pump) = 450 gpm = 1.0 cfs

For a diameter of pressure main 8"

Velocity = $\frac{2.44}{\pi/4 x (2/3)^2}$ = 7 ft/sec. This is too high, Try a larger diameter.

Adopt a size of pressure main 10"

$$V = \frac{2.44}{\frac{\pi}{4} x \left(\frac{10}{12}\right)^{12}} = 4.48 \text{ ft/Sec.}$$

Water hammer = $\frac{Vo\ C}{g}$

$$C = \frac{4720}{\sqrt{1 + \frac{KD}{Et'}}} = \frac{4720}{\sqrt{1 + \frac{3 \times 10^5 \times 10}{15 \times 10^6 \times 0.6}}}$$

$= 4080$

$$h = \frac{4080 \times 4.48}{32.2} = 518 \text{ ft.}$$

total water hammer = 518 x 12.36 (total static head)

= 530.36 ft.

= 233 psi.

It is within allowable of 250 psi. It is OK

$$\text{Minimum velocity} = \frac{1}{\frac{\pi}{4} \times \left(\frac{10}{12}\right)^2} = 1.84 \text{ ft/Sec.}$$ It is OK

For the larger pump of 1100 gpm

$$hf = fL \times \frac{V^2}{2g}$$

V = 4.48 ft/sec.

D = 10"

ϵ = .00085

$$\frac{\epsilon}{D} = \frac{.00085}{10/12} = .00102$$

VD" = 4.48 x 10 = 44.8

From graph[10] = .021

$$hf = .021 \times 1710 \times \frac{4.48^2}{64.4} = 11.2 \text{ ft.}$$

10. Victor L. Streeter Fluid Mechanics p. 220

Loss of head in bends and fittings

5 bends 10" diameter 90°, K = 0.25[11], V = 4.48 ft/sec.

1 gate value 10" diameter K = .06, V = 4.48 ft/sec.

1 check value 10" diameter K = 2.0, V = 4.48 ft/sec.

$$hf = (0.25 \times 5 + 1 \times .06 + 1 \times 2) \times \frac{4.48^2}{64.4}$$

$$= 1.01 \text{ ft.}$$

Total loss of head = 11.2 + 1.01

= 12.21 ft.

Total pumping head

Static head = 12.36

Frictional loss of head = 12.21

Total = 24.57 ft.

The characteristic curve of Peerless pumps indicate the following:

Curve NO; V 3958 - 4 size 8 x 8 x 4M and 6 x 8 x 4M

Discharge 1100gpm under 25 ft at efficiency 74 o/o

Sphere 4", imperler diameter $14\frac{1}{2}$", NO: V 1825 and V 1826

rpm 695; HP : 10.

$$NS = \frac{N\sqrt{Q}}{H^{3/4}} = \frac{695\sqrt{1100}}{25^{3/4}} = 2080 \text{ rpm}$$

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11. Hydraulic Institute, Pipe Friction Manual, p. 28.

The pump will be radial flow type.

<u>For smallerpump of 450 gpm</u>

Discharge = 1.0 cfs.

Diameter = 10"

$$V = \frac{1}{\frac{\pi}{4}\left(\frac{10}{12}\right)^2} = 1.84 \text{ ft/sec.}$$

There is no need to check for water hammer. It is already safe against a velocity of 4.48 ft/sec.

Frictional losses are

$$hf = f\ L \times \frac{V^2}{2g}$$

$$\frac{\epsilon}{D} = \frac{.00085}{10/12} = .001$$

$$VD'' = 1.84 \times 10 = 18.4$$

gron graph[12] = 0.022

$$\text{Loss of head in pipe } hf = 0.022 \times 1710 \times \frac{1.84^2}{64.4} = 1.97 \text{ ft.}$$

Loss of head in bends and fittings

7 bends 90° diameter 10", K = 0.25[13], V = 1.84 ft/sec.

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12. Victor L. Streeter Fluid Mechanics p. 220.
13. Hydraulic Institute, Pipe Friction Manual p. 28.

1 gate value diameter 10"   K = .06, V = 1.84 ft/sec.

1 check value diameter 10"   K = 2   V = 1.84 ft/sec.

$$hf = (7 \times 0.25 + 1 \times 0.06 + 2 \times 1) \times \frac{1.84^2}{64.4}$$

$$= 0.2 \text{ ft.}$$

Total loss of head

Loss of head in pipe = 1.97

Loss of head in bends and fitting = 0.20

Total = 2.17 ft.

Total pumping head

Static head = 12.36

Frictional loss of head = 2.17

Total = 14.53 or 15 ft.

The curve NO R - 1516 of M/S peerless pumps give the following details:

450 gpm at 16 ft. head under 59 % efficiency.

rpm 1160; Impeller diameter 8", NO; V 1416 and V 1417

size 4 x 5 x 3 M, 5 x 5 x 3 M

HP 3.0

This pump is suitable and this or any other pump of equivalent characteristic may be adopted.

$$NS = \frac{N\sqrt{Q}}{H^{3/4}} = \frac{1160\sqrt{450}}{15^{3/4}} = \frac{1160 \times 21.2}{7.62}$$

$$= 3200$$

The pump will be radial flow type

For the two pumps of 450 gpm in parallel

A characteristic curve of 10" diameter pipe line for a length of 1710 ft. is plotted as follows:-

(1) for Q = 450 gpm, H = 2.17 ft.

(2) for Q = 600 gpm, the loss of head will be as under:-

$$Q = 600 \text{ gpm} = 1.33 \text{ cfs.}$$

$$V = \frac{1.33}{\pi/4 \times (10/12)^2} = 2.42 \text{ ft/sec.}$$

$$VD = 2.42 \times 10 = 24.2$$

from graph[14] $f = 0.021$

$$hf = .021 \times 1710 \times \frac{(2.42)^2}{64.4}$$

$$= 3.26 \text{ ft.}$$

Loss of head in bends and values (as per page 79).

$$= (7 \times .25 + 1 \times .06 + 2 \times 1) \times \frac{2.42^2}{64.4}$$

$$= 3.81 \times \frac{2.42^2}{64.4}$$

14. Victor L. Streeter Fluid Mechanics , p. 220.

$= 0.35$ ft.

Total loss of head $= 3.26 + 0.35$

$= 3.61$ ft.

(3) For Q = 750 gpm, the loss of head will be as under:-

$Q = 750 \text{ gpm} = 1.67 \text{ cfs}$

$$V = \frac{1.67}{\pi/4 \times (10/12)^2} = 3.03 \text{ ft/sec.}$$

$VD = 30.3$

from graph[15] $f = 0.021$

$$hf = 0.021 \times 1710 \times \frac{3.03^2}{64.4} = 5.1 \text{ ft.}$$

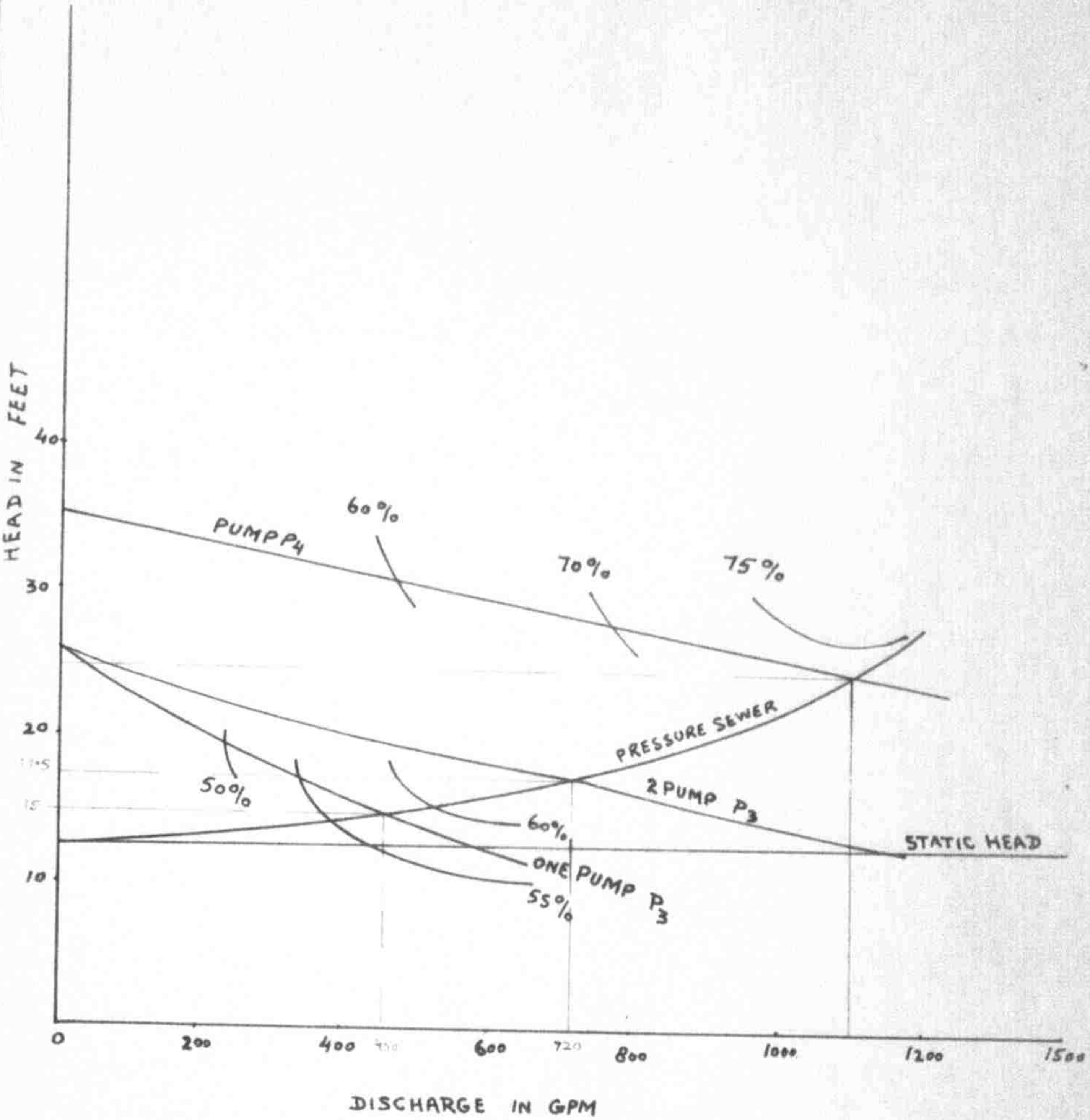
Loss of head in bends, and values:= $3.81 \times \frac{3.03^2}{64.4} = 0.55$ ft. (as per page 80).

Total loss of head $= 5.10 + 0.55 = 5.65$ ft.

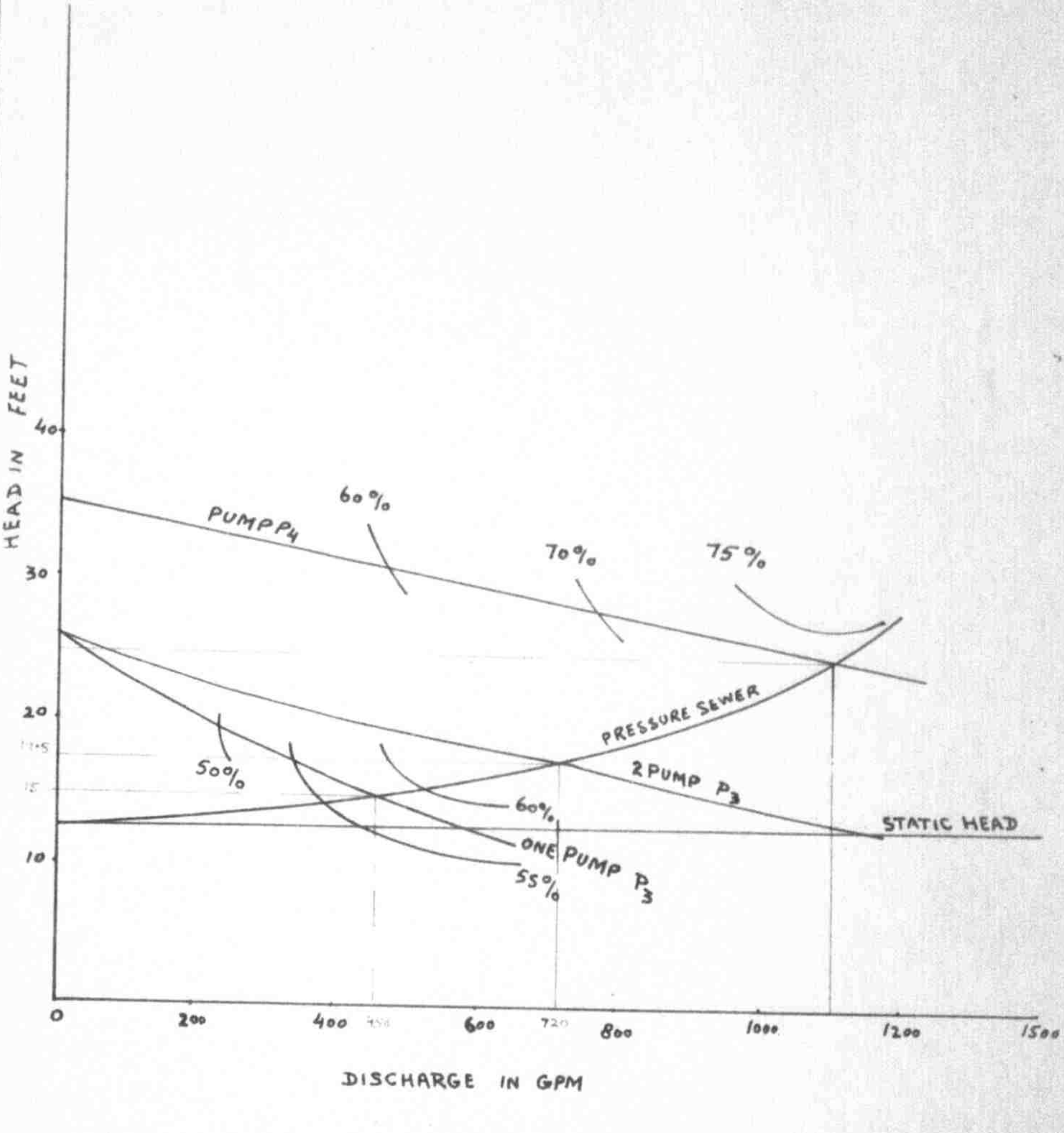
The head discharge curve of the 10" diameter pipe line has been plotted and the characteristic curve of the pumps and combination also plottedin a diagram on page 82. It appears that the two pumps will discharge 720 gpm at 17.5 ft.

The efficiency of the combined pumps will be that of single pump working for 360 gpm under of head of 17.5 ft. and it is from the same curve 57%. The performance of the pumps

15. Victor L. Streeter, Fluid Mechanics, p. 220.



CHARACTERSTIC CURVES OF PUMPS P_3 AND P_4 AND CAPACITY HEAD CURVE OF PRESSURE SEWER AT PUMPING STATION II



CHARACTERSTIC CURVES OF PUMPS P_3 AND P_4 AND CAPACITY HEAD CURVE OF PRESSURE SEWER AT PUMPING STATION II is satisfactory and the combination is suitable for adopting.

(C) PUMPING STATION III

Designed population as per calculations of net work sheet NO: 4 = 8334 persons. The discharge of sewage at PSIII excluding discharge from PS II will be as under:-

Domestic sewage at 40 gpcpd = 8334 x 40 = 333,360 gpd.

Maximum domestic sewage at $2\frac{1}{2}$ times average = 833,400 gpd.

Minimum domestic sewage at $\frac{1}{2}$ times average = 166,680 gpd.

Infilteration at 30 gpcpd = 8334 x 30 = 250,020 gpd.

Average daily discharge = 333,360 + 250,020 = 583,380 gpd.

Maximum daily discharge = 833,400 + 250,020 = 1083420 gpd.

Minimum daily discharge = 166,680 + 250,020 = 416,700 gpd.

Discharge at PS III including discharge from PSII

Average daily discharge = 846,020 + 583,380 = 1,429,400 cfs.

Maximum daily discharge = 1571,180 + 1083,420 = 2654,600cfs=40cfs.

Minimum daily discharge = 604,300 + 416,700 = 1021,000cfs=1.55cfs.

(I) Design of Screens

Effective depth of rack = 18" (diameter of sewer)

Area per rack = $\frac{18}{12} \times \frac{2}{12} = \frac{36}{144}$ sft

Area of screens $= \frac{Q}{V}$

$$\frac{4.0}{1.5} = 2.67 \text{ sft}$$

No. of racks $= 2.67 \times \frac{144}{36} = 12.8$ or 13

Width of racks $2.375 \times 13 = 30.9''$

Provide two racks each width 1.75 ft. for each grit channel.

Design of grit channel

$Q = 4.0$ cfs

Discharge of sewage per grit channel = 2.0 cfs

$d = 1.50$ ft.

$v = 1$ ft/sec.

Area of flow $= \frac{Q}{V} = 2.0$ sft

Width of channel $= \frac{2.0}{1.50} = 1.35$ ft.

Provide width of channel = 1.75 ft.

Volume of grit per day = 4 cft per million gallon $= \frac{1,429,400}{1,000,000} \times 4 = 5.7$ cft.

For a clearance interval of 7 days volume of grit $= 5.7 \times 7 = 39'9$ cft.

Depth required for grit collection $= \frac{39.9}{2 \times 12 \times 1.75} = 0.95$

Total depth of grit chamber:-

Depth of flow = 1.50 ft.

Depth of grit = 0.95 ft.

free board = 1.00

Total = 3.45 ft.

Design has been made on the basis of peak flow. For the periods of low flows only one channel will be used. The depth of flow for minimum discharge will be as under:-

Minimum sewage flow = 1.55 cfs

Width of channel = 1.75 ft

Depth of flow = $\frac{1.55}{1.75 \times 1} = 0.88$ ft

This is OK

Design of Parshall flume

Q max. = 4.0 cfs

Q min. = 1.55 cfs

Assume flume throat as 9" or 0.75 ft.

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

$$\frac{1.55}{4.0} = \frac{1.1\frac{(1.55)^{2/3}}{(4.1 \times 0.75)^{2/3}} - Z}{1.1\frac{(4.0)^{2/3}}{(4.1 \times 0.75)^{2/3}} - Z}$$

$$0.388 = \frac{695 - Z}{1.31 - Z}$$

$$0.695 - Z = 0.508 - 0.388\ Z$$

$$Z = 0.306$$

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

$$= 1.31 - 0.306$$

$$= 1.004\ ft.$$

Already provided 1.25 ft.

It is OK

$$d\ min. = 1.1\left(\frac{Q\ min.}{4.1\ W}\right)^{2/3} - Z = 0.695 - 0.306 = 0.389\ ft.$$

It is allowable

$$\text{Width of channel} = \frac{Q\ max.}{d\ max. \times V} = \frac{4.0}{1.25 \times 1} = 2.67\ ft.$$

already provided is 3.5 It is OK

$B = 1.5 \times (Q\ max.)^{1/3} = 1.5 \times (4.0)^{1/3} = 2.38$ ft.

B required as per table[1] = 2' — 10" or 3 ft.

From table[16] adopt, F = 1.0 ft, G = 1.5 ft, K = 3", N = $4\frac{1}{2}$"

Total length of grit channel = 12.0+3.0+1.0+1.5+1.0 = 18.5 ft.

Length of Screen 35 ft. including perforated metal plate form.

Total length of Screens, grit chamber and Parshall flume =22ft.

(3) Design of wet well, dry well and control room

Average daily discharge of sewer = 39,000 gph.
= 985 gpm.

---

16. Harold E. Babbit, Sewerage and Sewage Treatment, Seventh Ed. John Wiley & Sons, Inc., New York 1952, p. 387.

Assume capacity of wet well as 3 minutes average flow

Capacity of wet well = $\frac{985 \times 3}{7.5}$ = 394 cft.

Assume an effective depth of sewage as $3\frac{1}{2}$ ft.

Area of wet well = $\frac{394}{3.5}$ = 112.5 sft.

Adopt a size of wet well as 22' x 5' x $3\frac{1}{2}$'.

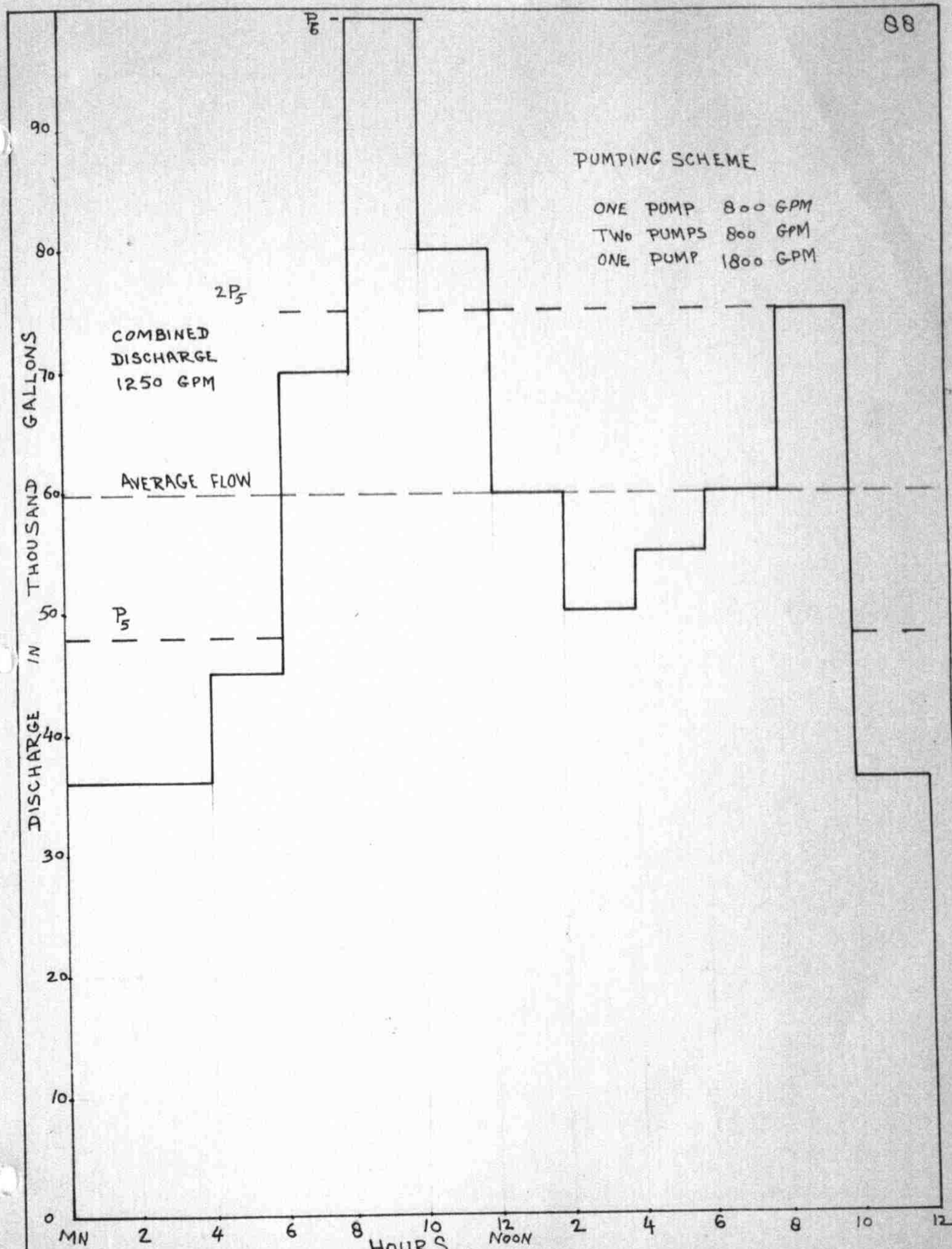
The grit chamber will be provided along the larger side of the wet well.

The dry well will be constructed adjacent to the wet well for accomodation of pumps. The area of the wet well will be 22' x 11'. This will give enough space to provide about five pumps expected to be installed at the station and for servicing and repairs of the pumps. Ladders and stairs will be provided in the wet well as well as in dry well to remove screenings and grit from the wet well and for the maintenance of the pumps in the dry well.

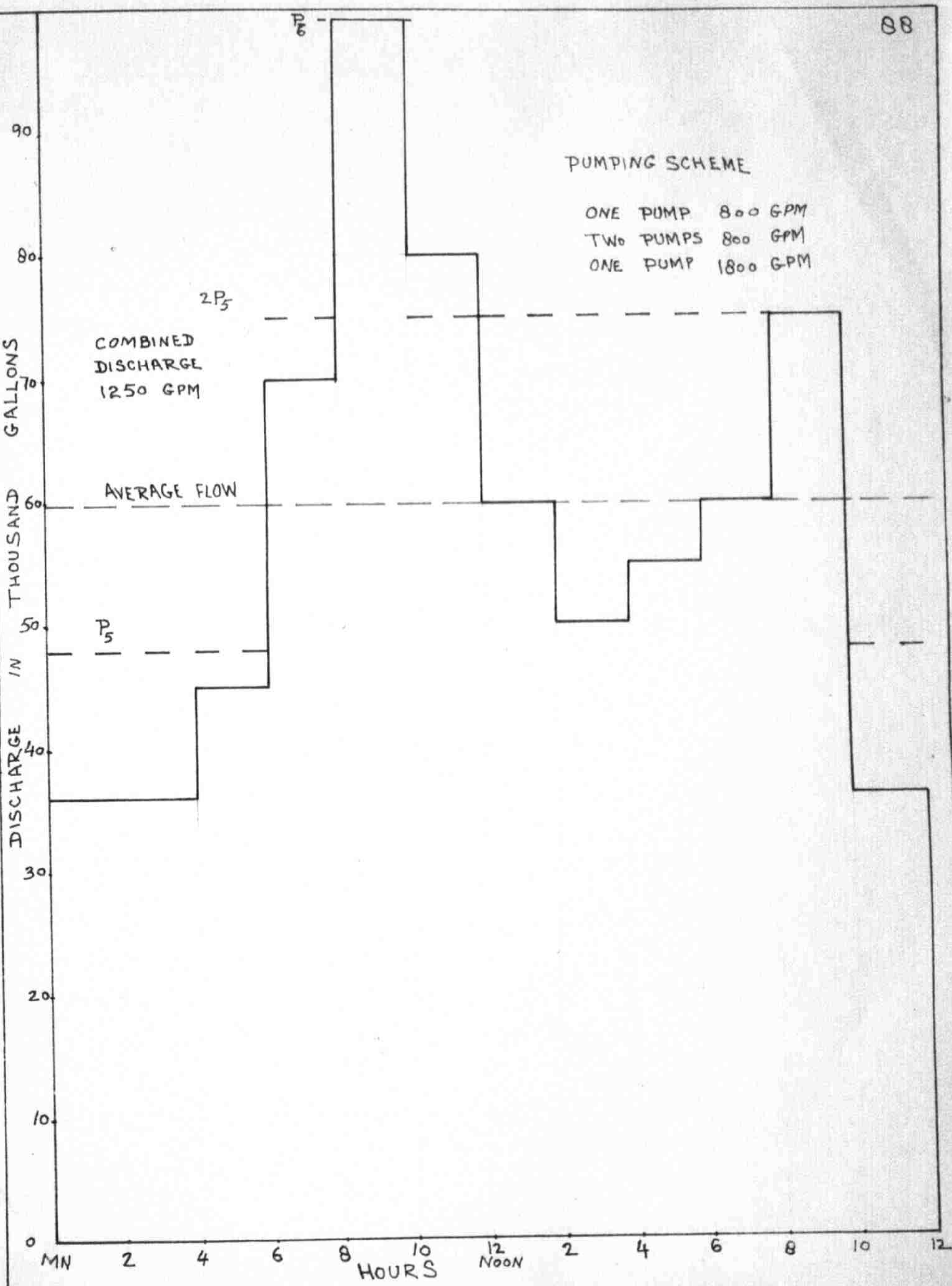
The wet well will be open from the top while the dry well will be roofed and controll room will be constructed over it to accomodate the electric motors, diesel engine set, switches and controls.

(4) Design of Pumping Units and Pressure Sewer

An estimated hourly flow diagram has been prepared as per page 88. It will be seen from the diagram that a suitable arrangement of pumps will be two pumps of 800 gpm each, and one pump of 1800 gpm.



ESTIMATED HOURLY SEWAGE FLOW AT PS III



ESTIMATED HOURLY SEWAGE FLOW AT PS III

The operation of these pumps will be controlled by automatic float controls.

During the period of low flows only one pump will work. In case however the level of sewage falls below the minimum allowable level of sewage in the wet well, this pump will stop automatically and will start again as the level of sewage rises to a starting level. As the flow of sewage into the wet well will increase the level will rise and when this level reaches to a certain height in the wet well, the float control will cause the second pump to start operation. As the level goes down to the lowest allowable level both these pumps will be put out of operation by automatic control. As the discharge increases with the operation of second pump, the frictional losses will increase with a proportional decrease in total discharge of two pumps. The calculation of loss of head and discharge on page 97 indicate that the combined discharge will be 1250 gpm. During the peak flows when the level reaches to the highest designed level of the wet well the third pump of 1800 gpm will start operation and will continue to operate till the level goes down to the lowest level of sewage in the wet well, when third pump will be put out of operation by automatic float control. The cycle will be continued. The automatic controls will be adjusted such that as the larger pump start operation, the two small pumps will be automatically switched off.

Three pumps of 800 gpm were tried. Characteristic curve of the combination of three pumps has been drawn on page 97A. But it was found that this combination delivers only 1410 gallons at the head of 31.5 ft. So this combination is not suitable.

This pumping station will serve practically the whole town. The population at the initial start of sewerage system will be much less, about half and it will take quite a considerable time when the designed population will be attained. Therefore only part of the total designed capacity will be installed. It is recommend that three pumps of 800 gpm each may be installed initially. Two pumps will work and one will remain as stand by. It is expected that these three pumps will work efficiently for about twenty five years. One of these pumps will be provided with a diesel set for emergency during power failures. It is expected that if the existing condition and pace of development continues, one pump of 1800 gpm will be required after thirty five years. This pump will be provided with diesel set as the life of the previous diesel set might have expired by that time. The smaller pump will be replaced by another one coupled with electric motor.

Horse Power and Diameter of Pressure Sewer

The static pumping head will be as under:-

Invert of sewer at the pumping station = - 10.10

Provide a loss of head of 6" in the Screens and grit channel

Invert of grit channel at the flume end = - 10.10 - 0.50

= - 10.60

Allow a fall of 6" in the wet well

From the invert of grit channel to the high level of sewage in the wet well. Elevation of low sewage level in the wet well.

= - 10.60 - 0.50 - 3.50

= - 14.60

Assume depth of sewage in the lagoon as 4 ft.

and loss of head in the distribution box as 6 inches

level of the ground at lagoon = 3.28

Sewage discharge level in the distribution box

= 3.28 + 4.0 + 0.5

= 7.78 ft.

The static head of pumping will be = 14.60 + 7.78

= 22.38 ft.

Length of pressure sewer = 3220 ft.

Maximum discharge = 1800 gpm = 4.00 cfs.

Minimum discharge = 800 gpm = 1.78 cfs.

Adopt size of pressure main 12"

$$V = \frac{4}{\pi/4 \times 1} = 5.08 \text{ ft/sec.}$$

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

$$C = \frac{4720}{\sqrt{1 + \frac{KD}{Et'}}} = \frac{4720}{\sqrt{1 + \frac{3 \times 10^5 \times 12}{15 \times 10^6 \times 0.6}}}$$

$$= 3990$$

$$h = \frac{3990 \times 5.08}{32.2} = 630 \text{ ft.}$$

$$= 272 \text{ psi}$$

This is more than the allowable pressure of 250 psi

Adopt size of pressure main 14"

$$V = \frac{4}{\pi/4 \left(\frac{14}{12}\right)^2} = 3.74 \text{ ft/sec.}$$

$$C = \frac{4720}{\sqrt{1 + \frac{KD}{Et'}}} = \frac{4720}{\sqrt{1 + \frac{3 \times 10^5 \times 14}{15 \times 10^6 \times 0.6}}} = 3900$$

$$h = \frac{3900 \times 3.74}{32.2} = 454 \text{ ft.}$$

Total water hammer = 454 + 22.38 (total static head)

= 476.38 ft.

= 207 psi.

It is within allowable limit of 250 psi and is OK

<u>For the larger pump of 1800 gpm</u>

$V = 3.74$ ft/sec.

$D = 14''$

$$\frac{\epsilon}{D} = \frac{0.00085}{14/12} = 0.00099 \text{ or } \cdot 001$$

$$VD'' = 3.74 \times 14 = 52.4$$

From graph[17] $f = 0.021$

$$hf = fL \times \frac{V^2}{2g} = 0{\cdot}021 \times 3220 \times \frac{3.74^2}{64.4} = 14.70$$

Loss of head in bends and valves.

$V = 3.74$ ft/sec.

(i) Bends 4 Nos., From graph[18] $K = 0.23$

(ii) Gate value 1 Nos., $K = 0.04$

(iii) Check value 1 Nos., $K = 2.0$

$$\text{Loss of head} = (4 \times 0.23 + 1 \times .04 + 1 \times 2.0)\ \frac{3.74^2}{64.4}$$

$$= 0.64 \text{ ft.}$$

Total loss of head

Loss of head in pipe = 14.70 ft.

Loss of head in Bends & value = <u>0.64</u> ft.

Total = 15.34ft.

Total pumping head

Static head = 22.38 ft.

Frictional loss = <u>15.34</u> ft.

= 37.72 ft.

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17. Victor L. Streeter, Fluid Mechanics, p. 220

18. Hydraulic Institute, Pipe Friction Manual, p. 28.

Curve No: V 4330 of Peerless pumps give the following details:

Q = 1800 gpm at 38 ft under efficiency 72 %;

Dimater of impeller $15\frac{1}{2}$" ; No: V 2260 and V 2261; Sphere 4-3/4";

Size 8 x 10 x $\frac{43}{4}$ S and 10 x 10 x $\frac{43}{4}$; RPM, 860; HP = 25;

This is a suitable pump

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

$$NS = \frac{860 \sqrt{1800}}{38^{\frac{3}{4}}} = 2380 \text{ RPM}.$$

The pump will be radial flow type.

For smaller pump of 800 gpm

diameter = 14", discharge = 1.78 cfs

$$V = \frac{1.78}{\pi/4 \times \left(\frac{14}{12}\right)^2} = 1.67 \text{ ft/sec}.$$

$$\frac{\epsilon}{D} = .001$$

$$VD" = 1.67 \times 14 = 23.4$$

from graph[19] $f = 0.0215$

$$hf = .0215 \times 3220 \times \frac{1.67^2}{64.4} = 3.00 \text{ ft}.$$

Loss of head in Bends and values.

$V = 1.67$ ft/sec.

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19. Victor L. Streeter, Fluid Mechanics, p. 220.

(i) Bends 4 Nos. $K = 0.23$[20]

(ii) Gate value 1 Nos. $K = 0.04$

(iii) Check value 1 Nos. $K = 2.0$

$$\text{Loss of head} = (4 \times 0.23 + 1 \times 0.04 + 1 \times 2) \times \frac{1.67^2}{64.4}$$

$$= 2.96 \times \frac{1.67^2}{64.4} = 0.13 \text{ ft.}$$

Total loss of head = 3.00 + 0.13 = 3.13 ft.

Total pumping head

Static head	= 22.38 ft.
Frictional losses	= 3.13 ft.
Total	= 25.51 or 26 ft.

Curve No: V 4303 of Peerless pumps give following details:-

Q = 800 gpm at 26 ft. under 67% efficiency;

RPM = 970; Impeller diameter = 11"; No: V 1974 and V 1975

Size 5 x 6 x 3 - 1/8L 6 x 6 x 3 - 1/8L; HP=10; Sphere 3-1/8"

This is a suitable pump.

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

$$NS = \frac{N\sqrt{Q}}{H^{3/4}} \qquad \frac{970\sqrt{800}}{26^{3/4}} = 2380 \text{ RPM}$$

---

20. Hydraulic Institute, Pipe Friction Manual, p. 28.

This pump will be radial flow.

For combination of the two pumps of 800 gpm

A characteristic curve of the 14" diameter pipe line 3220 ft. is plotted as follows:

(1) Q = 800 gpm, loss of head = 3.13 ft.

(2) Q = 1800 gpm, loss of head = 15.34 ft.

(3) Q = 1200 gpm = 2.67 cfs.

$$V = \frac{2.67}{\pi/4 \times \left(\frac{14}{12}\right)^2} = 2.5 \text{ ft/sec.}$$

$$VD'' = 2.5 \times 14 = 35$$

from graph[21] $f = 0.021$

$$hf = 0.021 \times 3230 \times \frac{2.5^2}{64.4}$$

$$= 6.55 \text{ ft.}$$

Loss of head in Bends and values:-

(i)	Bends 4 Nos,	K	=	0.23[22]
(ii)	Gate value 1 Nos,	K	=	0.04
(iii)	Check value 1 Nos.	K	=	2.00

$$hf = (4 \times 0.23 + 1 \times 0.04 + 2.0 \times 1)\ \frac{2.5^2}{64.4}$$

21. Victor L. Streeter, Fluid Mechanics, p. 220.

22. Hydraulic Institute, Pipe Friction Manual, p. 28.

$= 0.29$ ft.

Total loss of head $= 6.55 + 0.29$ ft.

$= 6.84$ ft.

(4) $Q = 1000$ gpm $= 2.22$ cfs.

$$V = \frac{2.22}{\pi/4 \times (14/12)^2} = 1.98 \text{ ft/sec.}$$

$VD'' = 27.8$

from graph $f = .022$

$$hf = 0.022 \times 3220 \times \frac{1.98^2}{64.4} = 4.31 \text{ ft.}$$

Loss of head in bends and value.

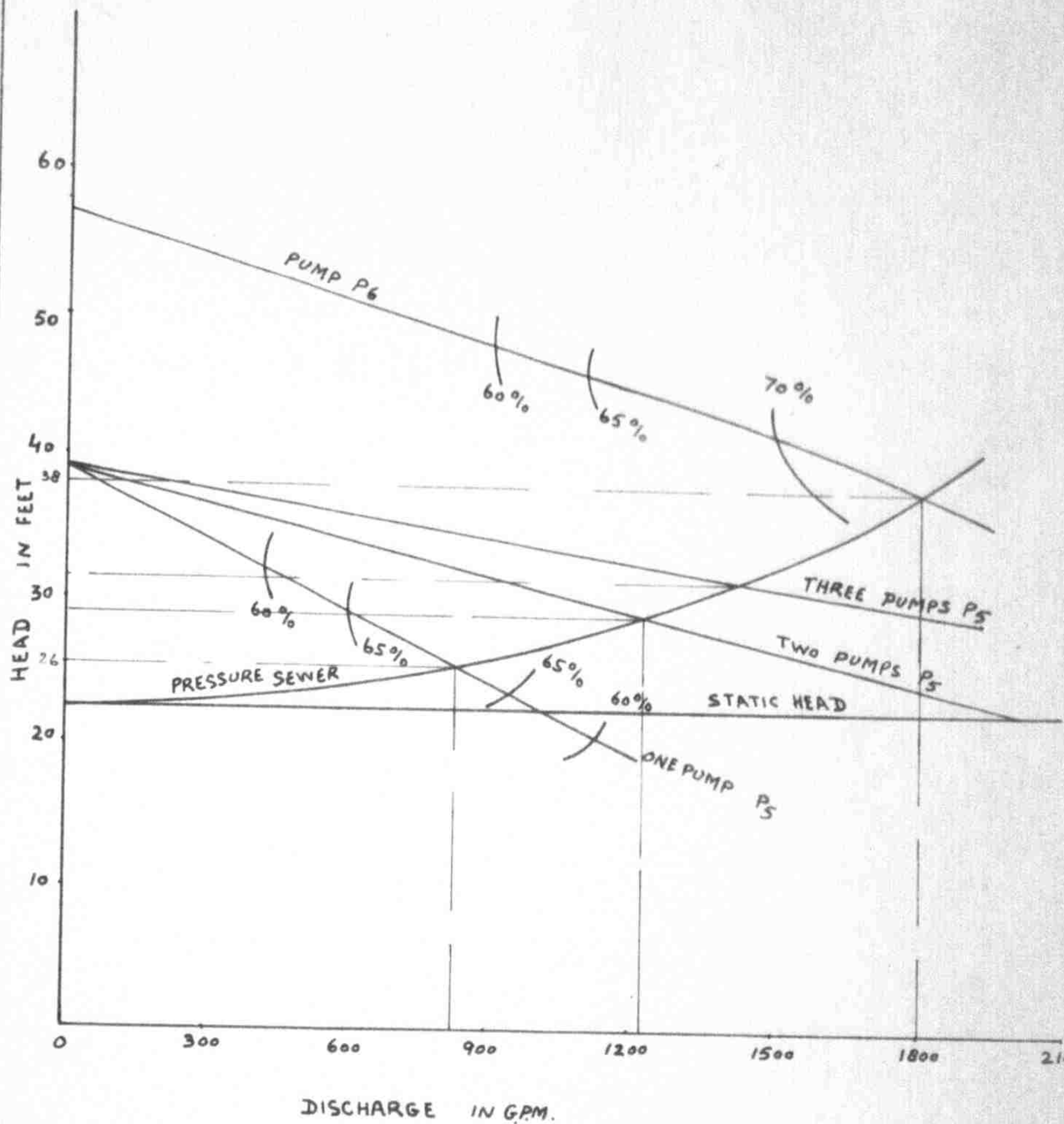
as per above $hf = (4 \times 0.23 + 1 \times 0.04 + 2.0 \times 1) \times \frac{1.98^2}{64.4}$

$= 0.18$ ft.

Total loss of head $= 4.31 + 0.18 = 4.49$ ft.

The discharge head curve of the pump and combination has been plotted over the curve of pipe line. It shows that the combined pump will deliver 1250 gpm under a head of 29.5 ft.

The efficiency of the combined pump will be that of a single pump working for 625 gpm under a head of 29.5 ft at 65%. This performance is satisfactory and is a suitable combination for adopting.



CHARACTERSTIC CURVE OF PUMPS AND

CAPACITY HEAD CURVE OF PRESSURE SEWER

AT PUMPING STATION III

The most economical design of pumping system as arrived at in the preceding discussions involves the use of three pumping stations right from the start of scheme. The newly proposed area for development in the south doesnot have any population at present and as such if the pumping station III is constructed, it will run under loaded for quite some time. There are three alternatives for consideration :-

(1) The site of lagoon is shifted from where it is proposed to a place nearer to the PSII or PSI so as to save a long length of pressure sewer and intermediate station PS III. This is not feasible as the area on the north and west is planted with palm trees and the land is costlier. On the east side it is sea. The present location is only available and suitable site.

(2) Provide a full length of pressure sewer from PS II to lagoon site initially and later when the population increases and the pumping station III is constructed, the designed system is adopted. This will involve into two difficulties.

(i) It will necessitate the laying of extra sewer from manhole S_1H_2 to PS III a distance of 1800 ft. In the present design, a main sewer carries this discharge. As such after sometime when PS III is constructed, this pressure sewer will be practically useless.

(ii) The long length of rising main will require greater capacity pumps which will need replacement when PSIII is constructed as those greater capacity pumps will work under very low efficiency under low heads.

(3) Construct only the pumping station III and main sewer from manhole S_1H_2 to PS III and keep other pumping stations same as per design. The sewers network for PS III will not be constructed at present. The sewers may be installed as the necessity arises with the increase in population. No changes in the pump sizes or pipes will be required. It will be seen from the pump selection that the peak flow from PSII, that is, 1100 gpm. will be fully taken up by the combination of small pumps of PS III which is 1250 gpm.

This third proposal appears reasonable. This will not involve double expenditure on the construction of pumping station III and the main sewer.The excess expenditure will be a very small portion of the total cost of the scheme and as such this may be adopted. A separate break up of cost of full scheme will be prepared which will indicate the approximate financial figures involved.

CHAPTER VIII

DESIGN OF TREATMENT WORKS

The design of oxidation lagoon varies in quite a wide range depending upon the climatic condition. For tropital climates Mr G. Van R Marais has developed a rational theory for the design of lagoon. This theory is based upon the mono-molecular theory which states that the rate of reaction at any time is proportional to the concentration. The following equation has been arrived at by Mr Marais.

$$S = \frac{So}{KR_1 + \frac{R_1}{R_2}}$$

Where S = BOD of effluent, So = BOD of influent,

K = constant, $R_1 = \frac{V}{Q_1}$

$R = \frac{V}{Q_2}$

Q_1 = Influent into the pond, Q_2 = Effluent from the pond,

V = Volume of lagoon.

Mr Marais made a large number of experiments on the lagoons under various conditions of loadings and developed an empirical formulae for the BOD of the effluent which is as under:

$$S = \frac{750}{0.6d + 8}$$

Where d = depth of lagoon.

Maraise method is a combination of theoretical and practical considerations. It is simple and practical. Experiments and actual lagoons designed on this basis have been found to be successful.

The design of lagoons was also investigated by Messrs Grotaas HB and W.J. Oswald. They have also developed certain equations. Their theory is based on the assumption that in the lagoon, all the oxygen required by the bacteria comes from the development of new photosynthesis. Their approach is theoretical and involves so many interrelated factors and assumptions that it is difficult to apply.

The design has therefore been made on Maraise theory and equations.

Design of Lagoons

Population as per forecast estimate = 18,760 persons.

It is assumed that each individual will produce a BOD of 0.17 lbs per day.

Total BOD = 18760 x 0.17

= 3206 lbs. per day.

Amount of sewage expected per day at 70 g.p.c.p.d. including subsurface infilteration = 18,760 x 70 =

= 1,313,300 g/day.

= 2.04 c.f.s.

$$BOD = \frac{3206 \times 1,000,000}{1,313,200 \times 8.34}$$

$$= 291 \text{ p.p.m.}$$

Adopt a depth of lagoon as 4 ft.

$$S = \frac{750}{0.6d + 8} = \frac{750}{0.6 \times 4 + 8}$$

$$= 72 \text{ p.p.m.}$$

The value of K has been given by Marais as 0.17 for tropical areas. As this area has similar characteristic of climate, the same value of K is adopted here.

No record as to the extent of evaporation is available. The humidity is generally high due to sea shore. In the summer months specially July, August and September humidity is very high sometimes reaching even upto 95 to 100 percent. The rainfall being very small, it is expected that there will be no effect on rainfall or evaporation on the working of lagoon. In that case R_1 will be equal to R_2 and the equation will become

$$S = \frac{So}{0.17R + 1}$$

Applying this equation we get

$$72 = \frac{291}{0.17R + 1}$$

$$0.17R + 1 = 4.04$$

R = 17.9 days or approximately 20 days.

Volume of Lagoon = $\frac{1,313,200 \times 20}{7.5}$ = 3,520,000 cft.

Surface area of lagoon = $\frac{3,520,000}{4}$

= 880,000 sft.

= 20.2 Acres.

Provide two lagoons each size 800' x 550' x 4'

BOD loading on lagoon = $\frac{3206}{20.2}$ = 158.5 lbs/acre/day.

According to Maraise, loading upto 174 lbs/acre/day is allowable for four feet deep lagoon with an initial BOD of 291 p.p.m.

So the design is OK.

It will be seen from the design that the expected BOD of the effluent from the primary lagoon will be 72 p.p.m. But Mr Marais, the author of the design theory, recommends that secondary and tertiary lagoons be constructed for the efficient performance of the system and further reduction of BOD. According to him a detention period of seven days should be provided for each. In this particular place, the ultimate disposal is at a point quarter a mile distance from the city, for this a BOD of 72 p.p.m. of effluent is not high and is not likely to cause any harm. Therefore no secondary or tertiary lagoon is essential. However, secondary lagoon will be constructed having a detention ~~of~~ period of seven days as an extra measure of safety. Like the primary lagoons these secondary lagoons will also be constructed in two ponds for flexibility of operation such that if need arises the secondary lagoons can be used in in parallel or series or even act as a tertiary lagoon.

The volume of the secondary lagoon will be

$$= \frac{1,313,200 \times 7}{7.5} = 1,234,000 \text{ c.f.t.}$$

Assume depth of lagoon as 4 ft.

$$\text{Area of lagoon} = \frac{1,234,000}{4} = 308,500 \text{ s.f.t.}$$

Provide two lagoons 55' x 280'

The population served initially will be much less than the designed population. The initial population will be about 50 percent of the designed population and only one unit will be sufficient. Since the cost of land and labour at present is cheap, the cost of lagoon will be small. It is, therefore, recommended that all the lagoons may be constructed initially. This will give flexibility of operation, and better control over the quality of effluent will be possible. These will be arranged in such a way that they can be worked in series as well as in parallel.

A distribution box will control the flow of sewage from the pressure sewer into the lagoons. The piping system has been shown in the drawing.

Inlet into the Lagoons

The inlet will be located at a distance of one third of length from the distribution box side to afford uniform spreading of sewage in the lagoon. A concrete pavement will be provided to avoid errosion of bed.

Outlet From the Lagoon.

The outlet from the lagoon will be arranged in such a way that the level of take off can be adjusted to the desired height. The outlet will be through a manually controlled man-hole and adjustable piping pieces which will discharge the effluent into the effluent channel as shown in the drawing at the required level.

Embankment

The embankment will be made up of the local sandy and silty soil. The slope of the embankment will be made 2:1 or two feet horizontal and one feet vertical.

The bed of the lagoon and the inner side of the embank-ment will be made impervious by a 6 inch layer of marl, a locally available material which when compacted acts like an impervious clay. This marl be layed in two layers of 3 inches each properly rolled and consolidated each time.

The width of the top of the embankment will be kept ten feet to allow a small car to pass over the embankment. A 6 inch layer of marl will also be laid on the top of embankment to provide a hard surface for motoring.

The slope of embankment on the outer side will be provided with vegetation to stop erosion of embankment.

A free board of 2.0 ft will be provided in the lagoon for emergency. This will enable the lagoon to be worked for a greater depth if necessary arises.

Tree Plantation

The city lies on the edge of the desert and plenty of sand is blown in summer. Trees will be planted on all the sides to lower the wind velocity and hence prevent entrance of sand in the lagoon which might create trouble in the biological action and reduce the effective volume.

Fencing

The lagoon will be fenced with barbed wire all around, in order to prevent any trespass of men and animals. A gate will be provided for entrance which will be kept locked.

Effluent Channel

An open trapezaidal channel will carry the effluent from the lagoon to the sea. This channel will be made by excavation in the ground and a concrete lining provided inside.

Design of Effluent Channel

Length of channel	1900 ft
Discharge of Effluent	2.04 c.f.s.

(Neglect all losses and assume the effluent quantity same as influent)

Available slop:

Elevation at lagoon	4.78
Discharge level at sea	1,00
Difference in elevation	3.78

Slope available = $\frac{4.78}{1900}$ = .0025

Assume a self cleansing velocity of 2 ft per second so that no sediment if any may be collected in the bed reducing the effective cross section.

Mannings formulae for velocity of flow and value of n = .012 has been adopted for calculation of cross section. A trapezaidal section is adopted for the section with side slopes 1:1 as it is more economical in construction.

$$A = by + y^2$$

For most economical section of trapezaidal channel

$$b = \frac{P}{3}$$

$$\text{or } b = \sqrt{2y^2} = y\sqrt{2}$$

$$A = y \cdot \sqrt{2y^2} + y^2 = y^2 . \sqrt{2+y^2} = y^2 \ (\sqrt{2} + 1)$$

$$= 2.414\ y^2$$

$$R = \frac{A}{3b} = \frac{2.414\ y^2}{3 \cdot \sqrt{2} \cdot y}$$

$$R = \frac{2.414y}{3\sqrt{2}} = 0.57y$$

$$v = \frac{1.49}{n} \times R^{2/3} \times S^{1/2}$$

$$2 = \frac{1.49}{} \ (0.57y)^{2/3} \times \ (.0025)^{1/2}$$

$$= 4.26y^{2/3}$$

$y = 0.32$ provide y = 8" and free board 6"

$$b = 6\sqrt{2}$$

$= 8.5"$ provide a bed width of 12"

CHAPTER IX

SUPPLEMENTARY WORKS

The supplementary works will include those structures w hich although are not directly involved in treatment or handling of sewage, are helpful in efficient maintenance of the system. These following works will be classified under supplementary works and will be provided. Construction will be done along with the other works :-

A. Residence for pump operator.

B. Transformer room.

C. Workshop.

D. Laboratory.

E. Compound wall and gate.

A. Residence for Pump Operator

The sewage system of the city of Saihat provides for three pumping stations. Although the pumping stations are small ones, and automatic pump controls are provided, still it is utmost necessary to keep a full time operator to look after the performance of the pumps at all time. A residence for an operator is therefore essential. A small but reasonable residence as shown in the drawing No. 13 is proposed to be constructed at each pumping station. The building will be constructed of R.C.C. framework with cement concrete hollow bricks partition. This will provide protection in the hot weather by air insulation.

As the treatment is recommended by lagoons, which does not require such maintenance, no residence is recommended for the treatment side. A part-time operator will be enough to take care of the maintenance of the lagoons, He will visit the lagoon twice or three times a week and make necessary adjustment as required under specific conditions.

B. Transformer Room

Electric motors require constant voltage. The voltage in the domestic power lines usually fluctuates according to the load on the system. This fluctuation is likely to cause damage to the electric motor. In order to protect the electric motor, it is usually recommended to that a separate transformer be provided at every pumping station. In this particular case, the pumping station I will be very small, and no separate transformer is required. For pumping station II and III a separate transfer is recommended at each of the stations. High tension line will be provided from the nearest available source to the pumping stations and will be converted to workable voltage by the transformer. The transformer will be accommodated in a separate room. A smallroom of 12 feet by 12 feet will be enough for the transformer. Position of the transformer room has been marked on the site plan of pump house.

C. Workshop

The sewerage system provides installation and maintenance of several pumps. This will result in frequent repair due to wear. A small workshop is therefore essential for the proper maintenance of the system. A small workshop of size 12' x 16' is therefore proposed at the pumping station III. The position is marked on the plan.

D. Laboratory

A small laboratory is usually essential for a treatment plant in order to exercise proper control of the system. For the city of Saihat the provision of a laboratory is not very essential due to following reasons :=

(1) The method of treatment by lagoon, do not require much of the maintenance.

(2) The city is small and consequently the control is not a difficult operation.

(3) The ultimate disposal of effluent is in sea. Therefore, a slight decrease in quality of effluent temporarily will not result in any hazards.

However, in order to exercise proper control and safeguard against possible contamination, a small laboratory is provided at the treatment works. The laboratory will consist of one room 12 feet by 14 feet. This will be enough for carrying out simple tests at the site of work. Position of the laboratory is marked on the lagoon plan.

E. Compound Wall and Gate

It is essential to prevent tresspass of men and animals at the pumping station and treatment works.

A barbed wire fencing with iron gate will be provided at the treatment works, while a masonary compound wall will be provided at the pumping station.

CHAPTER X

COST ESTIMATE

It has already been stated in chapter I under subhead 'Scope of Work' that the preparation of detailed specifications and cost estimates will be beyond the scope of the present work. It is therefore proposed here to prepare approximate quantities of work done and cost involved in order to give an approximate idea of the cost of the project for allocation of funds and inviting of tenders.

The actual rates for various items of work can only be determined accurately by competitive tenders or inviting rates from different dealers. For the purpose of this work approximate rates have been adopted for different items of work. Approximate quantities of various items will be worked out and the cost estimates will be based over them.

The brief specifications, estimate of quantities and estimate of cost will be as under.:

SPECIFICATIONS

A. SEWERS NETWORK

1. (a) <u>8 inches vitrified clay pipe</u> at an invert depth upto 5 feet as shown on the drawings. The pipe material should confirm

in dimension, and strength to the standard specifications. This item shall include all excavation, back filling, furnishing and installing in place 8" inside diameter pipe as specified, sheeting, dewatering, shoring and a layer of fine grannular fill underneath the pipe well compacted in place. The depth will be computed as average between manholes. The length of sewer will be measured horizontally with no deduction for manholes. The unit price per linear foot will include all labour material and equipment necessary to furnish and install the sewer as shown on the drawings and as specified complete job.

(b) <u>8 inch vitrified clay pipe</u> at an invert depth exceeding 5 feet. This item will be same as item No. 1(a) except for a greater depth.

2. <u>10 inches vitrified clay pipe</u> as shown on the drawings. The pipe material should confirm in dimension and strength to the standard specifications. This item shall include all excavations, back filling, furnishing and installing in place 10" internal diameter pipe as specified, sheeting, shoring, dewatering, disposal of water, layer of fine gravel underneath the pipe well compacted in place. The depth will be computed as an average between manhole. The length of sewer will be measured horizontally with no deduction for manholes. The unit price per linear foot will include all labour, material and equipment

necessary to furnish and install the sewer as shown on the drawings and as specified complete job.

3. 12 inches vitrified clay pipes as shown on the drawings. This will be same as for item No. 2.

4. 15 inches vitrified clay pipes as shown on the drawings. This will be same as for item No. 2.

5. 18 inches vitrified clay pipes as shown on the drawings. This will be same as for item No. 2.

6. Manholes at depth upto 5 feet as shown in the drawings, and constructed of plain concrete or precast concrete rings.

The item includes the following :-

(i) The concrete shall be of approved aggregates and cement in approved proportions with satisfactory curing arrangements.

(ii) The manhole covers will be of cast iron from approved manufacturer, and of standard dimensions.

(iii) The inner and outsides of wall shall be smooth plastered and inner side provided with cast iron steps. The unit price shall include construction of manhole, excavation and back fill. Labour material and equipment required for construction, complete job.

7. Manholes at depths greater than 5 for main sewers as shown in the drawings. This will be same as per item No. 6 except

for greater depth.

8. Manholes at a depth greater than 5 for trunk sewers. This will be same as per item No. 6 except for greater diameter and depth.

B. PUMPING STATION

1. Construction of wet well, dry well and control room as per drawing. The item shall include the following :-

(i) Dewatering by well points, sheet piling, and excavation.

(ii) Making necessary form work and steel reinforcement as per drawings.

(iii) Pouring of cement concrete of the required proportion and to correct size as per drawings.

(iv) Installation of suction pipes five numbers and pipe for float control of required sizes for each pumping station.

(v) Providing all necessary installation of ladders, stairs, screens, grit channel, flumes, emergency overflow.

(vi) Providing necessary openings for installation of pump sets, doors, corridors.

(vii) Washout pipe for pressure sewer.

(viii) Providing electric connection and lighting arrangement in control room, dry well and control room. The unit price will be for complete job as per drawings and as specified.

2. Installation of Pump Sets

This item will include the following :-

(i) Supply of pump as per specifications and approved characteristics.

(ii) The pump shall be vertical spindle, nonclog type.

(iii) The pump shall be fitted in the dry well for positive suction pressure.

(iv) The pump will be coupled with a vertical shaft electric water. The motor will be installed in the control room, and coupled by means of a shaft supported at an intermediate point.

(v) Automatic float control devices, furnished and installed at site, with alternate arrangements of manual control.

(vi) Gate value at suction and delivery, a check value at delivery, reducer of a required size and the required number of bends and supports and other necessary fittings. The unit price will be for the complete job installed and tested at site.

3. Diesel Engine Set. This item shall include furnishing and installation at site of a diesel engine set of required horse power and RPM. complete with coupling arrangement with the pump.

The unit price will be for the complete job installed and tested at site.

4. 6 inches Pressure Sewer. The item shall include furnishing and installing of pressure sewer of 6 inches internal diameter. The pipe material will be centrifugally spun cast iron of class 250 from approved manufacturer, having a working pressure of 250 psi. and should confirm in dimensions and strength to standard specifications. The pipe will be installed under the ground at an average depth of about 2 1/2 feet from the road surface, from the pumping station I to the manhole M_1H_5 installed with necessary bend pieces.

The unit price will be for the complete job installed at site and tested.

5. 10 inches pressure Sewer. This will be the same as per item No. 4 from pumping station No. II to manhole S_1H_2.

6. 14 inches pressure sewer. This will be the same as per item No. 4 from pumping station III to lagoons.

7. Dall Tube. This item will include the furnishing and installation of a Dall tube at each pumping station. The Dall tube will be recording type and of the standard make and specification. This will be installed outside the pumping station in a concrete chamber provided with an opening at the top.

The unit prices will be for the completed job installed at site and tested.

8. Providing moveable crane at pumping station III. This will include furnishing and installation at the pumping station III a moveable crane of 5 ton capacity mounted on a girder complete job installed and tested.

9. Construction of Operators Residence. This will include construction of residence for the operator at each pumping station as per drawings. The material used shall be of approved quality and specification.

The unit price shall include the following :-

(i) Building construction as per drawings and as specified by site engineer.

(ii) Installation of water supply sanitary and electrical fittings. The payment will be made for complete job.

10. Construction of transformer Room. This item will include construction of a transfermer room as per drawing at pumping stations II and III.

The material used shall be of approved quality and specification. Payment will be made for complete job.

11. Construction of Workshop. This will include the construction of a workshop as per drawing at the pumping station III. The material used shall be of approved quality and specifications. The payment will be made for complete job.

12. Compound Wall. This will include the construction of a compound wall at each of the pumping station as per drawing.

The material used shall be of approved quality and specification. Payment will be made for complete job with fine finish.

13. Gate. One steel gate of square steel bars fabrication will be furnished and installed at each pumping station.

C. OXIDATION LAGOONS

1. Embankment of Soil. The embankment will be made up of locally available silty and sandy soil. The soil will be laid in layers of 6 inches thick and well consolidated before laying another layer. The embankment shall be made to correct dimensions and lagoon size as per drawing.

2. Marl Layer. This will include laying of 6 inches thick marl layer on the lagoon area, inner embankment slope a and top of embankment. The marl will be spread in two layer well compacted each time to a total compacted thickness of 6 inches. The unit price will be for completed job.

3. Distribution Box. This will include construction of distribution box as per drawings including steel gate and operation mechanism. The material shall be of approved quality and specification. The unit price will be for completed job, excluding the cost of pipe.

4. Equilizing Manhole. This will involve the construction of equilizing manhole as per drawings. This unit price will

include all concrete work steel gate, operation mechanism and the cast iron pipe and concrete base at the pipe ends complete job.

5. Outlet Manhole. This will involve the construction of outlet manhole as per drawing. This will include the concrete work, steel gate, operation mechanism, cast iron pipe and concrete base at the cast iron pipe ends. The unit price will be for the complete job.

6. Pipe for inlet of Sewage. This will include 18 inches diameter vitrified clay pipe from the distribution box to the inlet piece. The unit price will include the cost of carriage, labor, material, inlet piece and all equipments, concrete base, complete job.

7. Construction of Outlet Channel: This item shall include the construction of effluent channel as per drawing and as specified. The concrete should be made up of approved aggregates, cement and proportions. Sufficient curing will be done for concrete. The embankment on either side shall be compacted. The unit price shall be for complete job finely furnished including the cost of under passage with the coastal road.

8. Construction of Laboratory. This item shall include the construction of a laboratory as per drawings at the lagoon site. The material used shall be of approved quality and

specification. The unit price shall include reasonable testing equipment and furniture as specified. The payment will be made for complete job.

9. Fencing. This will include wire fencing around lagoons as per details shown on the drawings. The unit price shall include furnishing and installation of barbed wire fence supported on steel angle of size 2" x 2" x 3"/16 on concrete base. The unit price will be for complete job.

10. Fence Gate. This will include furnishing and installing fence gate as shown in the drawing. The unit price will be for complete item of work.

11. Tree Plantation. This will provide for plantation of local kind of tree around the lagoon as shown in the drawing. The unit price shall be for at least 4 feet grown up trees as a complete job. The trees should not be at a distance greater than 25 feet centres.

ESTIMATE OF QUANTITIES

PUMPING STATION I.

(i) Sewer Network.

1 (a) 8" vitrified clay pipe at an invert depths upto 5' as per statement, page 122. 15855 ft.

(b) 8" vitrified clay pipe at an invert depth exceeding 5 ft. as per statement. 4794 ft.

QUANTITY OF SEWER PIPES

NAME OF SEWER	LENGTH OF VITRIFIED CLAY PIPES 8" UPTO 5' DEPTH FT.	8" BELOW 5' DEPTH FT.	10" FT.	12" FT.	15" FT.	18" FT.	LENGTH OF LATERAL SEWERS 8" FT.	NO. OF MANHOLE UP TO 5' DEPTH FT.	NO. OF MANHOLES BELOW 5' DEPTH FOR MAINS	NO. OF MANHOLES BELOW 5' DEPTH FOR TRUNKS
PS I										
TRUNK T_1	700	2870	-	-	-	-	2500	28	-	4
MAIN M_3	3400	605	-	-	-	-	3400	23	14	-
MAIN M_2	1275	-	-	-	-	-	2090	12	-	-
MAIN M_0	700	1320	-	-	-	-	1790	12	7	-
TOTAL	6075	4795					9780	75	21	4
PS II										
TRUNK T_2	320	-	1210	1290	-	-	3600	19	-	4
MAIN SEWER M_6	1720	-	-	-	-	-	2350	21	-	-
MAIN M_5	1660	-	-	-	-	-	2870	23	-	-
MAIN M_4	2780	-	-	-	-	-	4240	30	-	-
MAIN M_3	2840	-	-	-	-	-	2560	28	-	-
MAIN M_2	810	850	-	-	-	-	4860	27	3	-
MAIN M_1	710	-	-	-	-	-	7950	37	3	-
TOTAL	10840	850	1210	1290			28,430	185	6	4
PS III										
TRUNK T_3	-	730	1080	450	-	150	4720	17	-	10
MAIN M_8	1000	1150	-	-	-	-	1590	11	4	-
MAIN M_7	900	1770	-	-	-	-	3590	19	10	-
MAIN M_6	740	1370	-	-	-	-	3570	18	11	-
MAIN M_5	800	1360	-	-	-	-	3490	19	10	-
MAIN M_4	700	1480	800	-	-	-	4240	19	14	-
MAIN M_3	1160	-	-	-	-	-	1360	8	1	-
MAIN M_1	900	1580	250	-	1100	-	3420	18	15	-
MAIN S_1	600	-	-	720	-	-	980	6	2	-
MAIN S_2	1120	-	-	-	-	-	160	10	2	-
MAIN S_3	1250	-	-	-	-	-	1070	8	2	-
MAIN S_5	800	1060	-	-	-	-	3410	12	12	-
MAIN S_6	800	1390	-	-	-	-	3200	13	7	-
TOTAL.	10770	11890	2130	1170	1100	150	34,800	178	90	10

QUANTITY OF SEWER PIPES

NAME OF SEWER	LENGTH OF VITRIFIED CLAY PIPES 8" UPTO 5' DEPTH FT.	8" BELOW 5' DEPTH FT.	10" FT.	12" FT.	15" FT.	18" FT.	LENGTH OF LATERAL SEWERS 8" FT.	NO. OF MANHOLE UP TO 5' DEPTH FT.	NO. OF MANHOLES BELOW 5' DEPTH FOR MAINS	NO. OF MANHOLES BELOW 5' DEPTH FOR TRUNKS
PS I										
TRUNK T_1	700	2870	-	-	-	-	2500	28	-	4
MAIN M_3	3400	605	-	-	-	-	3400	23	14	-
MAIN M_2	1275	-	-	-	-	-	2090	12	-	-
MAIN M_0	700	1320	-	-	-	-	1790	12	7	-
TOTAL	6075	4795					9780	75	21	4
PS II										
TRUNK T_2	320	-	1210	1290	-	-	3600	19	-	4
MAIN SEWER M_6	1720	-	-	-	-	-	2350	21	-	-
MAIN M_5	1660	-	-	-	-	-	2870	23	-	-
MAIN M_4	2780	-	-	-	-	-	4240	30	-	-
MAIN M_3	2840	-	-	-	-	-	2560	28	-	-
MAIN M_2	810	850	-	-	-	-	4860	27	3	-
MAIN M_1	710	-	-	-	-	-	7950	37	3	-
TOTAL	10840	850	1210	1290			28430	185	6	4
PS III										
TRUNK T_3	-	730	1080	450	-	150	4720	17	-	10
MAIN M_8	1000	1150	-	-	-	-	1590	11	4	-
MAIN M_7	900	1770	-	-	-	-	3590	19	10	-
MAIN M_6	940	1370	-	-	-	-	3570	18	11	-
MAIN M_5	800	1360	-	-	-	-	3490	19	10	-
MAIN M_4	700	1480	800	-	-	-	4240	19	14	-
MAIN M_3	1160	-	-	-	-	-	1360	8	1	-
MAIN M_1	900	1580	250	-	1100	-	3420	18	15	-
MAIN S_1	600	-	-	720	-	-	980	6	2	-
MAIN S_2	1120	-	-	-	-	-	160	10	2	-
MAIN S_3	1250	-	-	-	-	-	1070	8	2	-
MAIN S_5	800	1060	-	-	-	-	3410	12	12	-
MAIN S_6	800	1390	-	-	-	-	3200	13	7	-
TOTAL	10770	11890	2130	1170	1100	150	34800	178	90	10

2.(a) Manholes upto 5 ft. depth as per statement 75

(b) Manholes exceeding 5 ft. depth as per statement for main sewers. 21

(c) Manholes exceeding 5 ft. depth as per statement for trunk sewer. 4

(ii) Pump Station

3. Construction of wet well, dry well, and control room as specified. 1

4. Installation of pumping sets as specified

(i) Pumps P_1 3

(ii) Pumps P_2 2

(iii) Diesel Engine large 1

(iv) Diesel Engine small 1

5. Dall tube as specified 1

6. Construction of operator's residence as specified 36' x 31' built up area. 1116 sft.

7. Construction of compound wall as specified 592 ft.

8. Installation of compound wall gate as specified 1

(iii) Pressure Sewer.

9. 6 inches diameter pressure sewer as specified 1920 ft.

B. PUMPING STATION II

(i) Sewer Network

1 (a) 8" vitrified clay pipe as per statement at an invert depth upto 5 ft. 39270 ft.

(b)	8" vitrified clay pipe as per statement at an invert depth exceeding 5 ft.	850 ft.
2.	10" vitrified clay pipe as per statement	2830 ft.
3.	12" vitrified clay pipe as per statement	1290 ft.
4 (b)	Manholes exceeding 5 ft. depth as specified for main sewers	13
(a)	Manholes upto a depth of 5 ft. as specified	185
(c)	Manholes exceeding 5 ft. depth for trunk sewer	4

(ii) Pump Station

5.	Construction of wet well, dry well and control room as specified.	1
6.	Installation of pumping sets as specified	
	(i) Pump P_3	3
	(ii) Pump P_4	2
	(iii) Diesel Engine	1
7.	Dall tube as specified	1
8.	Construction of operator's residence as specified 36' x 31' = 1116 sq.ft.	
9.	Construction of transformer's room as specified 13' x 13'	169 sq.ft.
10.	Construction of compound wall as specified	592 ft.
11.	Installation of compound wall gate as specified	1

(iii) Pressure Sewer.

12.	10 inches diameter cast iron pressure sewer as specified.	1710 ft.

C. PUMPING STATION III.

(i) Sewer Network.

1.(a)	8" vitrified clay pipe as per statement a at an invert upto 5 ft. depth	45,510 ft.
(b)	8" vitrified clay pipe at an invert depth exceeding 5 ft. as per statement.	11,890 ft.
2.	10" vitrified clay pipe as specified	2,13o ft.
3.	12" vitrified clay pipe as specified	1,170 ft.
4.	15" vitrified clay pipe as specified	1,100 ft.
5.	18" vitrified clay pipe as specified	150 ft.
6.	Manholes upto a depth of 5 ft as specified for mains and laterals	178 ft
7.	Manholes to a depth exceeding 5 ft. as specified for main sewers	80
8.	Manholes to a depth exceeding 5 ft. as specified for trunk sewer	10

(ii) Pump Station

9.	Construction of wet well, dry well, and control room as specified		1
10.	Installation of pumping set as specified		
	(i) Pump P_5	3	
	(ii) Pump P_6	2	

(iii) Diesel Engine large 1

(iv) Diesel Engine Small 1

11. Dall tube as specified 1

12. Providing a moving crane of 5 ton capacity installed in the station and tested. 1

13. Construction of operators residence as specified 36' x 31' = 1116 sqft.

14. Construction of transfermer room as specified 13' x 13' 169 sqft.

15. Construction of workshop as specified 16' x 12' = 192 sq.ft.

16. Construction of compound wall as specified 632 ft.

17. Construction of compound wall gate as specified 1

18. 14 inches diameter cast iron pressure sewer as specified 3220 ft.

D. OXIDATION LAGOON.

1. Embankment of soil as specified

$$(3 \times 1185 + 3 \times 1170)\ \frac{(10+34)}{2} \times 6.0 = 931{,}000 \text{ cft.}$$

2. Marl layer as specified.

$$2 \times 192 \times 542 + 2 \times 272 \times 542 + [4(792+14) + 4\ (272+14) + 6\ (542 + 14)] \times 14 + 3 \times 10\ [792 + 272 + 35 + 2 \times 17] + 3 \times 10\ [2 \times 542 + 35 + 2 \times 17] = 1{,}311{,}900 \text{ s.ft.}$$

3.	Distribution box as specified	1
4.	Inlet pipe of 18" diameter vitrified clay as specified for both primary lagoons	2 x 400 = 800 ft.
5.	Equalizing manhole as specified	6
6.	Outlet manhole as specified	4
7.	Construction of outlet channle	1900 ft.
8.	Construction of laboratory as specified	192 ft.
9.	Fencing as specified 2 (1270 + 1340) = 5220 ft.	
10.	Fence gate as specified	1
11.	Tree plantation as specified complete job.	

ABSTRACT OF COST

I.	Sewer network and PSI	=	SR 346,790
II.	Sewer network and PSII	=	SR 597,290
III.	Sewer network and PSIII	=	SR 862,480
IV.	Oxidation lagoon	=	SR 286,832
	Total Saudi Rials	=	2,093,392

Or US $ 464,500.00.

ESTIMATE OF COST

Item No.	Description	Estimated Quantities	Unit	Unit Price S.R.	Total Price S.R.
	PUMPING STATION I				
	(i) Sewer network				
1.a	8" vitrified C.P. at an invert depth up to 5 ft.	15855	ft.	8	126,840
1.b	8" V.C. pipe at an invert depth exceeding 5 feet.	4795	ft.	10	47,950
2.a	Manholes up to 5 ft. depth	75	each	300	22,500
2.b	Manholes exceeding 5ft depth for main sewer	21	each	400	8,400
2.c	Manhole exceeding 5ft depth for trunk sewer	4	each	500	2,000
				Total	207,690
	(ii) Pump station.				
3.	Construction of wetwell drywell and control room complete job.	1	each	20,000	20,000
4.	Installation of pump set as specified				
	(a) pump P_1	3	each	8000	24,000
	(b) pump P_2	2	each	10000	20,000
	(c) Diesel Engine large	1	each	5000	5,000
	(d) Diesel Engine small	1	each	4000	4,000
5.	Dall tube	1	each	500	500
6.	Operators residence	1116	sft	30	33,480
7.	Compound wall	592	ft	15	8,880
8.	Compound wall gate	1	each	--	200
				Total	116,060
	(iii) Pressure sewer				
9.	6 inches pressure sewer	1920	ft.	12	23,040
				Total for PSI.	346,790

Item No.	Description	Estimated Quantities	Unit	Unit Price	Total Price S.R.
	PUMPING STATION II				
	(i) Sewer network.				
1.a	8" vitrified clay pipe as per statement invert depth up to 5ft.	39,270	ft.	8	314,160
b	8" V.C. pipe as per statement invert depth exceeding 5ft.	850	ft.	10	8,500
2	10" V.C. pipe as per statement	2830	ft.	14	39,620
3	12" V.C. pipe as per statement	1290	ft.	18	23,220
4 a	Manholes up to a depth of 5 ft.	185	each	300	55,500
b	Manholes exceeding 5ft. depth for main sewer.	13	each	400	5,200
c	Manholes exceeding 5ft. depth for trunk sewer.	4	each	500	2,000
				Total	448,200
	(ii) Pump station.				
5	Construction of wet well and control room	1	each	20,000	20,000
6	Installation of pump sets as specified				
	(a) Pump P_3	3	each	9,000	27,000
	(b) Pump P_4	2	each	12,000	24,000
	(c) Diesel Engine large	1	each	6,000	6,000
7	Dall tube	1	each	500	500
8	Operators residence	1116	sft.	30	33,480
9	Transformer room	169	sft.	20	3,380
10	Compound wall	592	ft.	15	8,880
11	Compound wall gate	1	each	--	200
				Total	123,440

Item No.	Description	Estimated Quantities	Unit	Unit Price	Total Price
	(iii) Pressure Sewer				
12	10 inches cast iron pressure sewer	1710	ft.	15	25,650
			Total for PSII		597,290
	PUMPING STATION III				
	(i) Sewer network				
1.a	8" vitrified clay pipe as per statement at an invert depth up to 5ft, as specified	45570	ft.	8	364,560
b	8" V.C. pipe at an invert depth exceeding 5ft. as specified	11890	ft.	10	118,900
2	10" V.C. pipe as specified.	2130	ft.	14	29,820
3	12" V.C. pipe as specified.	1170	ft.	18	21,060
4	15" V.C. pipe as specified.	1100	ft.	22	24,200
5	18" V.C. pipe as specified.	150	ft.	28	4,200
6	Manholes up to a depth of 5 ft. as specified for mains and laterals.	178	each	300	53,400
7	Manholes exceeding 5ft. depth for main sewers.	80	each	400	32,000
8	Manholes exceeding 5ft. depth for trunk sewer.	10	each	500	5,000
	(ii) Pump Station				
9	Construction of wet well dry well, and control room as specified	1	each	20,000	20,000

Item No.	Description	Estimated Quantities	Unit	Unit Price	Total Price S.R.
10	Installation of pump sets as specified.				20,000
	(a) Pump P_5	3	each	10,000	30,000
	(b) Pump P_6	2	each	15,000	30,000
	(c) Diesel Engine large	1	each	8,000	8,000
	(d) Diesel Engine small	1	each	7,000	7,000
11	Dall tube as specified	1	each	800	800
12	Moving crane under girder	1	each	2,000	2,000
13	Operater residence	1116	sft.	30	33,480
14	Transformer room	169	sft.	20	3,380
15	Work shop	192	sft.	25	4,800
16	Compound wall	632	ft.	15	9,480
17	Compound wall gate	1	each		200
				Total=	149.140
	(iii) Pressure Sewer				
18	14 inches diameter pressure sewer of cast iron as specified.	3220	ft.	20	64,400
			Total for PSIII.		862,480

OXIDATION LAGOON

Item No.	Description	Estimated Quantities	Unit	Unit Price	Total Price S.R.
1	Embankment	931,000	cft.	10/100cft	93,100
2	Marl	1311,900	sft.	8/100 "	104,952
3	Distribution Box	1	each	1000	1,000
4	Vitrified clay pipe 18" diameter	800	ft.	25ft.	20,000
5	Equalizing manhole	6	each	2000	12,000
6	Out let manhole	4	each	1500	6,000
7	Out let channel	1,900	ft.	10	19,000
8	Fence	5,220	ft.	4	20,880
9	Fence gate	1	each	200	200
10	Laboratory	192	sft.	25	4,800
11	Tree plantation	job	job	5000	5,000
			Total for lagoon		286,832

BREAK UP OF COST

I	Sewers network	
	PS I	SR 207,690
	PS II	SR 448,200
	PS III	SR 648,940
	Total	SR 1,304,830
II	Pumping Station	
	PS I	SR 116,060
	PS II	SR 123,440
	PS III	SR 149,140
	Total	SR 388,640
III	Pressure Sewer	
	From PS I	SR 23,040
	From PS II	SR 25,650
	From PS III	SR 64,400
	Total	SR 113,090
IV	Oxidation Lagoon	SR 286,832
	Grand total =	SR 2,093,392

Or US $ 464,500.00

S U M M A R Y

The city of Saihat, Saudi Arabia, poses peculiar problems in the design of sewerage system, namely the growth of population, topography, saline and high subsoil waterlevel. Effort has been made to cope with the situation as economically as possible consistent with efficient performance of the system. The population forecast has been based on the master plan which was prepared under instructions of local and sponsoring authorities. The problem of topography and high water table have been overcome by providing three pumping stations. Econmy in the cost of pressure sewer and pumping has been effected by utilizing two pumping stations as lift stations. Vitrified clay pipe has been adopted for the sewers for reasons of durability and resistant to saline water and corrosive substances present or produced in sewage.

Oxidation lagoons have been designed for the treatment of sewage. The ultimate disposal of the effluent will be in the sea. The lagoons will be located at a distance of quarter mile from the edge of city towards the south.

Essential supplementary works such as operator's residences, transformer room, workshop and laboratory have been provided as shown on the drawings.

Approximate cost estimate has been prepared. The total cost works out to be US $ 464,500.00 or $ 24.80 per head. This cost is quite reasonable.

APPENDIX I

POPULATION FORECASTING METHODS

The following methods are generally used for population forecasting :-

(i) Arithmetic progression.

(ii) Geometric progression.

(iii) Graphical extension of population.

(iv) Curvilinear rate of growth.

(v) Ratio method.

(vi) Logistic curve.

(vii) Decreasing rate of growth.

Each of the above methods is described below in brief as applicable to Saihat.

(i) Arithmetic Progression. The increase during the past few years is assumed as constant and this increase is added to the existing population at every equal interval. This method under ordinary circumstances give a very low figure. But in the case of Saihat, the increase of population during the past few years had been abnormal and with this rate of population increase, the population in fifty years is expected to be as follows :

Increase in five years = 3565

Increase in fifty years = 35650

Population after 50 years = 7565 + 35650 = 43,215.

This figure appears to be too high and such a figure cannot be expected to reach after fifty years under the present and expected future circumstances.

(ii) Geometric Progression. This method implies a uniform percentage rate of growth. When plotted, it produced 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 is not applicable for this city.

(iii) Graphical Extension of Population. A curve is plotted with population as ordinate and time as abscissa. The curve is extended into the future by judgement of its general tendency and knowledge of the characteristics of the city. As we have only two figures of population a curve based on them cannot be expected to lead to an acceptable conclusion without some other information.

(iv) Curvilinear Rate of Growth. In this method a curve is plotted for the population of the city in question to any convenient scale. Similar curves are plotted for cities similar to and of larger population, the zero of the abscissas of each city being taken at the year when its population was the same as that of the city in question. 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 no other records of population are available, this method is also not applicable for this city.

(iv) Ratio Method. This method is based upon the belief that the population of cities or other areas will have a relationship to the population of the whole country. This method requires firstly the computation of the local to national population ratio in two to four census years and secondly the forecast of the national population. As no record of national census is available the method is not applicable.

(v) 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:

$$Y = \frac{k}{1+me^{a_1 x}}$$

Where Y = Population at any time x.

k = Saturation population.

m, a_1 are constant.

The constants are determined by selecting three points uniformly spaced along x axis on the curve of past population record.

This method[1] cannot be applied in this case as the record is available for two years only. This record is not enough to indicata a representative past increase.

(vii) Decrease in Rate of Growth. As a general rule it is found that as the population of city increases, the rate of growth decreases. Result 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 data, it cannot be applied here.

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[1] J.E. McLean, "More Accurate Population Estimates By Means of Logistic Curves," Civil Engineering, Vo. 32 (February 1952), p. 35.

APPENDIX II

PIPE MATERIALS

The selection of pipe material depends on the following factors:-

(1) Characteristic of sewage.

(2) Life expectancy.

(3) Resistance to Scour.

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

(5) Ease of handling and installation.

(6) Strength to resist structural failures

(7) Type of joint, watertightness and ease of assembly

(8) Availability in sizes required.

(9) Cost of material, handling and installation.

(10) Site condition.

(11) Local material.

It will be seen from the above that no one material will meet all the requirements or conditions encountered in sewer design. As the designed diameters of the sewer pipes come out to be very small the cast-in-place sewer pipes will be uneconomical and will not be considered in further discussion. No locally manufactured pipe is available.

Available Pipe Material

The various types of pipes available and normally used in sewers are :

(1) Cast iron pipe.

(2) Cement concrete pipe.

(3) Asbestos cement pipe.

(4) Vitrified clay pipe.

These types of pipes are briefly discussed below in order to determine their suitability for the system under design.

(1) Cast Iron Pipes. The advantages of these pipes are long laying lengths, water tight joints, ability to withstand high lateral pressures and external loads.

These pipes are not generally used for sewers for reasons of high cost and corrosion. Cast iron pipes are corrosion resistant in most natural soils, but are subject to corrosion by acids, septic sewage and acid soils. These pipes are used in sewerage only under special conditions like sea outfalls and pressure sewer with suitable protection.

(2) Cement Concrete Pipes. Cement concrete pipes have been used and are being used very extensively as pipe sewers. The advantages of these pipes are :

(i) Wide range of pipe sizes and long laying lengths.

(ii) Relative ease with which the required strength may be provided.

(iii) Relative ease and small equipment required for manufacture.

The disadvantage of concrete pipes is that it is subject to corrosion. The biochemical reaction that takes place in the sewers is as follows:

The biological reaction starts in the sewage while flowing through the sewer. In the absence of ventilation, anaerobic bacteria become effective producing Hydrogen Sulphide. This hydrogen sulphide is converted to sulfuric acid in presence of moisture by sulfure oxidizing bacteria known as 'thiobacillus'.[4] present in sewage. The sulfuric acid being highly corrosive reacts with the lime content of cement forming calcium sulphate and calcium Aluminium sulphate. These compounds being soft and putty like, cause pitting on the inner surface of pipe. This process continues to weaken the cement concrete pipe until it leads to local failure of pipes.

This action of inside corrosion can be much reduced if the production of hydrogen sulphide gas is checked by promoting condition in the sewers unfavourable to the growth of anaerobic bacterial which produce hydrogen sulphide. This can be achieved by two methods :

(1) <u>Ventilation</u>: The sewers are designed for maximum peak flows which occurs for a very short time. The sewers thus

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[4]Ross E McKinney, <u>Microbiology for Sanitary Engineers</u>, (New York: McGraw Hill Book Co., Inc., 1962), p. 171.

run full for a very short time and for a major portion of the time the flow is much less than full. Ventilation can be easily provided by untrappedvents at each house connection. The air will then circulate and will not allow anaerobic bacteria to develop. Ventilation also helps reduction of explosive atmosphere.

(2) Ample Slopes. Sufficient slopes are provided for sewage to flow quickly to the disposal units, and have no time to stagnate or flow slowly thus preventing the development of septic and anaerobic condition in the sewers.

The concrete pipes can thus be protected from damage by sewage. Now in this particular area the water table is very high and most of the sewers will be laid under water. This ground water is contaminated with sea water and contains a high percentage of sodium, calcium and magnesium salts. These salts will react with the lime and alumina present in cement and will cause pitting and finally failure of pipe in a manner similar to acid corrosion due to sewage as stated above. In this case, however, the protection is not easy and if provided will be very costly, making the selection of concrete pipes for pipe sewers as unsuitable for this area with high water table. These pipes can however be used in the old city where the depth of water table is sufficient and there are much less possibilities of outside corrosion.

(3) Asbestos Cement Pipe. These pipes have the following advantages over the concrete pipes :

1. Light weight.
2. Ease of handling,
3. Tight joints,
4. Long length of pipes,
5. Rapidity in laying the pipes.

The chemical properties of these pipes are similar to that of concrete pipes. In this case also the protection coating on the outerside will be uneconomical and these pipes also cannot be used for the areas having shallow depth of water table. These pipes can, however, be used in the old city where the water table is quite low and there are less possibilities of outside corrosion and are preferable to concrete pipes in view of the above advantages. Although the cost of these pipes is more than that of concrete pipe, in this particular region, there being no factory manufacturing concrete pipe, the freight and handling charges will be higher for concrete pipes in comparison to asbestos cement pipes and the final cost is likely to be the same for both cases.

(4) <u>Vitrified Clay Pipes.</u> Vitrified clay pipes (glazed) possess a very high resistance to corrosion and erosion. These are very durable and meet most of the requirements. The smooth and impervious surface offers good hydraulic properties. However, the maximum size of vitrified clay pipe is limited partly by the difficulties in forming and glazing larger pipes and partly

because of weight which make it expensive for shipment and difficult to handle. A disadvantage of vitrified clay pipe is its brittleness that may result in damage during transportation and handling on the job.

In this particular design, as the area involved is very small and the diameters of pipe sewers are also small upto a maximum of eighteen inches, not much difficulty willbe involved.

The cost of these pipes is usually more in comparison with the concrete pipes and asbestos cement pipes, but in view of the heavy cost of protection of concrete or asbestos pipe from the outside, the vitrified clay pipe is likely to be most economical.

Conclusion (Use of Material)

It is, therefore, recommended that for the old city the sewers will be laid of asbestos cement pipes or concrete pipes or vitrified clay pipes. Alternate tenders may be invited and those costing less may be adopted. For new development where the subsoil water level is very high, vitrified clay pipes will be used.

For asbestos cement pipes or concrete pipes, joints with rubber ring is recommended for reasons of greater flexibility and better water tightness while for vitrified pipes bituminous type of joints are proposed.

APPENDIX III

METHODS OF DISPOSAL

Basically there are two methods of disposal of sewage :

I. Disposal of sewage without treatment.

II. Disposal of sewage with treatment.

These methods can be sub-divided as follows :-

I. Disposal of sewage without treatment :

(a) Disposal into water

(b) Disposal on the land.

II. Disposal after treatment by the following methods :

(a) Conventional treatment

(b) Oxidation Lagoons

(c) Oxidation Ditches.

The sewage after treatment is not dangerous and can be disposed of easily without producing any undesirable effect. The above methods are discussed below in order to determine the suitability of the methods of disposal for this place.

I. Disposal of Sewage Without Treatment

(a) Disposal into Water. The sewage is discharged into large bodies of water where transformation of putrescible organic substance into stable organic compounds and finally into stable organic substances takes place by natural agents under aerobic conditions without causing offensive conditions. The dilution factor, the velocity and direction of currents are important factors which should be taken into account, else pollution of water, forming of sludge banks, and unsighty floating matter on the surface of water, are likely to occur.

This method is most economical when local conditions allows such a disposal. Normally such type of disposal is recommended only when a dilution factor of 500:1 normally prescribed for rivers, is obtainable. The nuisance which is likely to be produced due to floating solids may be eliminated by screening the sewage before disposing it off in the sea. However, the following factors have to be considered for disposal of sewage into the sea for this place.

(i) To get this dilution a greater depth of outfall sewer will be required. Usually a depth of twentyfive feet is required for discharge of sewage. This much depth will require a long length of the outfall sewer as the coastal land slopes down very gently towards the sea. The soil being sandy and silty, it is extremely difficult to lay the outfall sewer in such a long length, and the cost will be prohibitive.

(ii) The coast being subjected to high tides which carries sea water far inland, it is likely that sea water in greater concentration of sewage will be carried to the sea coast and may contaminate the swimming beaches and recreation spots.

It will, therefore, be more economical if the sewage is treated by some suitable and economical process and the effluent is discharged into the sea through an open ditch.

(b) Disposal on Land. This method consists in discharge of sewage upon the surface of the ground from which a part of the sewage evaporates while the remainder percolates down to the subsoil water level. Crops can be grown over the land in water shortage areas if the soil condition allow it.

Percolation can be effected by subsurface drainage also. In this method the sewage is discharged into the perforated pipes burried underground. Bad odors can thus be prevented.

The chances of pollution of subsoil water due to this method are quite remote. The sewage as it percolates gets filtered through the soil. The floating and suspended solids are retained in the soil or surface of ground. The sewage comes in contact with soil bacteria and with the help of the soil bacteria and various minerals, the sewage organic solids are converted into non-putrescible stable and harmless compounds. As the process takes long time, the pathogenic bacteria dies out, and therefore the subsoil water is not contaminated. The favourable factors for such a disposal in the city of Saihat are the following :

(i) Available cheap land.

(ii) Small rainfall.

This method, however, requires greater depth of water table. But the subsoil water level in this area is very high

It will, therefore, be more economical if the sewage is treated by some suitable and economical process and the effluent is discharged into the sea through an open ditch.

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(i) Available cheap land.

(ii) Small rainfall.

This method, however, requires greater depth of water table. But the subsoil water level in this area is very high specially in the plain area on the south of the city. The surface soil is sandy and silty. The whole surface soil will then become saturated with sewage shortly. Percolation and oxidation will not be achieved. The sewage will stagnate on the ground and will cause unsanitary conditionaall around. This method therefore cannot be adopted here.

II. Disposal After Treatment

(a) Conventional Method. The conventional method consists of providing primary treatment, secondary treatment and sludge disposal.

(1) Primary Treatment involves the following processes:-

(i) Screening

(ii) Grit removal

(iii) Grease removal

(iv) Sedimentation of sludge.

(2) Secondary Treatment involves the oxidation of sewage effluent from primary sedimentation tanks by either of the following processes:-

(i) Filtration by either intermittent,contact or trickling filters.

(ii) Aeration by activated sludge process or contact aerators.

(iii) Chlorination.

(iv) Oxidation ponds.

(3) Sludge is disposed of by any of the following methods:

(i) Digestion and drying.

(ii) Incineration.

(iii) Vacuum filters.

(iv) Lagooning.

(v) Barging.

(vi) Landfills.

(vii) Centrifuging.

The conventional treatment process involves an elaborate system of structures, and equipment requiring large amount of money for its initial construction and maintenance. The financial resources of the city do not allow the undertaking of such an elaborate and expensive undertaking and other available methods require study.

(b) Oxidation Ditches. The sewage is treated by aerobic oxidation. The oxygen of air is utilized in the process. The screened sewage is made to flow in a small endless channel and is churned at one point by a rotor which cause splashing in sewage. More and more oxygen of air comes in contact with the sewage which is oxidised aerobically.

This method of treatment has the following advantages:-

(1) It is simple and more economical than treatment by conventional method. There is no problem of sludge digestion.

(2) A better quality of effluent is obtained.

(3) Requires a relatively small area.

(4) There are no odor problems or unsighty conditions.

(5) The plant can withstand shock loads without causing troubles.

(6) No specialist or highly skilled operator is necessary.

(7) There are no submerged machinery or equipments. The rotor is partly submerged and therefore the maintenance is simple and economical.

However this method has got the following disadvantages:-

(1) The plant requires constant use of power, mechanical and electrical equipments, thus increasing cost of installation as well as maintenance.

(2) The plant requires constant supervision which increases the cost of maintenance.

(c) Oxidation Lagoons. Oxidation lagoons are ponds which receive raw sewage directly. The lagoons provide large water surface at which oxygen is dissolved from the atmosphere and is utilized by the aerobic bacteria for the satisfaction of BOD. Nitrates and phosphates present in sewage promote the growth of algae. Algae absorbs carbondioxide produced by bacteria and aids the aerobic action by producing oxygen. This oxygen is used by bacteria for reduction of BOD. About 70 percent to 90 percent of the BOD reduction is ordinarily achieved and the final effluent may be discharged at any suitable place without danger of pollution or producing any hazard to life and safety.

The oxidation lagoons are the most suitable methods of treatment of raw sewage in small areas where large areas of land are available at low cost.

The oxidation lagoon has the following advantages in general :

(1) The construction of lagoon is very simple and cheap. No machinery, equipment or heavy structures are involved.

(2) The maintenance of lagoons is very simple and cheap. No wear and tear of machinery or structure is involved.

(3) Several units can be easily and economically added.

(4) Lagoons can be recovered easily.

(5) There is no sludge problem.

(6) Very high bacterial efficiency is attainable.

(7) Long storage kills pathogenic bacteria.

(8) The operation is flexible and the plant can take considerable overloading and work underloaded efficiently.

However, the oxidation lagoons have some disadvantages which are listed below :-

(1) Disagreeable odors may occur under adverse conditions.

(2) The overloading may cause to make lagoon a breeding centre for diseases.

(3) Greater difficulty in effective disinfection for control of disease.

(4) Large and cheap land is required.

(5) Sand blowing in the area may cause some problems.

These disadvantages are not of serious nature and can be overcome as follows for the present city :-

(1) Proper maintenance will not allow disagreeable odors to develop. The lagoons is to be located quite far outside the city at least a quarter mile and as such it is not expected to cause any damage to health and safety even if some septic conditions may develop at any time due to unavoidable reason.

(2) Land is already available very cheap and in plenty and does not constitute any problem.

(3) Sand problem can be eliminated to a considerable extent by tree plantation all around.

The city of Saihat has the following additional advantages which promote adopting of sewage oxidation lagoons as treatment process:

(1) Plenty of sunshine is available all the year except for a very few days of rain storm.

(2) Humidity is very high in summer and hence the evaporation rates are low.

(3) The surface of ground and embankments can be made impervious by a soil known as marl which is found in plenty in the coastal areas.

Conclusion (Treatment of Sewage)

The comparison of the relative advantages and disadvantages of the different methods of treatment of sewage and their suitability for thie area clearly indicates that the most suitable, cheap, and economical method of sewage treatment will be by oxidation lagoons. This will require the minimum finances for construction and maintenance and will give an effluent of low BOD suitable to be discharged into the sea.

This method of treatment had gained wide acceptability and popularity in the region. Sewage oxidation lagoons, installed in Dhahran since about fifteen years are functioning satisfactorily. Very large oxidation lagoons are also under construction in the cities of Khobar and Dammam. It can thus be expected that the oxidation lagoons will work satisfactorily for Saihat also.

DESIGN OF SEWER NETWORK

MANHOLE No	LENGTH FT.	INCT. AREA ACRE	POPLN. DENSITY PERSONS PER ACRE	INCT. POPLN.	DOMESTIC SEWAGE: INCRT. DISCHARGE IN GPD	DOMESTIC SEWAGE: CUMUL. DISCHARGE IN GPD	DOMESTIC SEWAGE: MAX. DISCHARGE IN GPM	INFILTERATION: INCRT. FLOW IN GPM	INFILTERATION: CUMUL. FLOW GPM	TOTAL DISCHARGE GPM	TOTAL DISCHARGE C.F.S.	GROUND ELEVATION: UPPER M.H. FT	GROUND ELEVATION: LOWER M.H. FT	DIAMETER OF PIPE INCHES	SLOPE	FALL OF SEWER FT	VELOCITY OF FLOW FT/SEC.	INVERT LEVEL: UPPER M.H. FT	INVERT LEVEL: LOWER M.H. FT	CAPACITY FLOWING FULL CFS
SEWERS OF PUMPING STATION PS I																				
MAIN SEWER M_3																				
$M_3H_7 - M_3H_6$	1040	7.56	61	462	18480	18480	32.1	9.6	9.6	41.7	0.093	12.00	9.84	8	.00325	3.38	2	9.33	5.95	0.70
$M_3H_6 - M_3H_5$	620	11.08	31	344	13760	32240	56.0	7.2	16.8	72.8	0.162	9.84	8.40	8	.00325	2.62	2	5.95	3.33	0.70
$H_5 - H_4$	790	6.55	31	203	8120	40360	70.0	4.2	21.0	91.0	0.202	8.40	5.20	8	.00325	2.57	2	3.33	0.76	0.70
$H_4 - H_3$	440	5.65	31	175	7000	47360	82.2	3.7	24.7	106.9	0.238	5.20	3.00	8	.00325	1.44	2	0.76	-0.68	0.70
$H_3 - H_2$	110	3.14	31	97	3880	51240	88.8	2.0	26.7	115.5	0.257	3.00	2.60	8	.00325	0.36	2	-0.68	-1.04	0.70
$H_2 - H_1$	795	3.74	31	116	4640	55880	97.0	2.4	29.1	126.1	0.280	2.60	2.30	8	.00325	2.58	2	-1.04	-3.62	0.70
$H_1 - T_1H_3$	220	3.37	31	104	4160	60040	104.0	2.2	31.3	134.3	0.299	2.30	2.10	8	.00325	0.72	2	-3.62	-4.34	0.70
MAIN SEWER M_2																				
$M_2H_3 - M_2H_2$	725	5.87	31	182	7280	7280	12.7	3.8	3.8	16.5	0.037	6.80	3.28	8	.00325	2.46	2	4.13	1.67	0.70
$H_2 - H_1$	400	6.40	31	198	7920	15200	26.4	4.1	7.9	34.3	0.074	3.28	2.00	8	.00325	1.30	2	1.67	0.37	0.70
$H_1 - H_0$	150	5.95	31	185	7400	22600	39.3	3.9	11.8	51.1	0.114	2.00	2.00	8	.00325	0.49	2	0.37	-0.12	0.70
MAIN SEWER M_0																				
$M_0H_4 - M_0H_3$	470	4.67	31	144	5760	5760	10	3.0	3.0	13.0	0.029	2.0	2.0	8	.00325	1.53	2	-0.67	-2.20	0.70
$H_3 - H_2$	440	8.63	31	267	6680	12440	21.6	5.6	8.6	30.2	0.067	2.0	2.0	8	.00325	1.43	2	-2.20	-3.63	0.70
$H_2 - H_1$	480	6.66	31	207	8280	20720	36.0	4.3	12.9	48.9	0.107	2.0	2.0	8	.00325	1.56	2	-3.63	-5.19	0.70
$H_1 - T_1H_1$	630	6.29	31	195	7800	28520	49.5	4.1	17.0	66.5	0.148	2.0	2.0	8	.00325	2.05	2	-5.19	-7.24	0.70
TRUNK SEWER T_1																				
$T_1H_7 - T_1H_6$	720	5.0	31	155	6200	6200	10.8	3.2	3.2	14.0	0.031	6.20	6.15	8	.00325	2.34	2	3.53	1.19	0.70
$H_6 - H_5$	750	6.9	31	212	8480	14680	25.5	4.4	7.6	33.1	0.074	6.15	4.92	8	.00325	2.44	2	1.19	-1.25	0.70
$H_5 - H_4$	800	19.28	31	595	23800	38480	66.8	12.4	20.0	86.8	0.193	4.92	2.00	8	.00325	2.60	2	-1.25	-3.85	0.70
$H_4 - H_3$	770	3.37	31	108	4320	42800	74.4	2.3	22.3	96.7	0.215	2.00	2.00	8	.00325	2.50	2	-3.85	-6.35	0.70
$H_3 - H_2$	220	M_3	31	1563	62520	105320	183.0	32.6	54.9	237.9	0.529	2.00	2.00	8	.00325	0.71	2	-6.35	-7.06	0.70
$H_2 - H_1$	210	1.95	31	60	2400	107720	187.0	1.3	56.2	243.2	0.540	2.00	2.00	8	.00325	0.68	2	-7.06	-7.84	0.70
$T_1H_1 - H_0$	100	M_0	31	813	32520	140240	244	17	73.2	317.2	0.688	2.00	2.00	8	.00325	0.25	2	-7.84	-8.09	0.70
SEWERS OF PUMPING STATION PS II																				
MAIN SEWER M_6																				
$M_6H_3 - M_6H_2$	570	8.16	61	498	19,920	19920	34.6	10.4	10.4	45.0	0.100	21.32	11.48	8	0.017	9.84	4.5	18.65	8.81	1.65
$H_2 - H_1$	500	4.19	61	255	10200	30120	52.4	5.3	15.7	68.1	0.151	11.48	6.56	8	0.01	4.92	3.5	8.81	3.89	1.20
$H_1 - T_2H_6$	650	3.51	61	212	8480	3860	67.1	4.4	20.1	87.2	0.196	6.56	3.25	8	0.005	3.31	2.0	3.89	0.58	0.70
MAIN SEWER M_5																				
$M_5H_3 - M_5H_2$	450	5.73	61	350	14000	14000	24.3	7.3	7.3	31.6	0.07	18.04	11.48	8	.0145	6.53	3.85	15.37	8.84	1.35
$H_2 - H_1$	550	5.76	61	352	14080	28080	48.8	7.3	14.6	63.4	0.141	11.48	6.56	8	.09	4.95	4.50	8.84	3.89	1.60
$H_1 - T_2H_5$	660	6.22	61	380	15200	43280	75.2	7.9	22.5	97.7	0.216	6.56	3.80	8	.00425	2.80	2.30	3.89	1.09	0.80
MAIN SEWER M_4																				
$M_4H_3 - M_4H_2$	900	6.25	61	381	15240	15240	26.4	7.9	7.9	34.3	0.076	11.48	6.56	8	.005	4.50	2.5	8.81	4.31	0.90
$M_4H_4 - M_4H_2$	900	7.22	61	441	17640	32880	56.8	9.2	17.1	73.9	0.164	14.76	6.56	8	.00875	7.86	3.3	12.09	4.23	1.10
$H_2 - H_1$	480	4.72	61	288	11520	44400	77.1	6.0	23.1	100.2	0.223	6.56	5.75	8	.00325	1.56	2.0	4.23	2.67	0.70
$H_1 - T_2H_4$	500	7.12	61	434	17360	61760	108.0	9.1	32.2	140.2	0.312	5.75	4.20	8	.00325	1.63	2.0	2.67	1.04	0.70

MANHOLE NO.	LENGTH FT.	INCRT. AREA ACRES	POPLN. DENSITY PERSONS PER ACRE	INCRT. POPLT.	DOMESTIC SEWAGE INCRT. DISCHARGE GPD	DOMESTIC SEWAGE CUMUL DISCHARGE GPD	DOMESTIC SEWAGE MAX. DISCHARGE GPM	INFILTERATION INCRT FLOW GPM	INFILTERATION CUMUL. FLOW GPM	TOTAL DISCHARGE GPM	TOTAL DISCHARGE CFS	GROUND ELEVATION UPPER MH FT	GROUND ELEVATION LOWER MH FT	DIAMETER OF PIPE INCHES	SLOPE	FALL OF SEWER FT.	VELOCITY OF FLOW Ft/SEC.	INVERT LEVEL UPPER MH FT	INVERT LEVEL LOWER M.H. FT	CAPACITY FLOWING FULL CFS
MAIN SEWER M_3																				
$M_3H_5-M_3H_4$	450	8.42	61	514	20560	20560	35.7	10.7	10.7	46.4	0.103	12.30	9.90	8	.00525	2.36	2.5	9.63	7.27	0.90
H_4-H_3	450	3.74	61	228	9120	29680	51.5	4.8	15.5	67.0	0.149	9.90	8.80	8	.00325	1.46	2.0	7.27	5.81	0.70
H_3-H_2	460	2.92	61	178	7120	36800	63.9	3.7	19.2	83.1	0.185	8.80	7.40	8	.00325	1.49	2.0	5.81	4.32	0.70
H_2-H_1	800	4.57	61	278	11120	47920	83.2	5.8	25.0	108.0	0.240	7.40	5.30	8	.00325	2.60	2.0	4.32	1.72	0.70
$H_1-T_2H_3$	680	11.72	61	716	28640	76560	133.0	14.9	39.9	172.9	0.382	5.30	4.10	8	.00325	2.21	2.0	1.72	-0.49	0.70
MAIN SEWER M_1																				
$M_1H_4-M_1H_2$	710	7.11	31	220	8800	8800	15.3	4.6	4.6	19.9	0.042	2.00	3.40	8	.00325	2.30	2.0	-0.67	-2.97	0.70
$M_1H_5-M_1H_2$	700	DISCHARGE FROM PUMPING STATION I									0.704	2.00	3.40	10	.0025	1.75	2.0	-0.67	-2.45	1.10
$M_1H_2-M_1H_1$	600	9.38	31	291	11640	20440	36.5	6.1	10.7	47.2	.105+.704 =0.809	3.40	3.28	10	.0025	1.50	2.0	-2.97	-4.47	1.10
$M_1H_1-T_2H_1$	320	21.15	31	655	26200	46640	80.8	13.7	24.4	105.2	.22+.704 =.924	3.28	2.60	10	.0025	0.80	2.0	-4.47	-5.27	1.10
TRUNK SEWER T_2																				
$T_2H_6-T_2H_5$	320	M_6, 3.51	31	1073	42920	42920	74.5	22.4	22.4	96.9	0.215	3.25	3.80	8	.00325	1.04	2.0	0.58	-0.46	0.70
H_5-H_4	550	M_5, 6.34	31	1198	47920	90840	157.7	24.9	47.3	205.0	0.456	3.80	4.20	10	.0025	1.37	2.0	-0.56	-1.93	1.10
H_4-H_3	660	M_4	.	1544	61760	152600	265.0	32.2	79.5	344.5	0.765	4.20	4.10	10	.0025	1.65	2.0	-2.03	-3.68	1.10
H_3-H_2	640	M_3	.	1914	76560	229160	398.0	39.9	119.4	517.4	1.148	4.10	3.40	12	.002	1.28	2.0	-3.78	-5.06	1.57
H_2-H_2'	200	17.78	31	554	18120	247280	429.0	9.5	128.9	557.9	1.240	3.40	3.20	12	.002	0.40	2.0	-5.16	-5.56	1.57
$H_2'-H_1$	450	8.79	31	272	10880	258160	448.0	5.7	134.6	582.6	1.295	3.20	2.60	12	.002	0.90	2.0	-5.66	-6.76	1.57
$T_2H_1-T_2H_0$	100	M_1, M_2, PS I	.	2025	81000	339160	589.0	42	176.6	765.6	1.7+0.704	2.60	2.60	15	.0015	0.15	2.0	-6.86	-7.01	2.50
MAIN SEWER M_2																				
$M_2H_3-M_2H_2$	810	6.93	31	215	8600	8600	14.9	4.5	4.5	19.4	0.043	2.00	2.50	8	.00325	2.63	2	-0.67	-3.30	0.70
H_2-H_1	500	6.18	31	192	7680	16280	28.2	4.0	8.5	36.7	0.082	2.50	2.60	8	.00325	1.62	2	-3.30	-4.92	0.70
$H_1-T_2H_1$	350	14.60	31	452	18080	34360	59.5	9.5	18.0	77.5	0.172	2.60	2.60	8	.00325	1.14	2	-4.92	-6.06	0.70

DESIGN OF SEWERS NETWORK

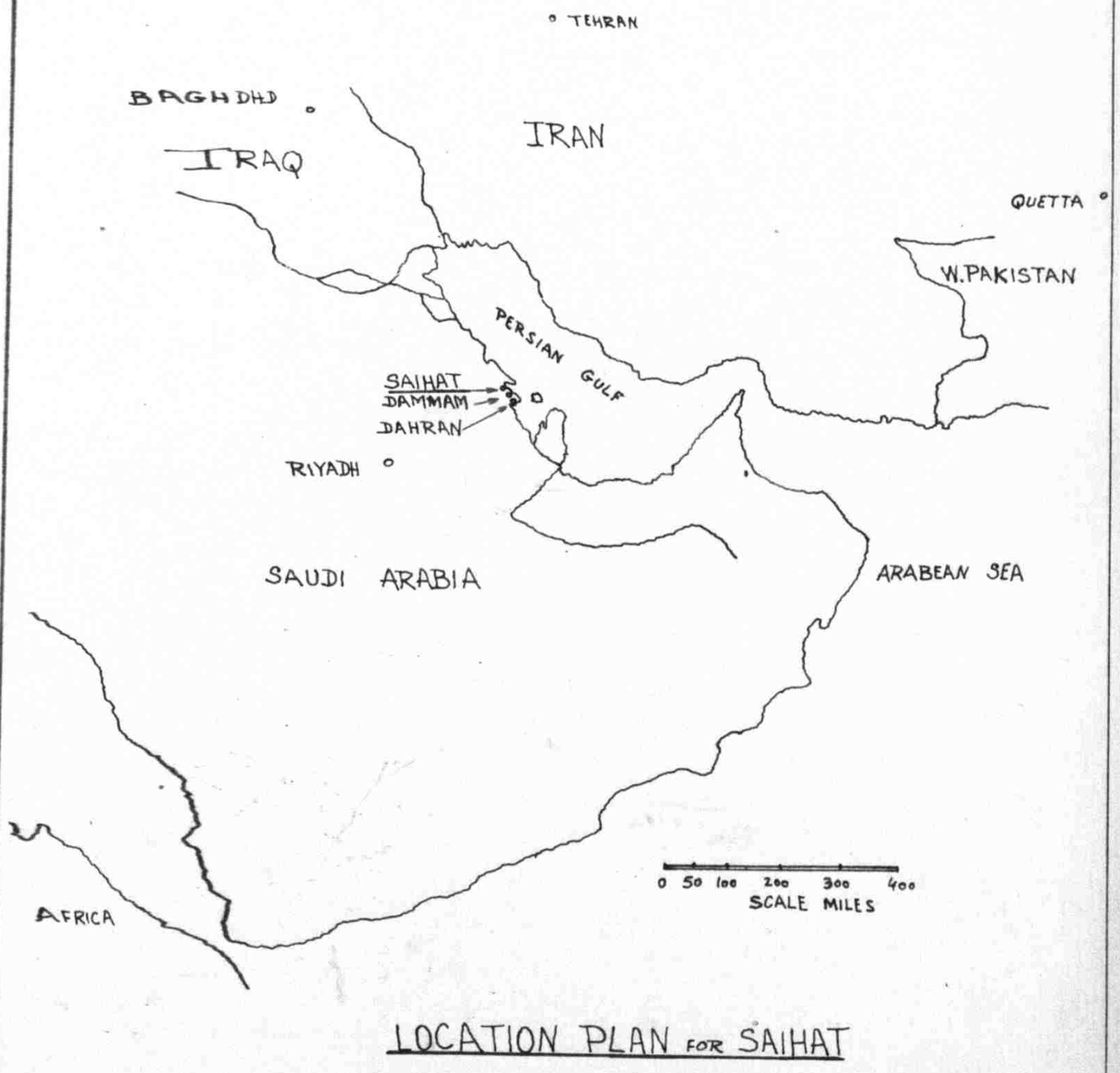
SHEET No: 3

MANHOLE No	LENGTH Ft.	INCRT. AREA ACRE	POPLN. DENSITY POPULATION PER ACRE	INCRT. POPLTN.	DOMESTIC SEWAGE: INCRT DISCHARGE GPD	DOMESTIC SEWAGE: CUMUL. DISCHARGE GPD	DOMESTIC SEWAGE: MAX. DISCHARGE GPM	INFILTERATION: INCRT. FLOW GPM	INFILTERATION: CUMUL FLOW GPM	TOTAL DISCHARGE GPM	TOTAL DISCHARGE CFS.	GROUND ELEVATION: UPPER M.H. FT	GROUND ELEVATION: LOWER MH FT	DIAMETER OF PIPE INCH	SLOPE	FALL OF SEWER FT	VELOCITY OF FLOW FT/SEC	INVERT LEVEL: UPPER MH FT	INVERT LEVEL: LOWER MH FT	CAPACITY FLOWING FULL CFS
SEWERS OF PUMPING STATION PS III																				
MAIN SEWER M_7																				
M_7H_6 - M_7H_5	850	3.18	26	83	3320	3320	5.8	1.7	1.7	7.5	0.017	3.80	3.28	8	.00325	2.76	2.0	1.13	-1.63	0.70
H_5 - H_4	410	2.77	26	72	2880	6200	10.7	1.5	3.2	13.9	0.031	3.28	3.30	8	.00325	1.33	2.0	-1.63	-2.96	0.70
H_4 - H_3	200	5.69	26	148	5920	12120	20.0	3.1	6.3	26.3	0.059	3.30	3.30	8	.00325	0.65	2.0	-2.96	-3.61	0.70
H_3 - H_2	630	5.98	26	155	6200	18320	31.8	3.2	9.5	41.3	0.092	3.30	3.25	8	.00325	2.05	2.0	-3.61	-5.66	0.70
H_2 - H_1	400	5.05	26	131	5240	23560	40.8	2.7	12.2	53.0	.0120	3.25	3.28	8	.00325	1.30	2.0	-5.66	-6.96	0.70
H_1 - T_3H_3	180	6.29	26	164	6560	30120	52.4	3.4	15.6	68.0	.0150	3.28	3.30	8	.00325	0.58	2.0	-6.96	-7.54	0.70
MAIN SEWER M_6																				
M_6H_7 - M_6H_6	740	13.67	26	355	14200	14200	24.6	7.4	7.4	32.0	0.071	3.80	3.80	8	.00325	2.40	2.0	1.13	-1.27	0.70
H_6 - H_5	200	2.36	26	61	2440	16640	28.9	1.3	8.7	37.6	0.082	3.80	3.80	8	.00325	0.65	2.0	-1.27	-1.92	0.70
H_5 - H_4	220	2.40	26	62	2480	19120	33.2	1.3	10.0	43.2	0.096	3.80	3.80	8	.00325	0.71	2.0	-1.92	-2.63	0.70
H_4 - H_3	200	2.44	26	63	2520	21640	37.6	1.3	11.3	48.9	0.109	3.80	3.90	8	.00325	0.61	2.0	-2.63	-3.24	0.70
H_3 - H_2	350	6.74	26	176	7040	28680	49.8	3.7	15.0	64.8	0.144	3.90	3.90	8	.00325	1.14	2.0	-3.24	-4.38	0.70
H_2 - H_1	200	3.56	26	94	3760	32440	56.4	2.0	17.0	73.4	0.163	3.90	3.90	8	.00325	0.65	2.0	-4.38	-5.03	0.70
H_1 - T_3H_4	200	3.74	26	97	3880	36320	63.0	2.0	19.0	82.0	0.182	3.90	3.90	8	.00325	0.65	2.0	-5.03	-5.68	0.70
MAIN SEWER M_5																				
M_5H_3 - M_5H_2	1660	6.93	26	180	7200	7200	12.5	3.8	3.8	16.3	0.036	5.30	4.20	8	.00325	5.39	2.0	+2.63	-2.76	0.70
H_2 - H_1	200	7.49	26	206	8240	15440	26.8	4.3	8.1	34.9	0.078	4.20	4.20	8	.00325	0.65	2.0	-2.76	-3.41	0.70
H_1 - T_3H_5	300	9.36	26	244	9760	25200	43.7	5.1	13.2	56.9	0.126	4.20	4.10	8	.00325	0.97	2.0	-3.41	-4.38	0.70
MAIN SEWER M4																				
M_4H_8 - M_4H_7	950	3.86	26	100	4000	4000	7.0	2.1	2.1	9.1	0.020	4.90	4.60	8	.00325	3.08	2.00	2.13	-0.95	0.70
H_7 - H_6	450	3.93	26	105	4200	8200	14.2	2.2	4.3	18.3	0.041	4.60	4.50	8	.00325	1.46	2.00	-0.95	-2.41	0.70
H_6 - H_5	180	9.63	26	250	10,000	18200	32.6	5.2	9.5	42.1	0.094	4.50	4.40	8	.00325	0.58	2.00	-2.41	-2.99	0.70
H_5 - H_4	200	5.05	26	131	5240	23440	40.7	2.7	12.2	52.9	0.117	4.40	4.30	8	.00325	0.65	2.00	-2.99	-3.61	0.70
H_4 - H_3	400	4.98	26	130	5200	28640	49.7	2.7	14.9	64.6	0.144	4.30	4.20	8	.00325	1.30	2.00	-3.61	-4.91	0.70
H_3 - H_2	340	3.37	26	88	3520	32160	56.0	1.8	16.7	72.7	0.162	4.20	4.10	10	.0025	0.85	2.00	-4.91	-5.76	1.10
H_2 - H_1	200	3.18	26	83	3320	35480	61.6	1.7	18.4	80.0	0.178	4.10	4.05	10	.0025	0.50	2.00	-5.76	-6.26	1.10
M_4H_1 - T_3H_4	260	3.11	26	81	3240	38720	67.2	1.7	20.1	87.3	0.194	4.05	3.90	10	.0025	0.65	2.00	-6.26	-6.91	1.10
MAIN SEWER M_3																				
M_3H_2 - M_3H_1	900	4.12	26	107	4280	4280	7.4	2.2	2.2	9.6	0.021	4.00	3.35	8	.00325	2.92	2.0	1.33	-1.59	0.70
M_3H_1 - T_3H_3	260	5.16	26	135	5480	9680	16.8	2.8	5.0	21.8	0.049	3.35	3.30	8	.00325	0.85	2.0	-1.59	-2.44	0.70
MAIN SEWER S_1																				
S_1H_3 - S_1H_2	600	4.49	26	116	4640	4640	8.1	2.4	2.4	10.5	.030	2.5	3.10	8	.00325	1.95	2.0	-0.17	-2.25	0.70
S_1H_2 - S_1H_1	200	(PSII+3.15)	26	82	3280	7920	13.7	1.7	4.1	17.8	.040+2.40	3.10	3.10	12	.005	1.00	3.2	-2.25	-3.25	2.56
S_1H_1 - M_1H_1	320	7.94	26	206	8240	16160	28.0	4.3	8.4	26.4	.060+2.40	3.10	3.10	12	.005	2.60	3.2	-3.25	-5.85	2.56
MAIN SEWER S_2																				
S_2H_4 - S_2H_3	500	2.62	26	68	2720	2720	4.7	1.4	1.4	6.1	0.013	2.50	2.50	8	.00325	1.62	2	-0.17	-1.79	0.70
H_3 - H_2	200	2.44	26	63	2520	5240	9.1	1.3	2.7	11.8	0.026	2.50	2.70	8	.00325	0.65	2	-1.79	-2.44	0.70
H_2 - H_1	220	3.00	26	78	3120	8360	14.5	1.6	4.3	18.8	0.042	2.70	2.90	8	.00325	0.71	2	-2.44	-3.15	0.70
H_1 - M_1H_1	200	7.70	26	200	8000	16360	28.4	4.2	8.5	26.9	0.060	2.90	3.10	8	.00325	0.65	2	-3.15	-3.80	0.70

DESIGN OF SEWERS NETWORK

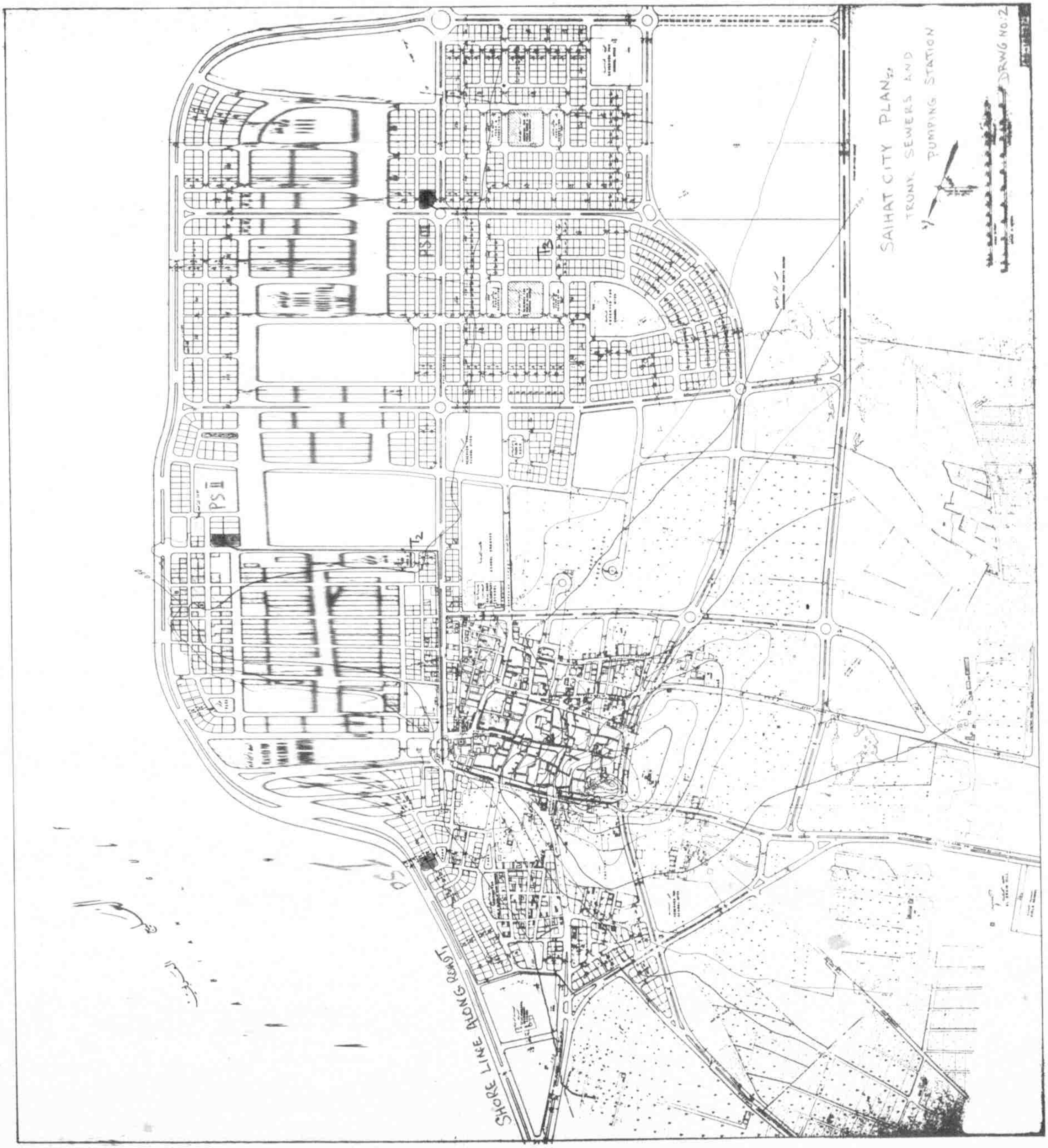
SHEET NO: 4

MANHOLE NO	LENGTH FT.	INCR. AREA ACRE	POPLN. DENSITY PERSONS PER ACRE	INCRMT. POPLTN.	DOMESTIC SEWAGE INCRMT. DISCHARGE GPD	CUMUL. DISCHARGE GPD	MAX. DISCHARGE GPM	INFILTERATION INCRMT. FLOW GPM	CUMUL. FLOW GPM	TOTAL DISCHARGE GPM	TOTAL DISCHARGE CFS	GROUND ELEVATION UPPER MH FT	LOWER MH FT	DIAMETER OF PIPE INCHES	SLOPE	FALL OF SEWER FT.	VELOCITY OF FLOW FT/SEC	INVERT LEVEL UPPER MH FT	LOWER MH FT	VELOCITY FLOWING FULL CFS
SEWERS OF PUMPING STATION III CONTINUED																				
MAIN SEWER S_3																				
$S_3H_3 - S_3H_2$	720	1.98	26	52	2080	2080	3.6	1.1	1.1	4.7	0.010	4.40	4.10	8	.00325	2.34	2	1.73	-0.61	0.70
$H_2 - H_1$	270	3.37	26	88	3520	5600	9.7	1.8	2.9	12.6	0.028	4.10	4.00	8	.00325	0.87	2	-0.61	-1.48	0.70
$H_1 - M_1H_6$	260	3.56	26	93	3720	9320	16.2	1.9	4.8	21.0	0.047	4.00	3.80	8	.00325	0.80	2	-1.48	-2.28	0.70
MAIN SEWER M_1																				
$M_1H_7 - M_1H_6$	1300	7.94	26	206	8240	8240	14.4	4.3	4.3	18.7	0.042	4.60	3.80	8	0.00325	4.22	2	1.93	-2.29	0.70
$H_6 - H_5$	200	S_3	26	233	9320	17560	30.4	4.9	9.2	39.6	0.088	3.80	3.70	8	0.00325	0.65	2	-2.29	-2.94	0.70
$H_5 - H_4$	400	2.40	26	62	2480	20040	34.8	1.3	10.5	45.3	0.100	3.70	3.70	8	.00325	1.30	2	-2.94	-4.24	0.70
$H_4 - H_3$	200	6.10	26	158	6320	26360	45.6	3.3	13.8	59.4	0.132	3.70	3.60	8	.00325	0.65	2	-4.22	-4.87	0.70
$H_3 - H_2$	380	5.88	26	153	6120	32480	56.4	3.2	17.0	73.4	0.163	3.60	3.20	8	.00325	1.24	2	-4.87	-6.11	0.70
$H_2 - H_1$	250	7.67	26	200	8000	40480	70.3	4.2	21.2	91.5	0.203	3.20	3.10	10	.0025	0.62	2	-6.21	-6.83	1.10
$H_1 - T_3H_1$	1100	S_1, S_2 7.26 + PS II	26	1002	40080	80560	140.0	21.2	42.4	182.4	0.405 + 2.400 = 2.805	3.10	3.00	15	.0025	2.75	2.5	-6.93	-9.68	3.00
TRUNK SEWER T_3																				
$T_3H_5 - T_3H_4$	730	19.47 M_5	26	1136	45440	45440	78.7	23.7	23.7	102.4	0.228	4.10	3.90	8	.00325	2.37	2	-4.48	-6.85	0.70
$H_4 - H_3$	660	M_4, M_6 3.93	26	1770	71,200	116640	202.6	37.1	60.8	263.4	0.585	3.90	3.30	10	.0025	1.65	2	-6.95	-8.60	1.10
$H_3 - H_2$	220	M_3, M_7	26	995	39,800	156440	271.6	20.7	81.5	353.1	0.784	3.30	3.20	12	.002	0.44	2	-8.70	-9.14	1.57
$H_2 - H_1$	230	11.98	26	312	12480	168920	293.5	6.5	88.0	381.5	0.848	3.20	3.00	12	.002	0.46	2	-9.24	-9.70	1.57
$H_1 - H_0$	150	M_8, M_1, S_6	26	681	27240	196260	340.8	11.2	100.2	442.0	0.985 + 2.805 + T_3 = 3.963	3.00	3.00	18	.002	0.30	2.7	-9.80	-10.10	4.80
MAIN SEWER M_8																				
$M_8H_2 - M_8H_1$	600	5.47	26	142	5680	5680	9.9	2.9	2.9	12.8	0.028	3.25	3.00	8	.00325	2.11	2	0.58	-1.53	0.70
$M_8H_1 - T_3H_1$	1550	20.76	26	539	21560	27240	47.3	11.2	14.1	61.4	0.137	3.00	3.00	8	.00325	5.04	2	-1.53	-6.57	0.70
MAIN SEWER S_5																				
$S_5H_4 - S_5H_3$	1300	5.98	26	156	6240	6240	10.8	3.3	3.3	14.1	0.031	2.00	2.00	8	.00325	4.22	2	-0.67	-4.89	0.70
$H_3 - H_2$	1800	5.62	26	146	5840	12080	20.9	3.0	6.3	27.2	0.061	2.00	2.00	8	.00325	0.58	2	-4.89	-5.47	0.70
$H_2 - H_1$	190	4.64	26	120	4800	16880	29.3	2.5	8.8	38.1	0.085	2.00	2.00	8	.00325	0.62	2	-5.47	-6.09	0.70
$H_1 - T_3H_7$	190	4.86	26	126	5040	21920	38.1	2.6	11.4	49.5	0.110	2.00	2.00	8	.00325	0.62	2	-6.09	-6.71	0.70
MAIN SEWER S_6																				
$S_6H_2 - S_6H_1$	1550	7.87	26	205	8200	8200	14.2	4.3	4.3	18.5	0.041	2.00	2.00	8	.00325	4.80	2	-0.67	-5.47	0.70
$S_6H_1 - T_3H_7$	640	11.92	26	310	12400	20600	35.8	6.5	10.8	46.6	0.103	2.00	2.00	8	.00325	2.06	2	-5.47	-7.53	0.70
$T_3H_7 - T_3H_6$	220	S_5 5.05	2.6	679	27160	47760	83.0	14.1	24.9	107.9	0.240	2.00	2.70	10	.0025	0.55	2	-7.53	-8.18	1.10
$T_3H_6 - T_3H_1$	200	4.67	2.6	122	4880	52640	91.5	2.5	27.4	118.9	0.263	2.70	3.00	10	.0025	0.50	2	-8.18	-8.68	1.10

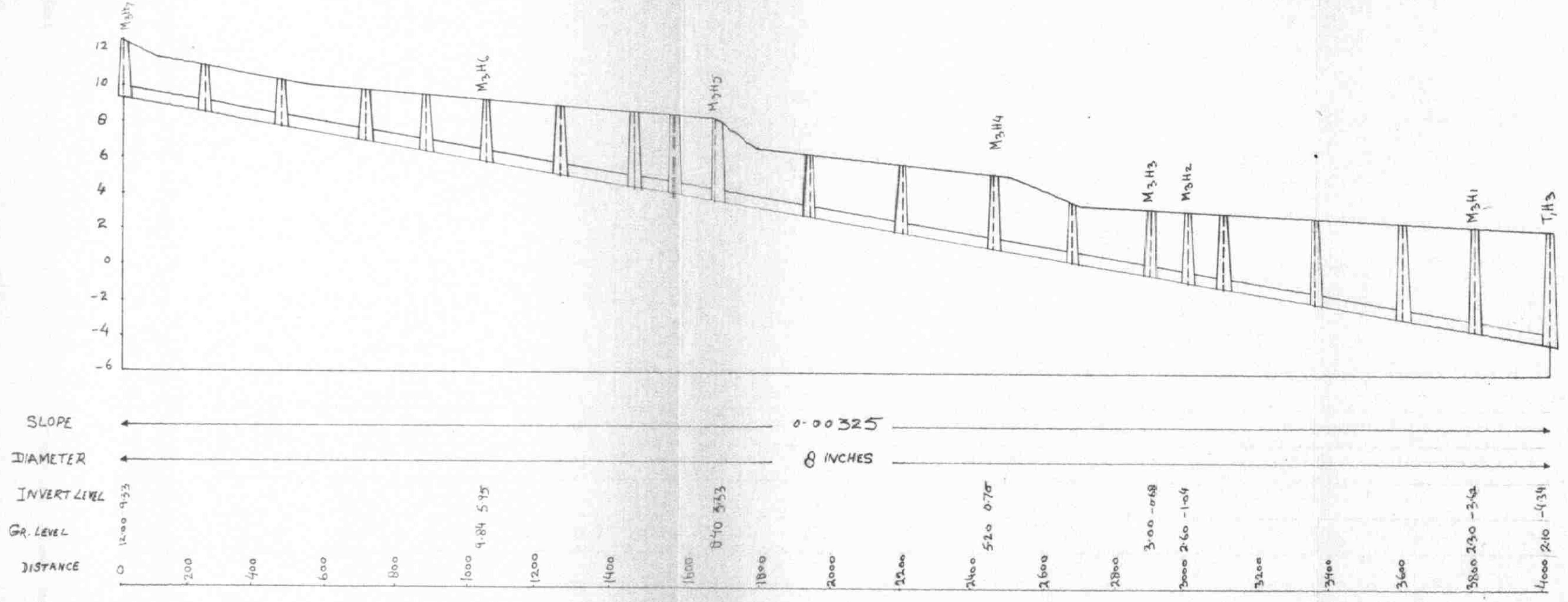


LOCATION PLAN FOR SAIHAT

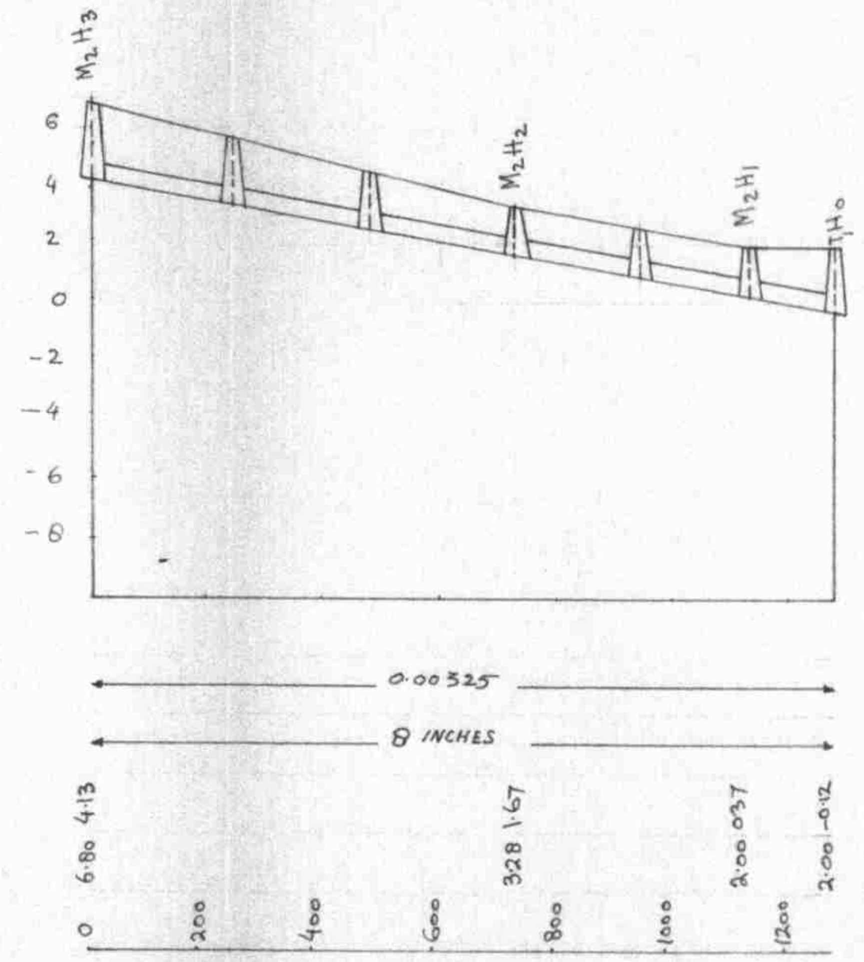
DRAWING NO: 1


SAIHAT CITY PLAN
TRUNK SEWERS AND PUMPING STATION
DRWG NO: 2
PS I
PS II
PS III
T2
T3
SHORE LINE ALONG COAST


الخليج العربي
ARABIAN GULF
TRUNK SEWER T_1
PS I
EMERGENCY OVER FLOW
PRESSURE SEWER
TRUNK SEWER T_2
PS II
10 PRESSURE SEWER
EMERGENCY OVER-FLOW
EMERGENCY OVER FLOW
PS III
10 PRESSURE
TRUNK SEWER T_3
SCHOOL GROUNDS
RESERVED FOR SCHOOL SITES
PARK & RECREATION
RESERVED FOR PUBLIC BLDGS & SHOPPING CENTER
RESERVED FOR SPORTS CENTER
AL GRAIN HILL
BIBLE SCHOOL
NOTE
TRUNK SEWER SHOWN
MAIN SEWER SHOWN
LATERAL SEWER SHOWN
MANHOLE SHOWN
POPULATION 61/ACRE
POPULATION 31/ACRE
POPULATION 26/ACRE
PRESSURE SEWER
ALL DIAMETER 8" EXCEPT AS SHOWN
SHORE LINE COINCIDED WITH COAST ROAD
SEWERAGE SYSTEM SAIHAT SAUDI ARABIA
SEWERAGE NETWORK
ENGINEER SYED MAZHAR ABBAS
DRAWING No: 3
DATE: FEBRUARY 1965
SCALE IN FEET
SCALE IN METERS



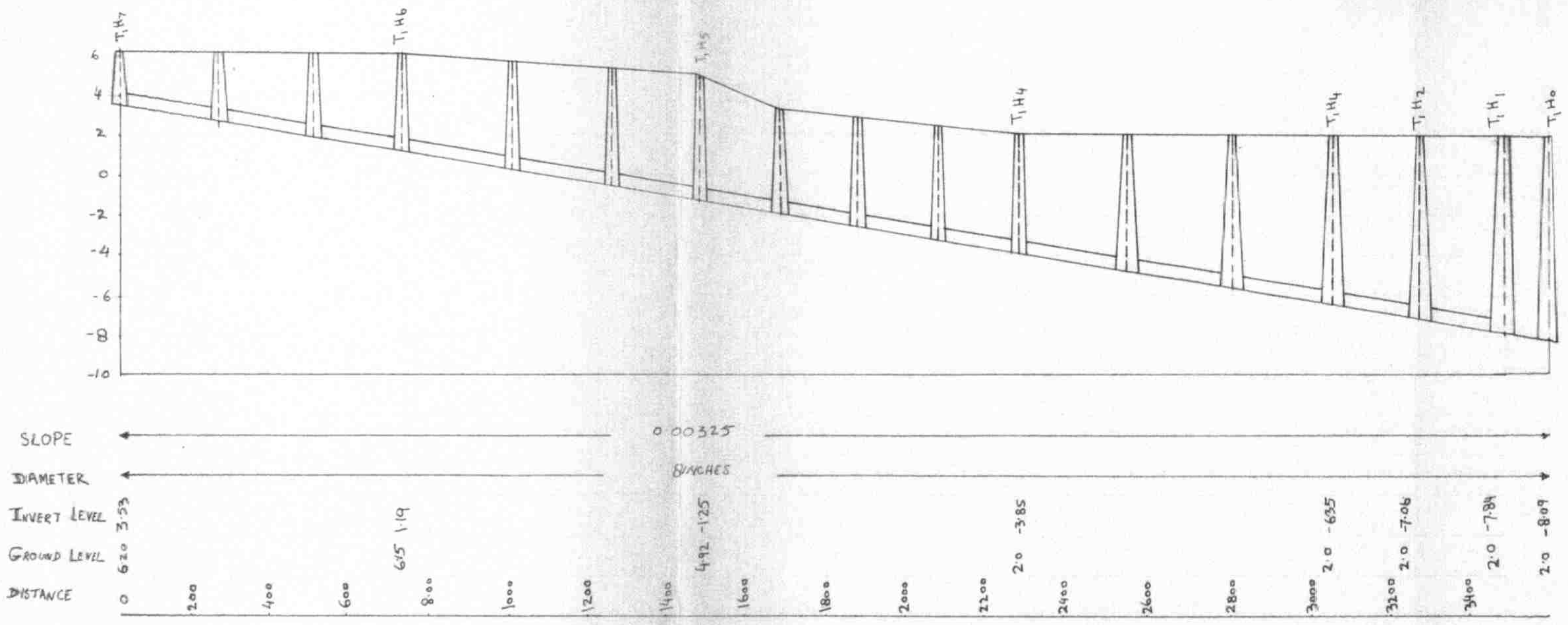
MAIN SEWER M_3



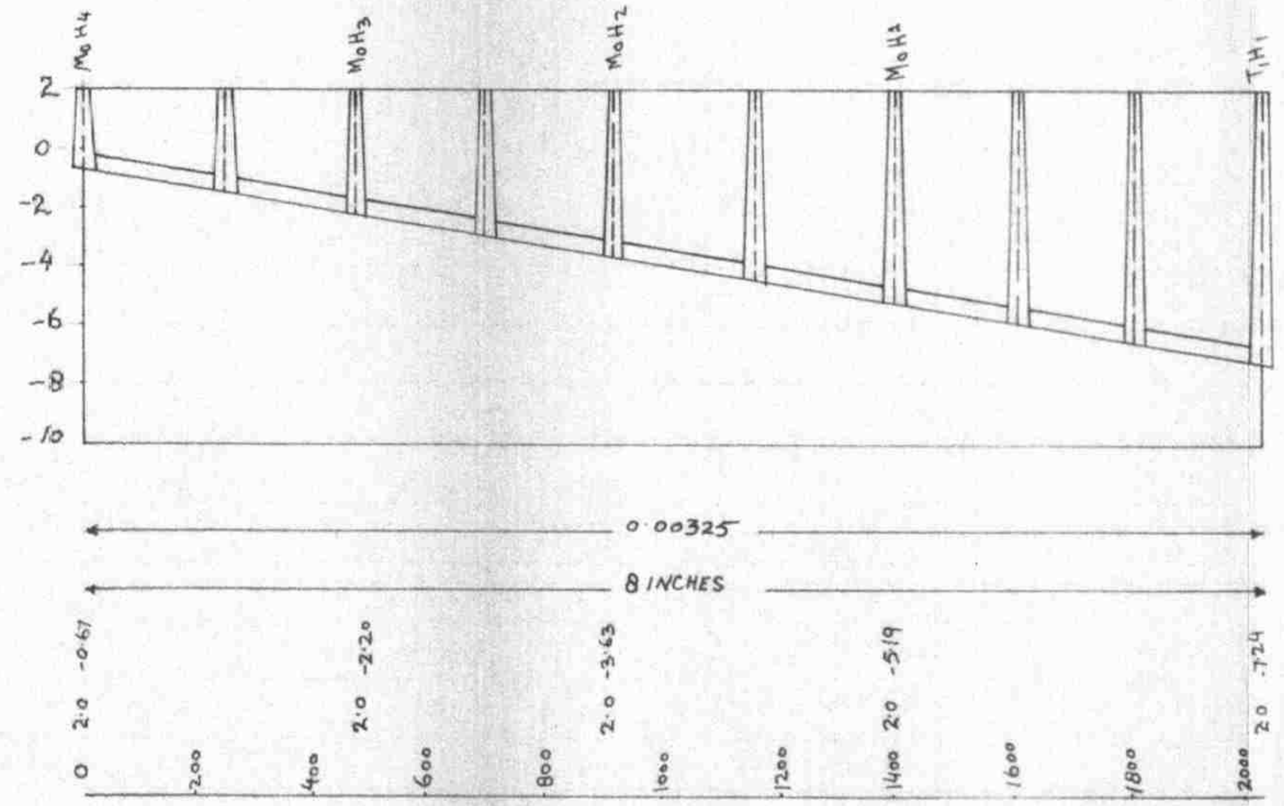
MAIN SEWER M_2

NOTE: GROUND ELEVATIONS OBTAINED FROM CONTOURS

SEWERAGE SYSTEM, SAIHAT SAUDI ARABIA	
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION I	
ENGINEER SYED MAZHAR ABBAS	
DRAWING NO: 4	SCALE: HOR: 250' : 1" VER: 5' : 1"
DATE FEBRUARY 1965	



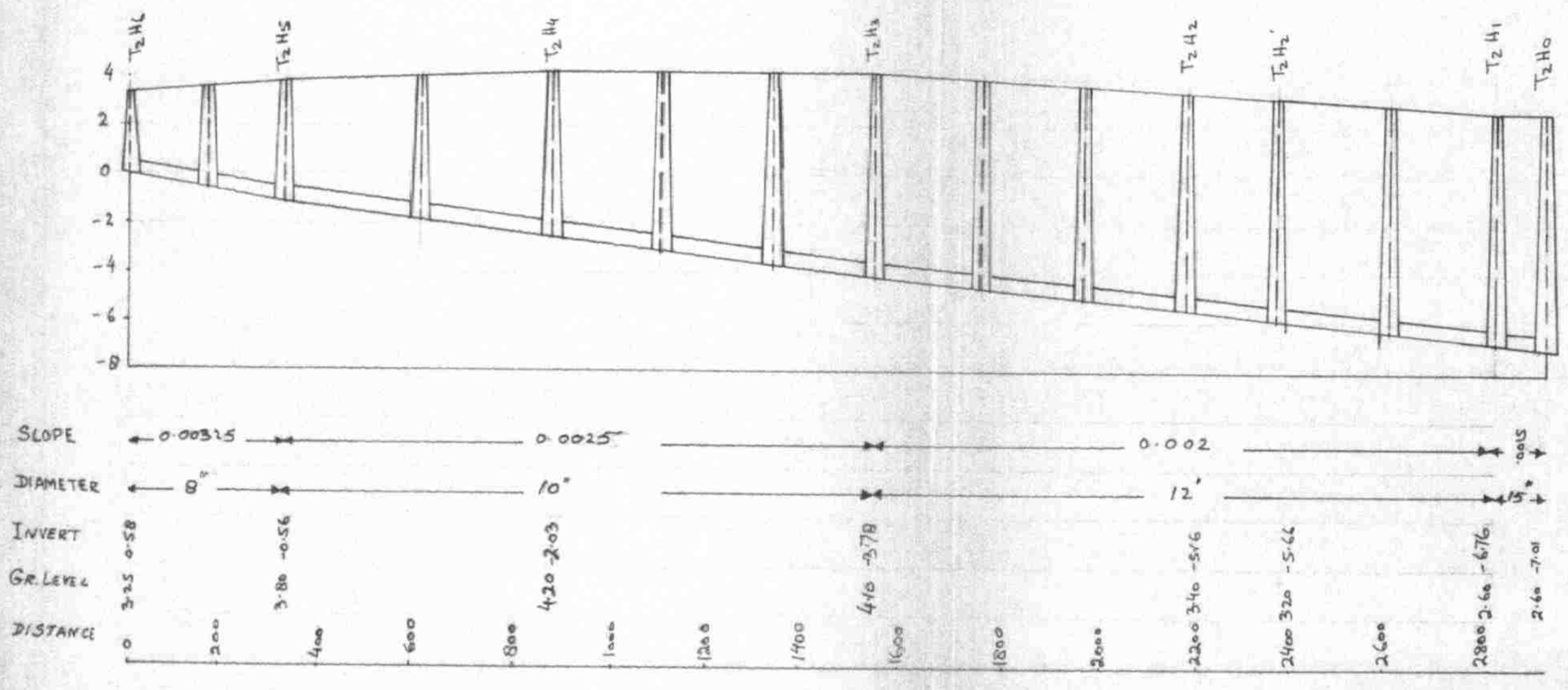
TRUNK SEWER T_1



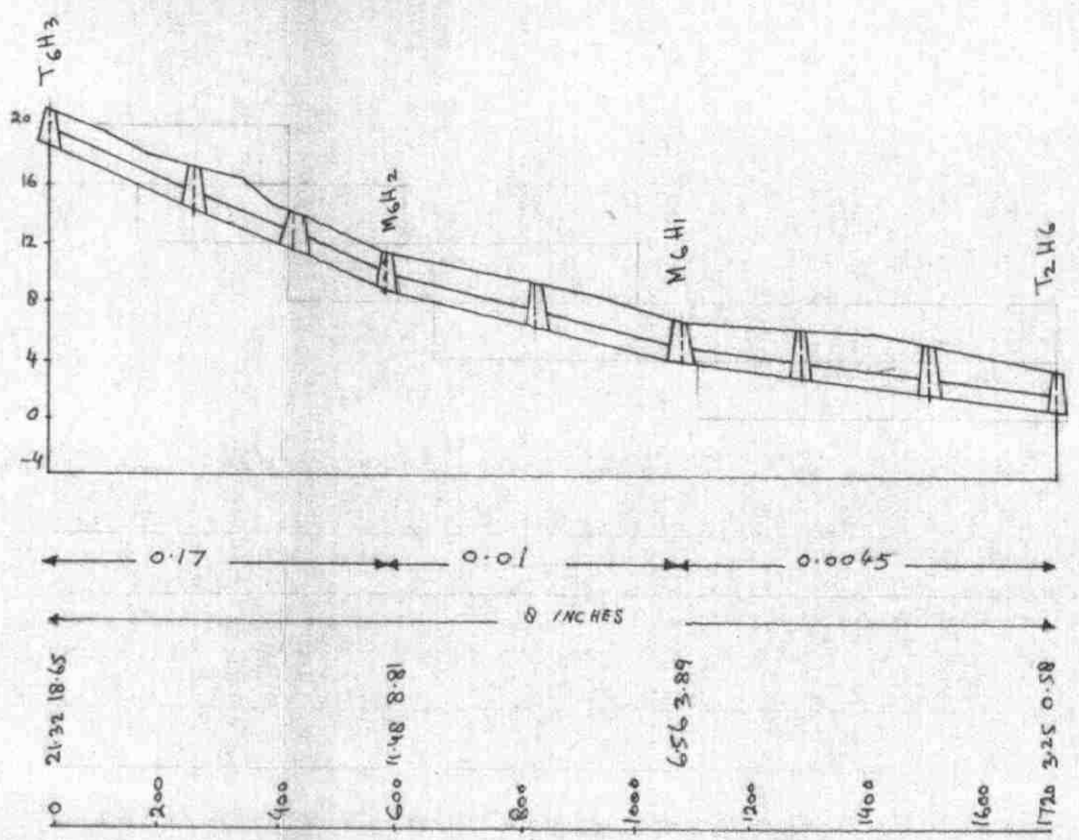
MAIN SEWER M_0

NOTE; GROUND ELEVATIONS OBTAINED FROM CONTOURS

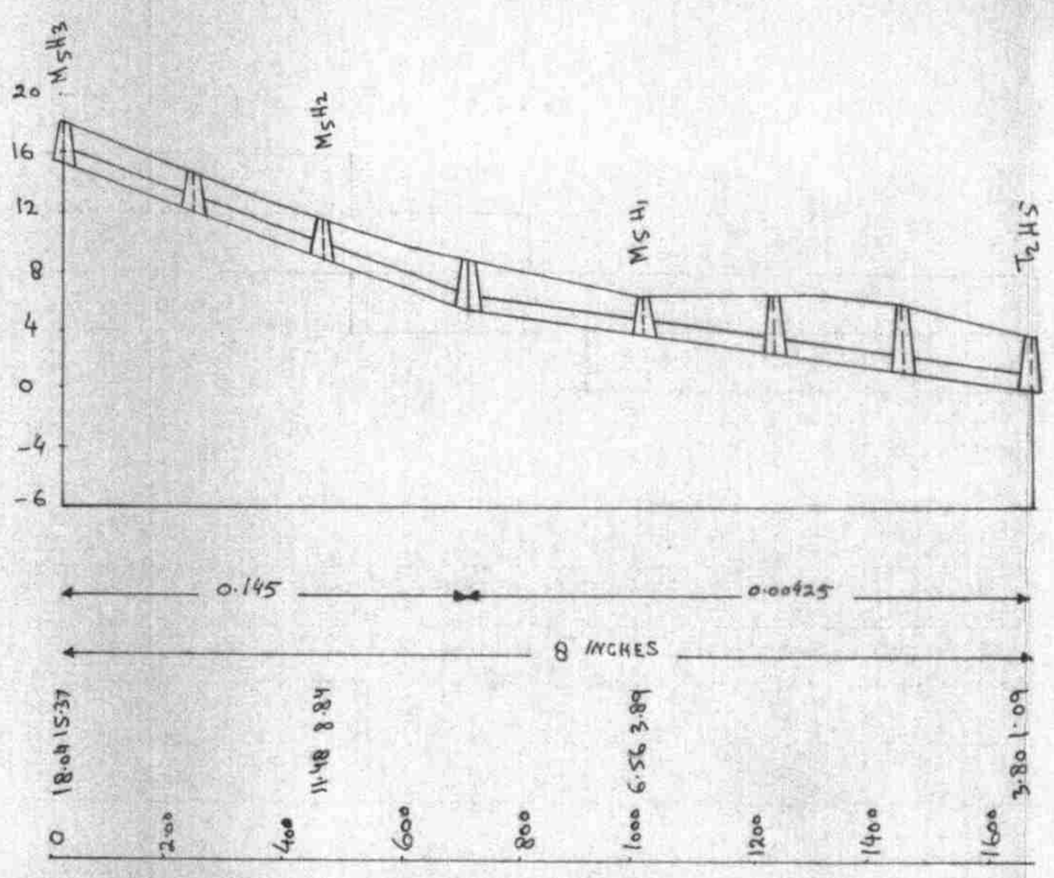
SEWERAGE SYSTEM SAIHAT, SAUDI ARABIA	
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION I	
ENGINEER SYED MAZHAR ABBAS	
DRAWING No: 5	SCALE: HOR: 250' = 1" VER: 5' = 1"
DATE FEBRUARY 1965	



TRUNK SEWER T_2



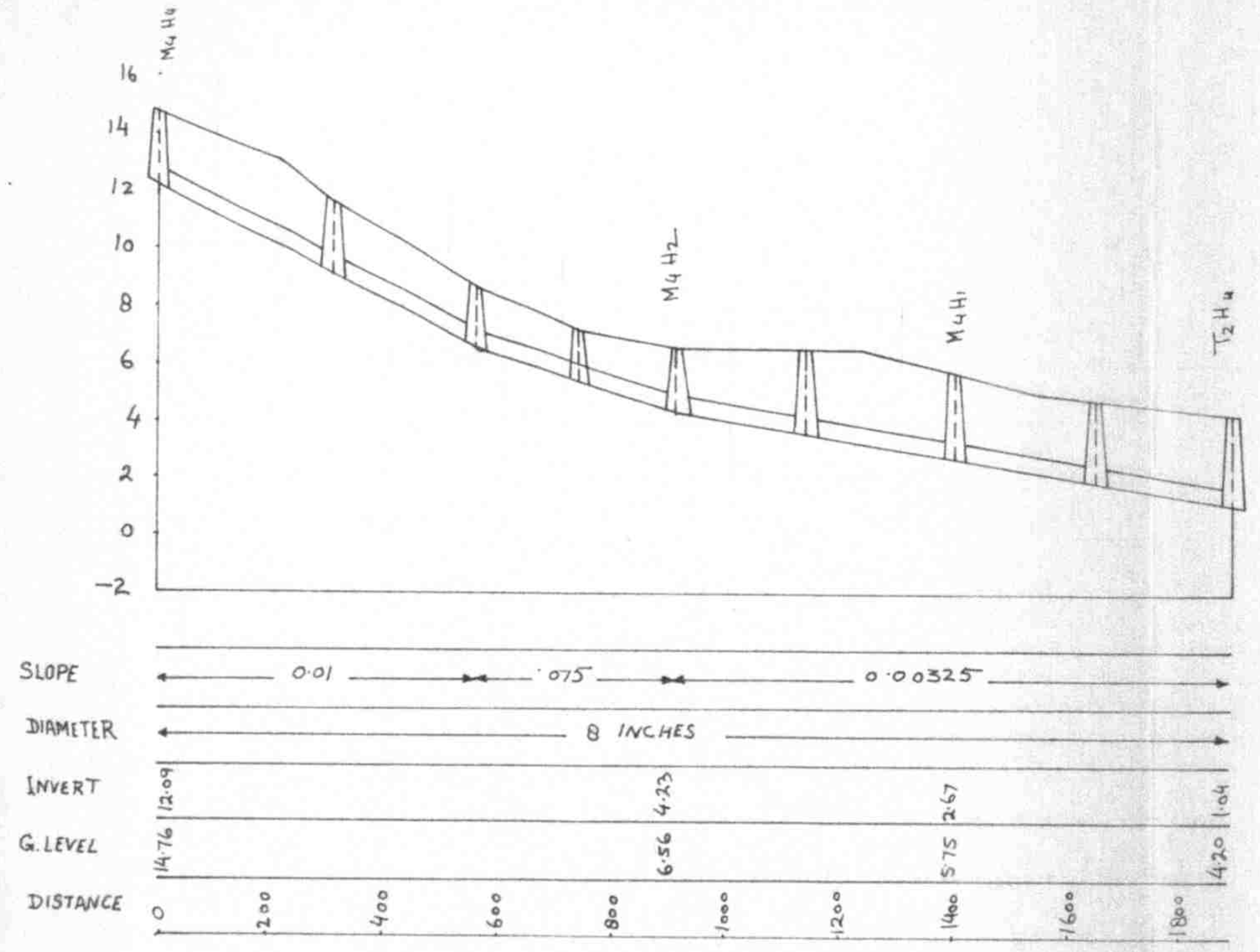
MAIN SEWER M_6



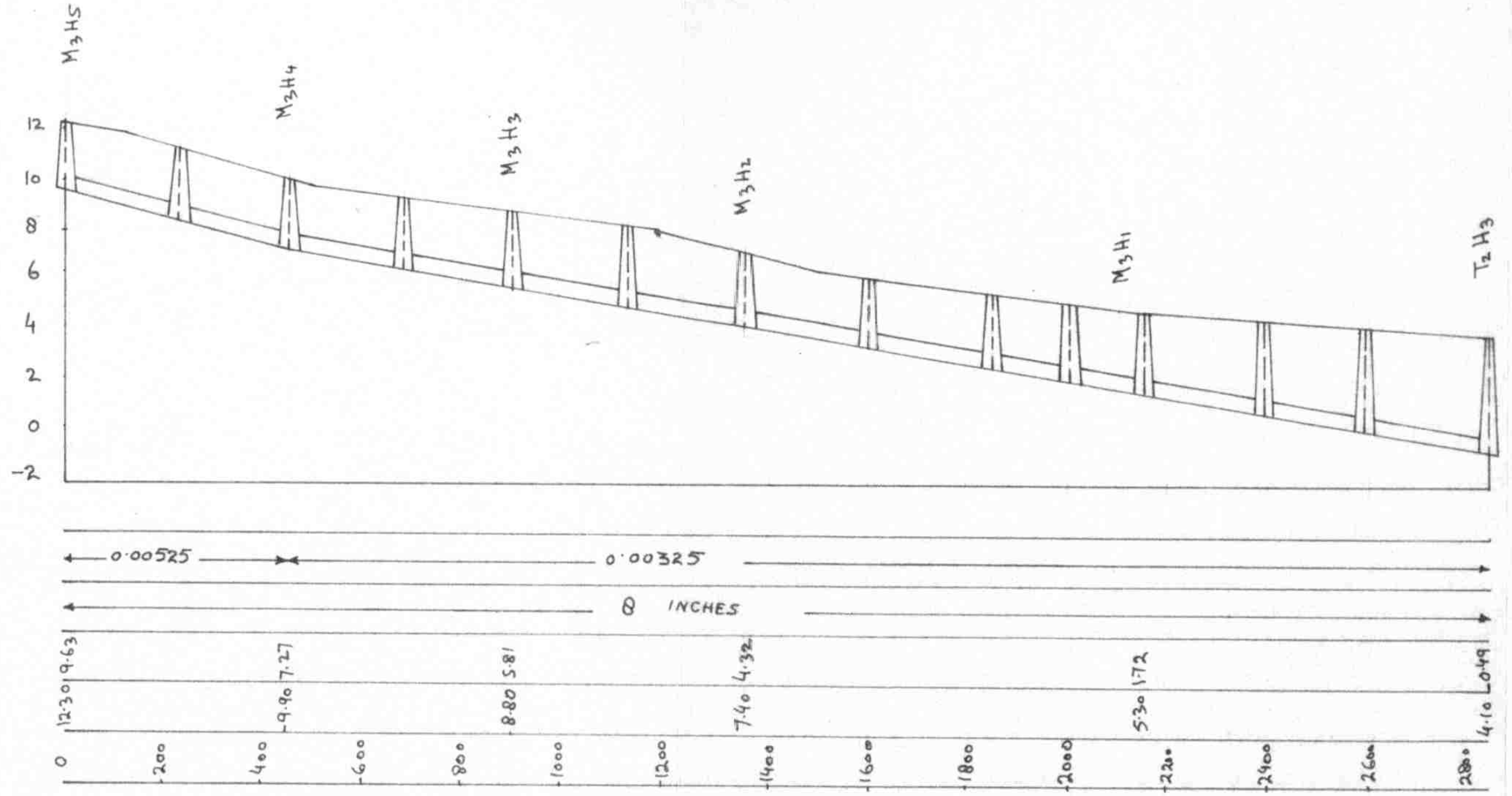
MAIN SEWER M_5

NOTE: GROUND ELEVATIONS OBTAINED FROM CONTOURS

SEWERAGE SYSTEM SAIHAT SAUDI ARABIA	
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION II	
ENGINEER SYED MAZHAR ABBAS	
DRAWING No: 6	SCALE: HOR: 250' = 1" VER: 5'/10' = 1"
DATE FEBRUARY 1965	



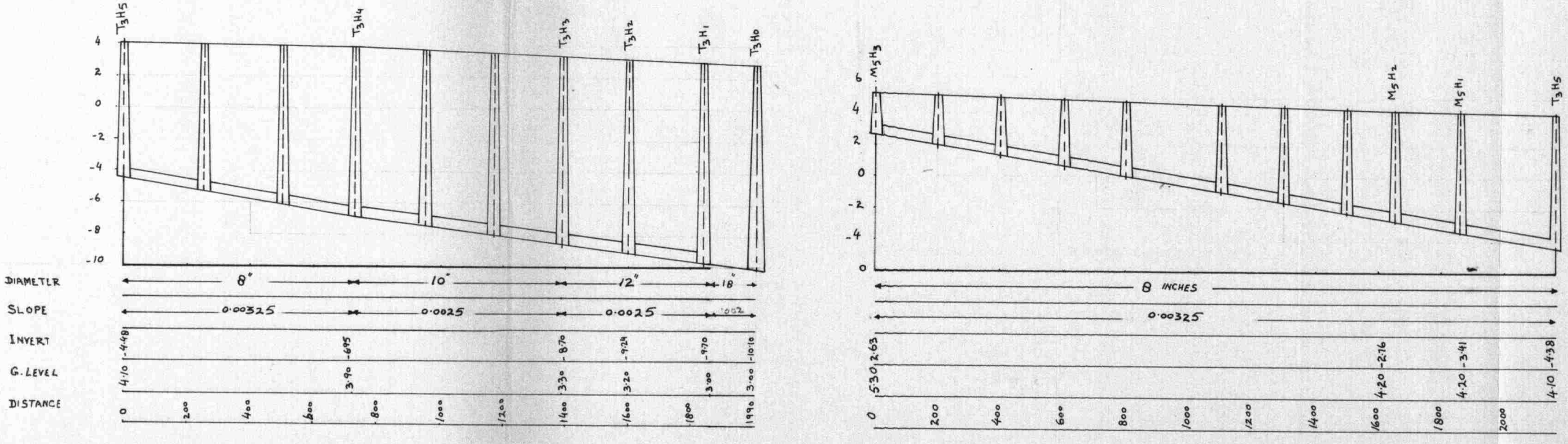
MAIN SEWER M_4



MAIN SEWER M_3

NOTE: GROUND ELEVATIONS OBTAINED FROM CONTOURS

SEWERAGE SYSTEM SAIHAT, SAUDI ARABIA	
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION II	
ENGINEER SYED MAZHAR ABBAS	
DRAWING NO: 7	SCALE: HOR: 250'=1" VER: 5'=1"
DATE FEBRUARY 1965	

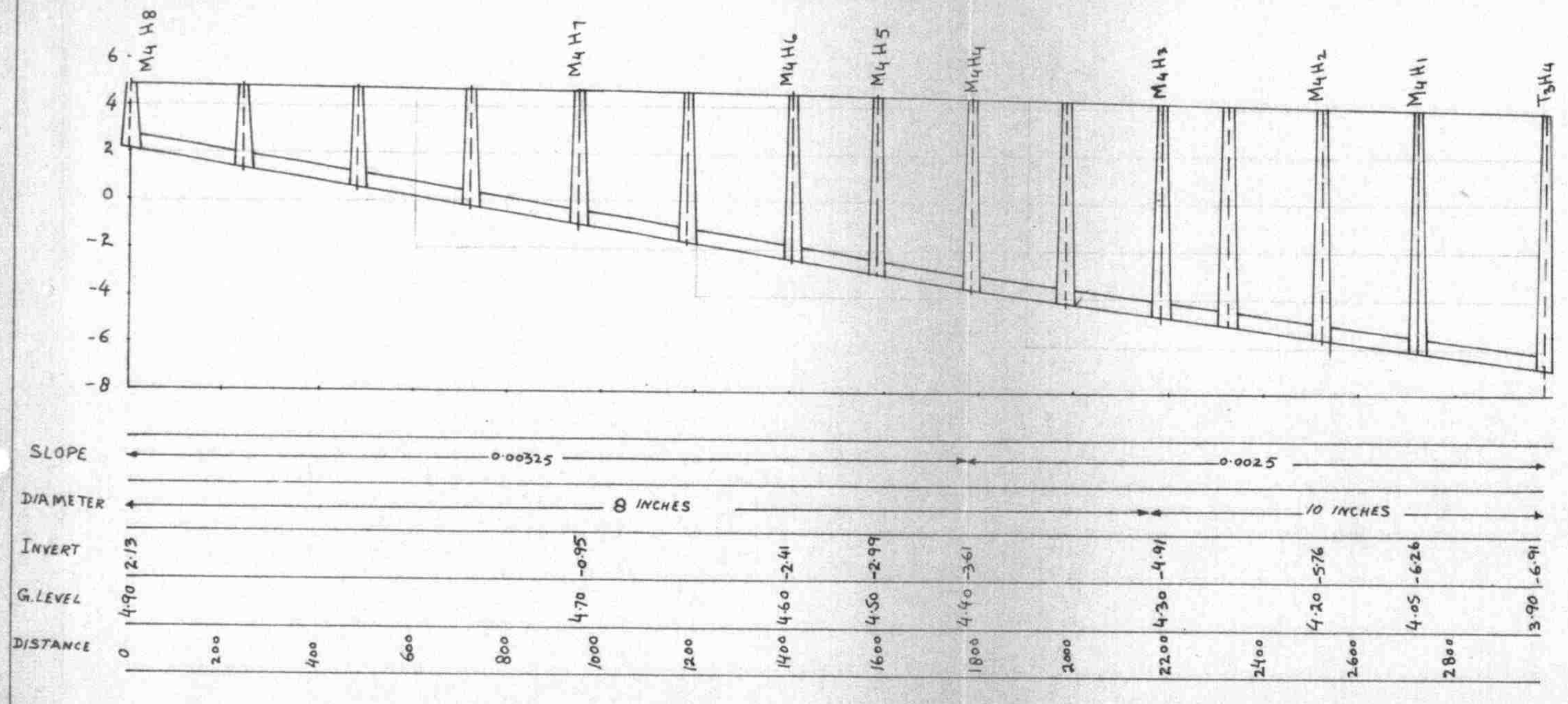


TRUNK SEWER T3

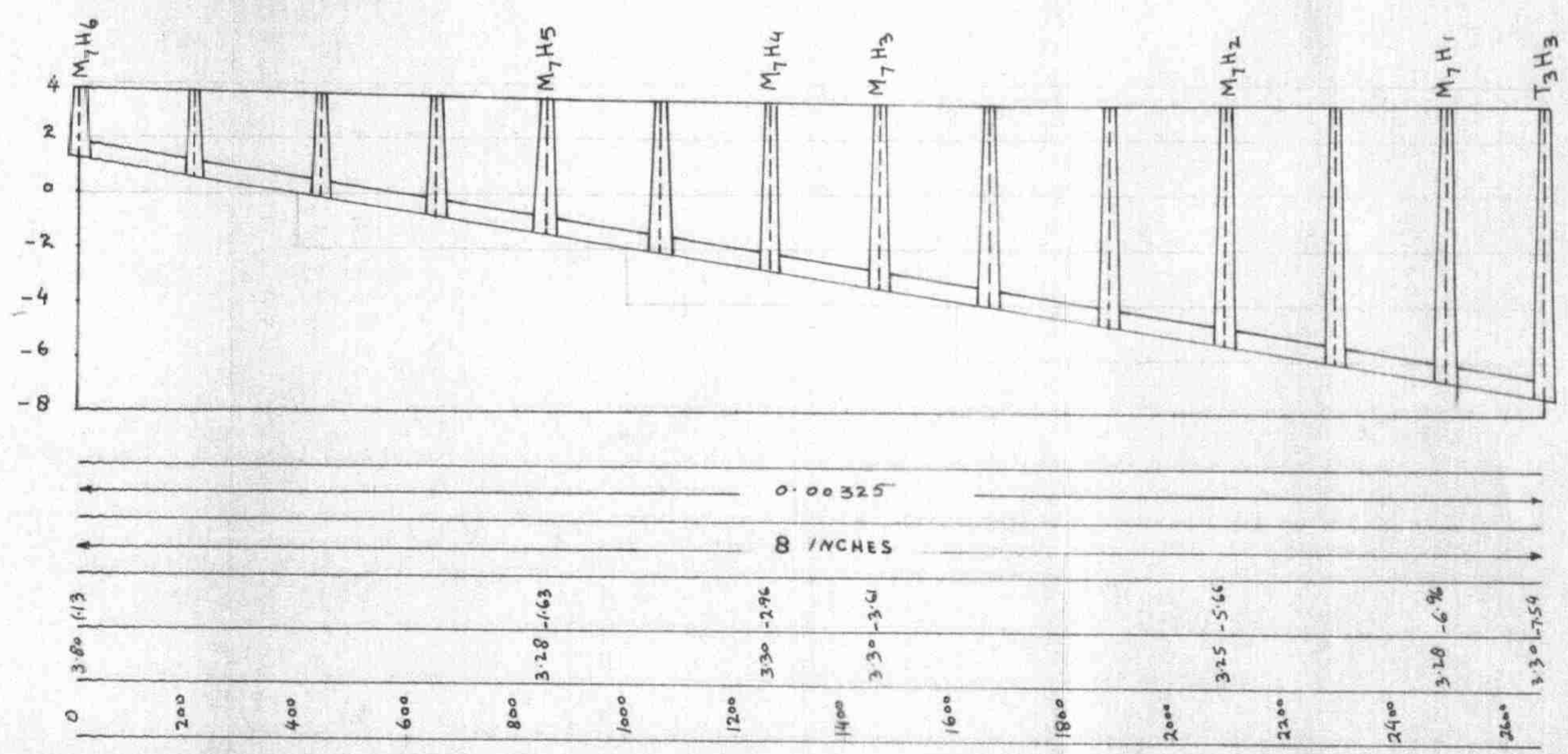
MAIN SEWER M5

NOTE: GROUND ELEVATIONS OBTATAINED FROM CONTOURS

SEWERAGE SYSTEM SAIHAT SAUDI ARABIA	
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION III	
ENGINEER SYED MAZHAR ABBAS	
DRAWING NO 8	SCALE: HOR: 250' = 1" VER: 5' = 1"
DATE FEBRUARY 1965	



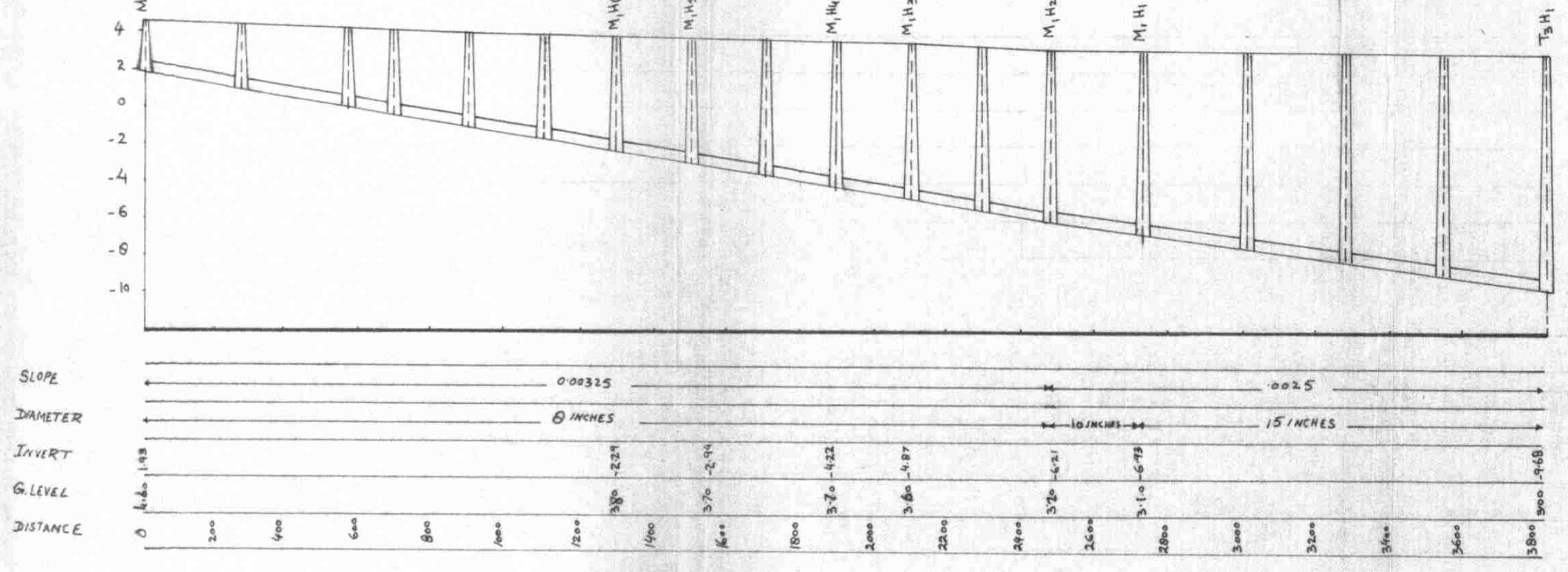
MAIN SEWER M4



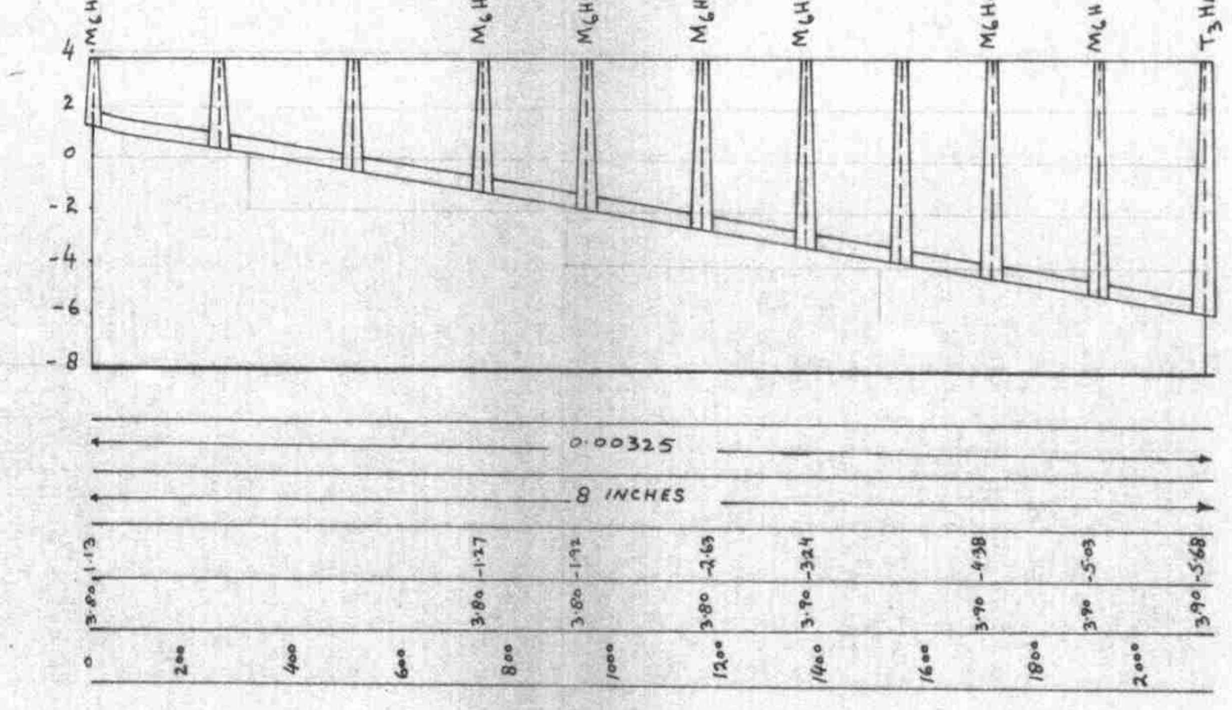
MAIN SEWER M7

NOTE: GROUND ELEVATIONS OBTAINED FROM CONTOURS

SEWERAGE SYSTEM SAIHAT, SAUDI ARABIA	
LONGITUDINAL SECTION OF SEWERS FEEDING PUMPING STATION III	
ENGINEER SYED MAZHAR ABBAS	
DRAWING NO: 9	SCALE: HOR: 250' = 1" VER: 5' = 1"
DATE: FEBRUARY 1965	



MAIN SEWER M_1



MAIN SEWER M_6

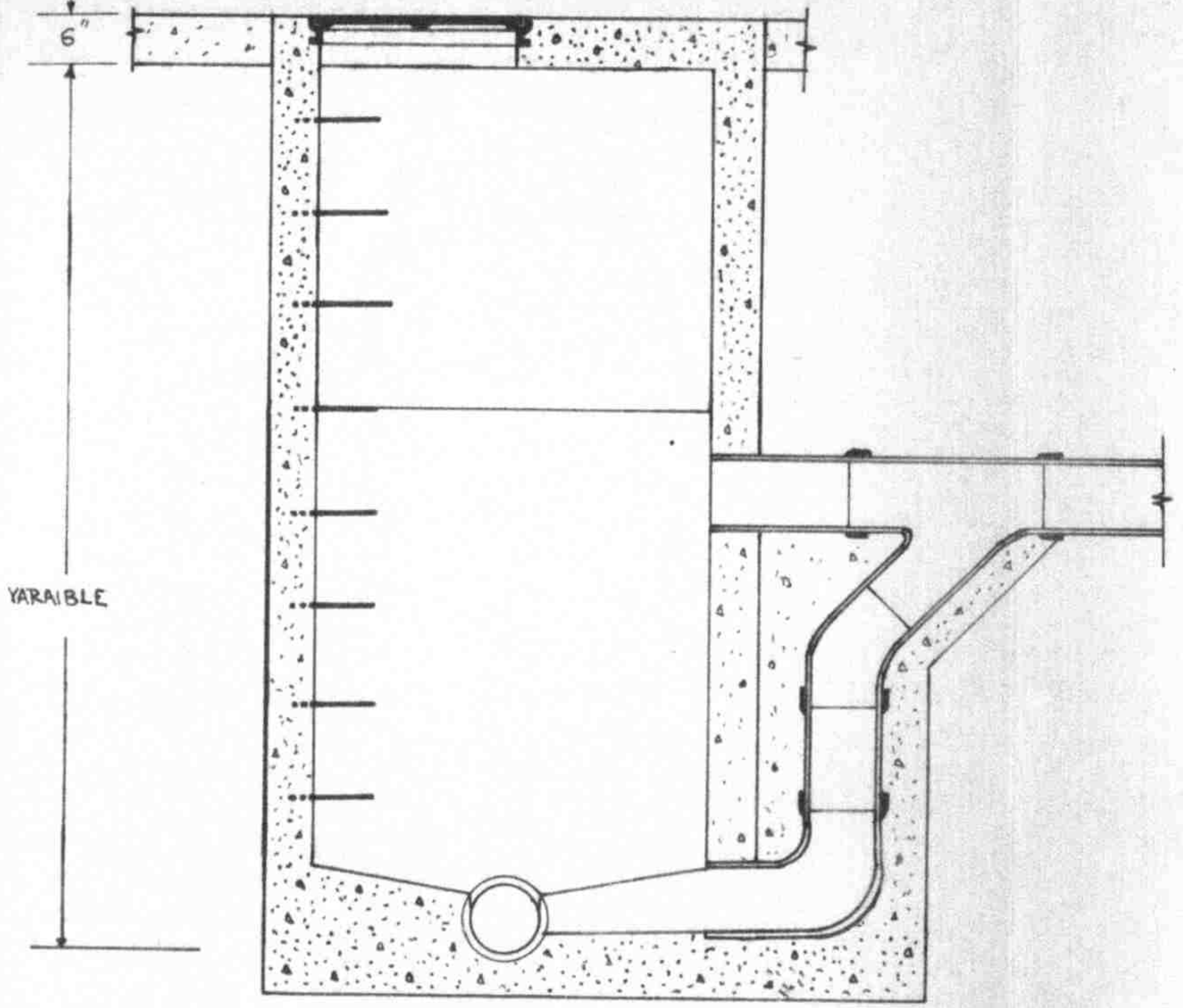
NOTE: GROUND ELEVATIONS FROM CONTOURS.

SEWERAGE SYSTEM SAIHAT, SAUDI ARABIA

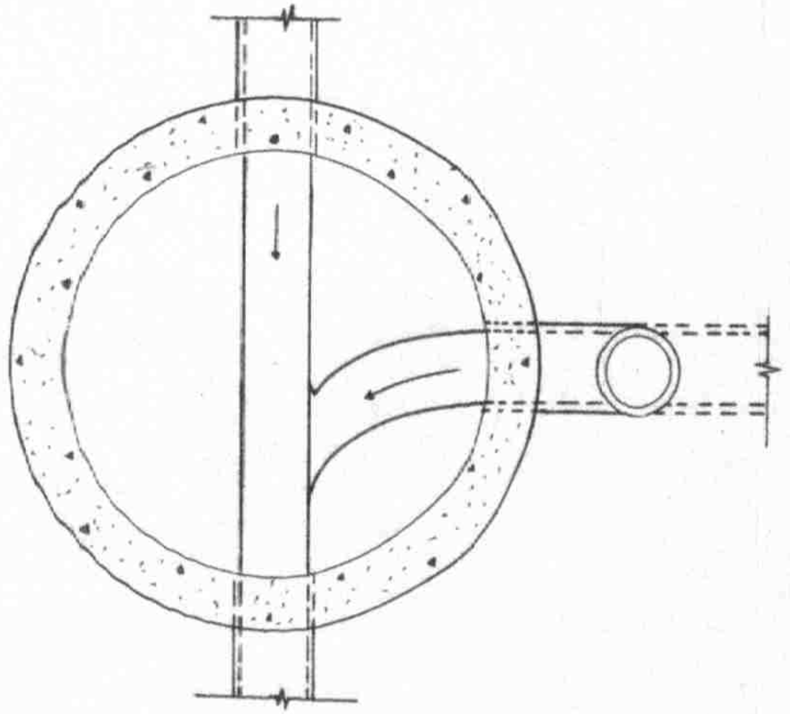
LONGITUDINAL SECTIONS OF SEWERS FEEDING PUMPING STATION III

ENGINEER SYED MAZHAR ABBAS

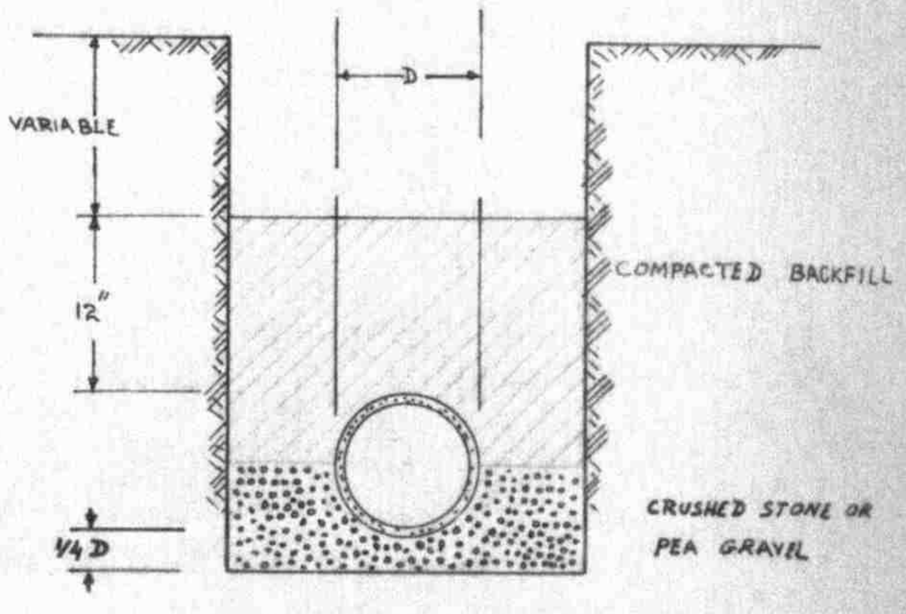
DRAWING NO: 10	SCALE: HOR: 250' = 1" VER: 5' = 1"
DATE: FEBRUARY 1965	



DROP MANHOLE
SECTIONAL ELEVATION
SCALE 1/20



SECTIONAL PLAN

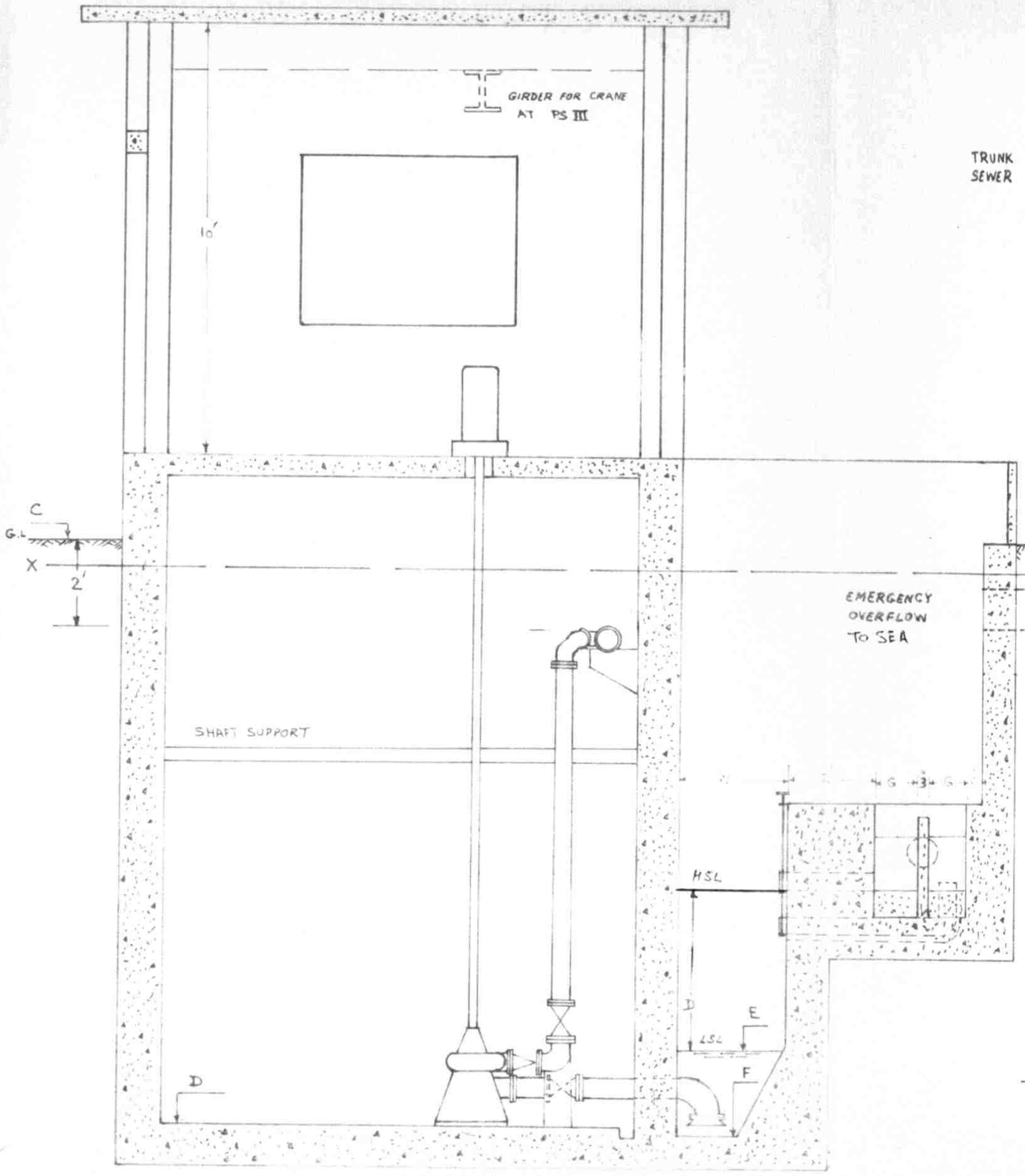


SEWER BEDDING

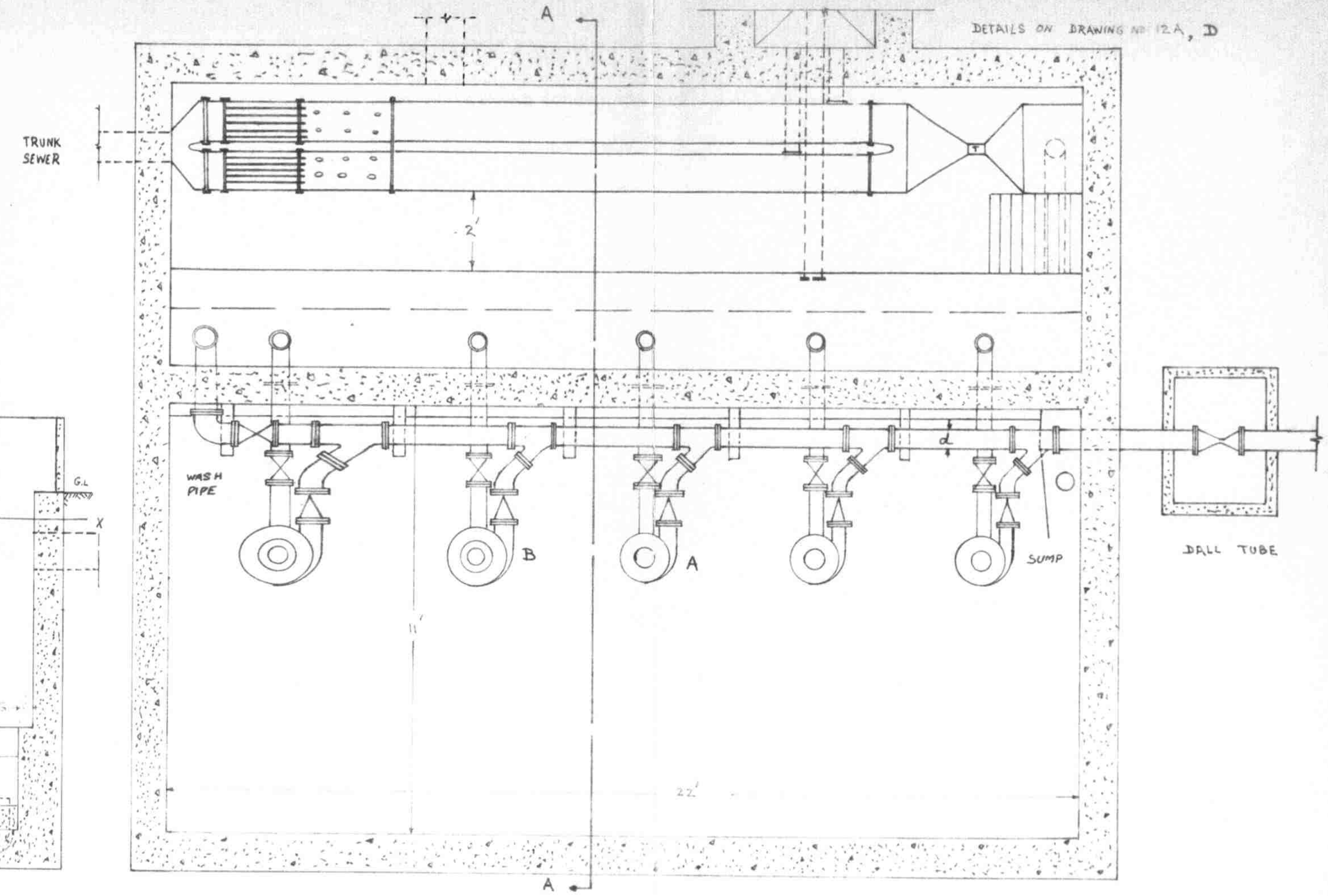
NOTE

SEWER BEDDING AS PER WPCF
MANUAL OF PRACTICE (1960) P. 203.

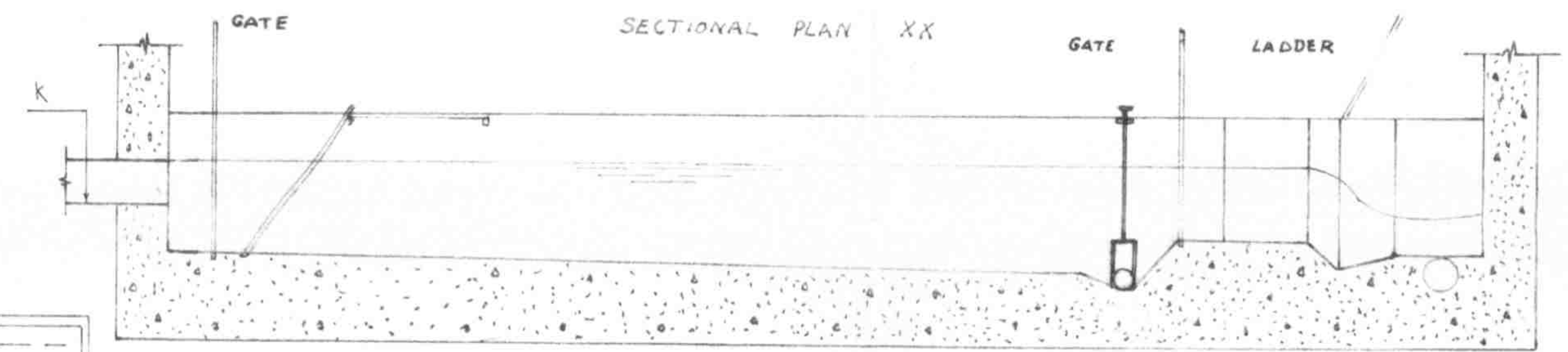
SEWERAGE SYSTEM SAIHAT, SAUDI ARABIA		
DETAILS OF MANHOLE AND SEWER BEDDING		
ENGINEER	SYED MAZHAR ABBAS	
DRAWING No:		11
SCALE	:	AS SHOWN
DATE	:	FEBRUARY, 1965



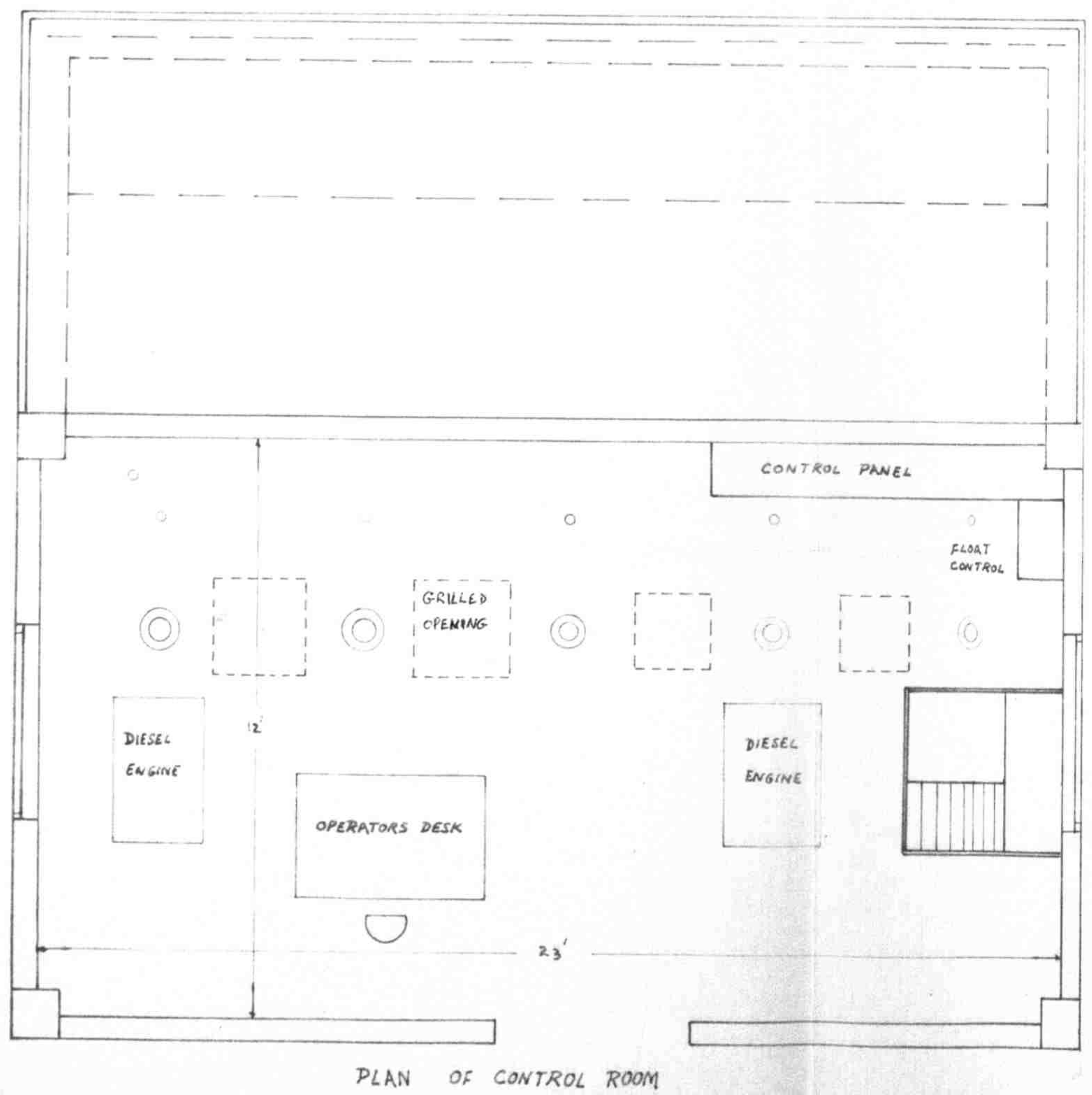
SECTION AT AA



SECTIONAL PLAN XX



SECTION THROUGH GRIT CHANNEL



PLAN OF CONTROL ROOM

NOTE:-

THE DIMENSIONS OF ALL THE THREE PUMPING STATIONS HAVE BEEN KEPT THE SAME. THE DIFFERENCE WHEREVER OCCURS IS SHOWN AS UNDER:

SIZE OF	PS I	PS II	PS III
A	P_1	P_3	P_5
B	P_2	P_4	P_6
C	+2.00	+2.60	+3.00
D	−13.25	−12.76	−14.06
E	−11.59	−11.01	−16.60
F	−13.59	−13.01	−16.35
G	1.00'	1.00'	1.75'
K	−8.09	−7.01	−10.10
T	3"	6"	9"
d	6"	10"	14"
W	2.5'	3.0'	3.5'

SEWERAGE SYSTEM, SAIHAT
SAUDI ARABIA

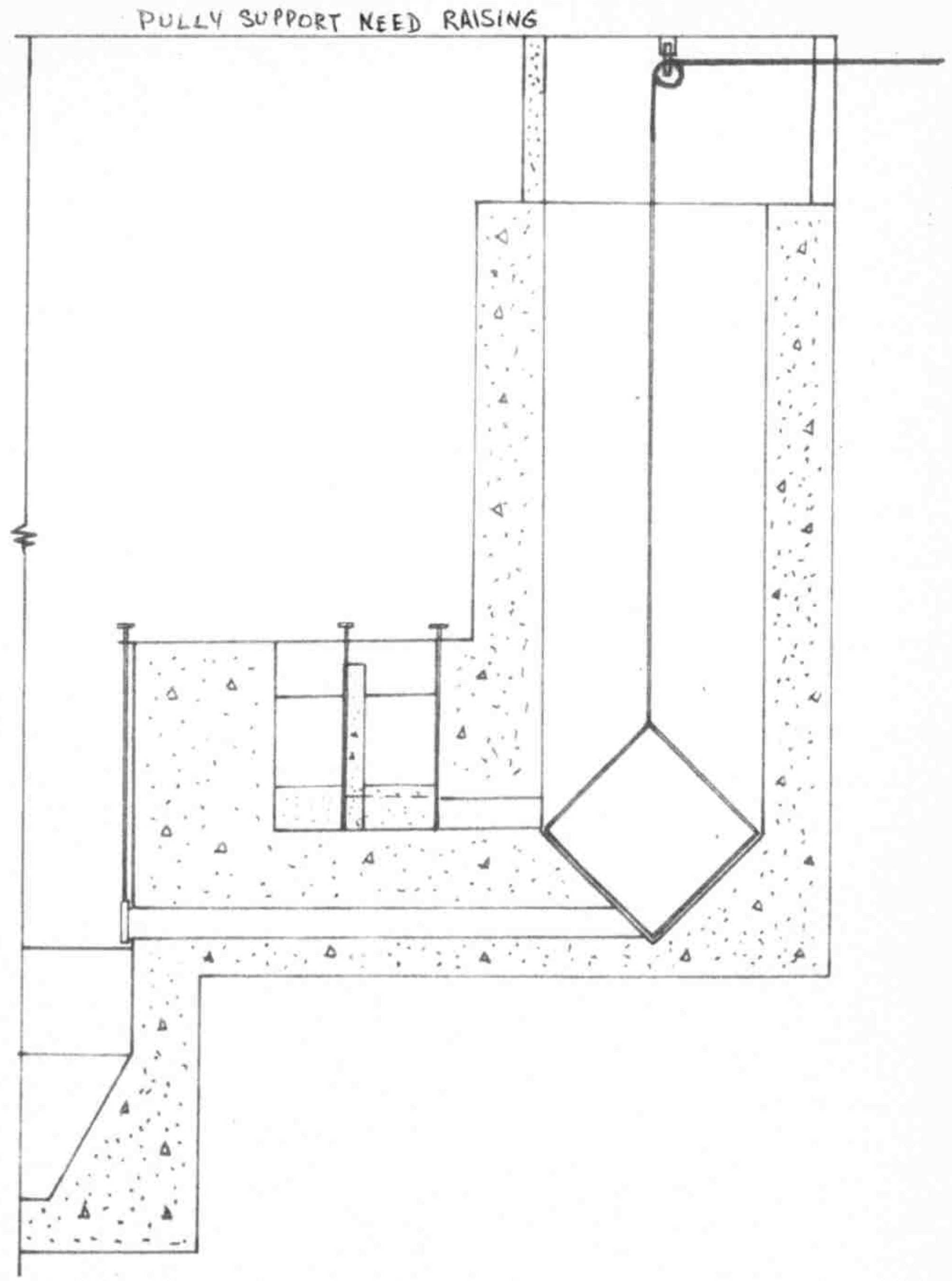
PUMPING STATIONS

ENGINEER SYED MAZHAR ABBAS

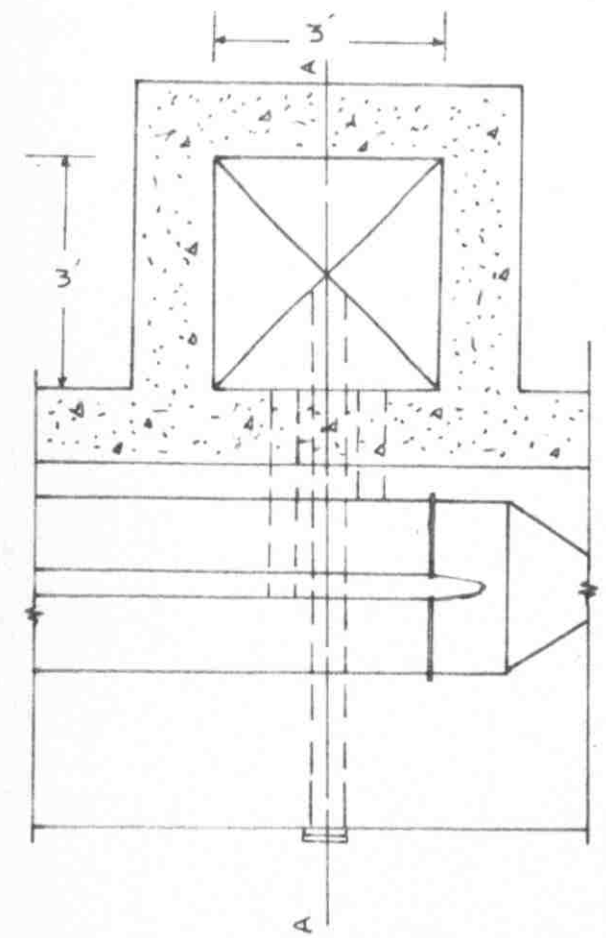
DRAWING NO: 12

SCALE : 1/30

DATE FEBRUARY 1965

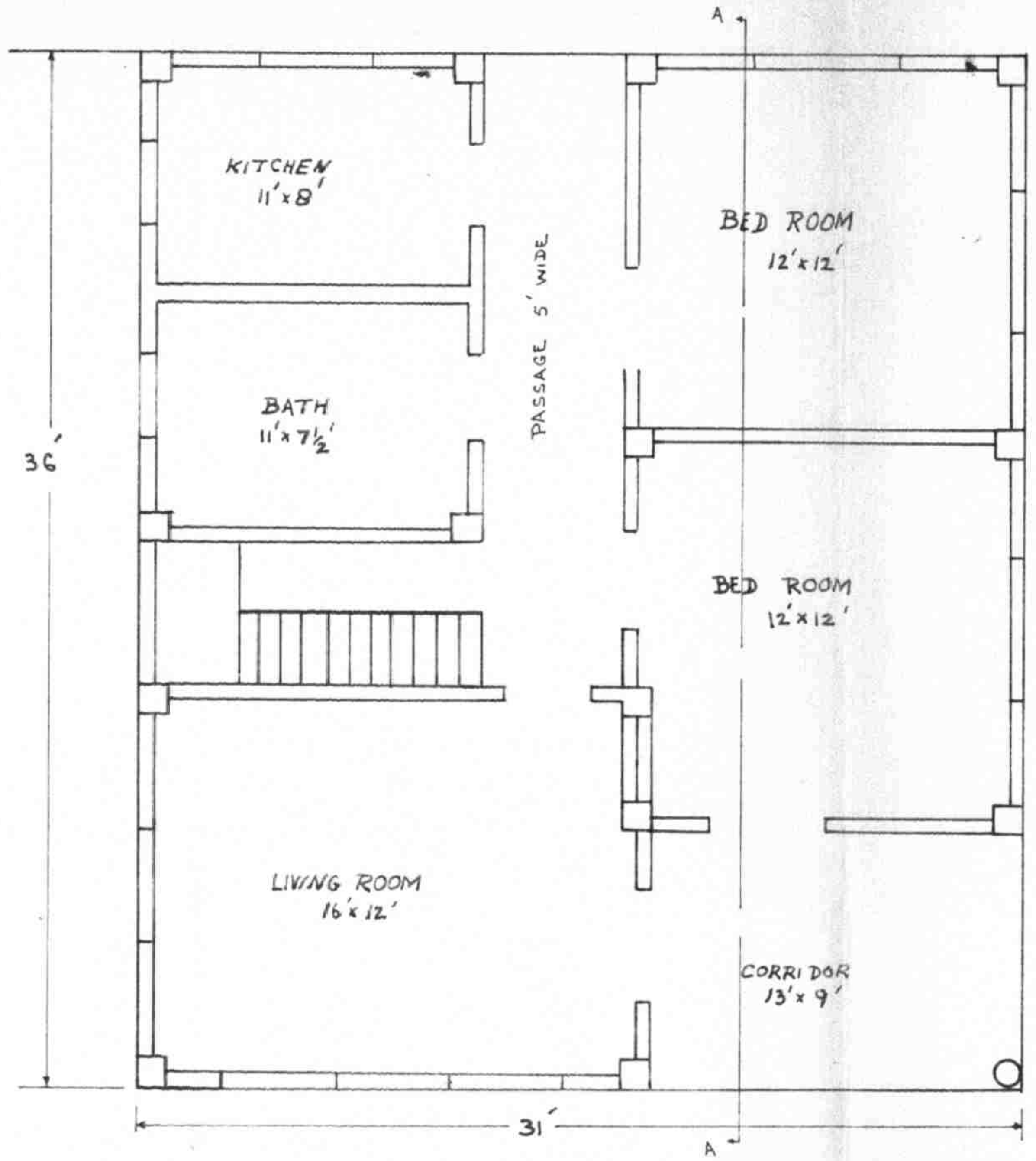


SECTION AA

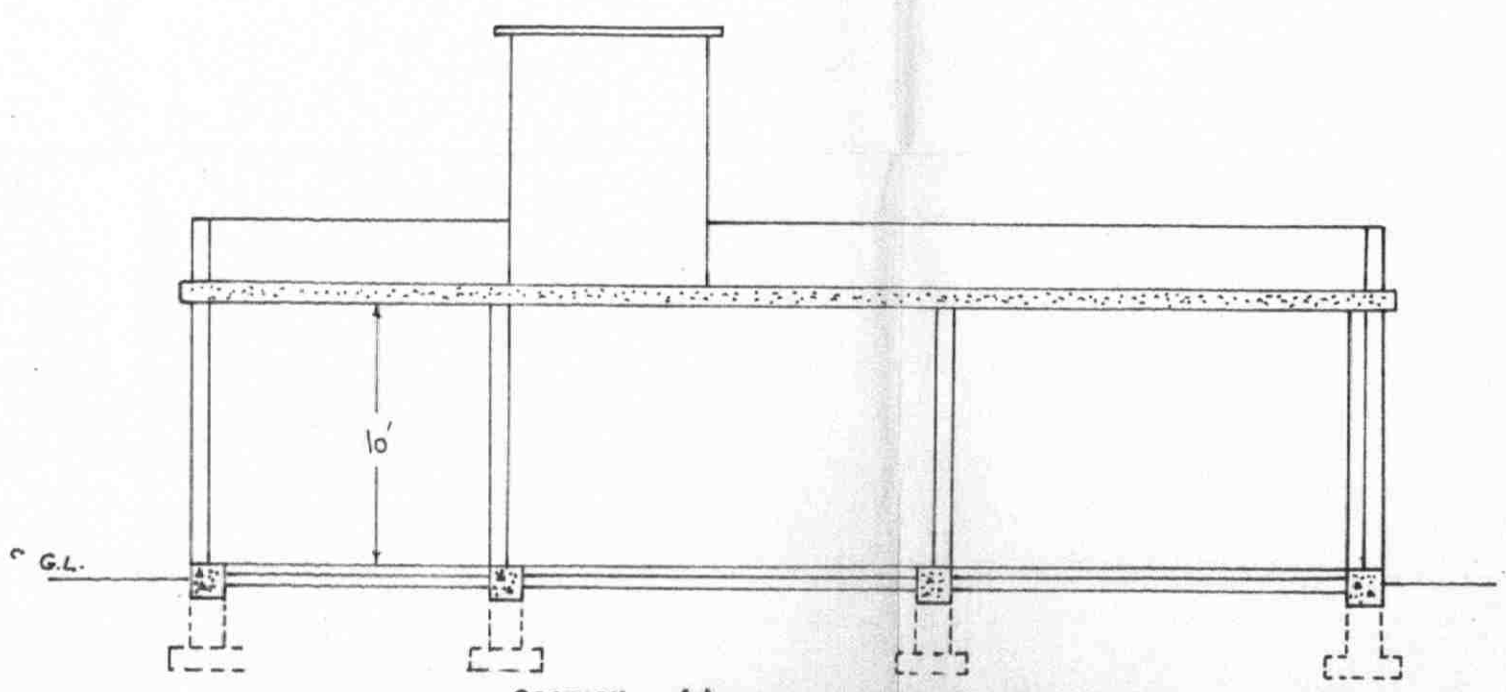


DETAIL D OF DRWG. No: 12

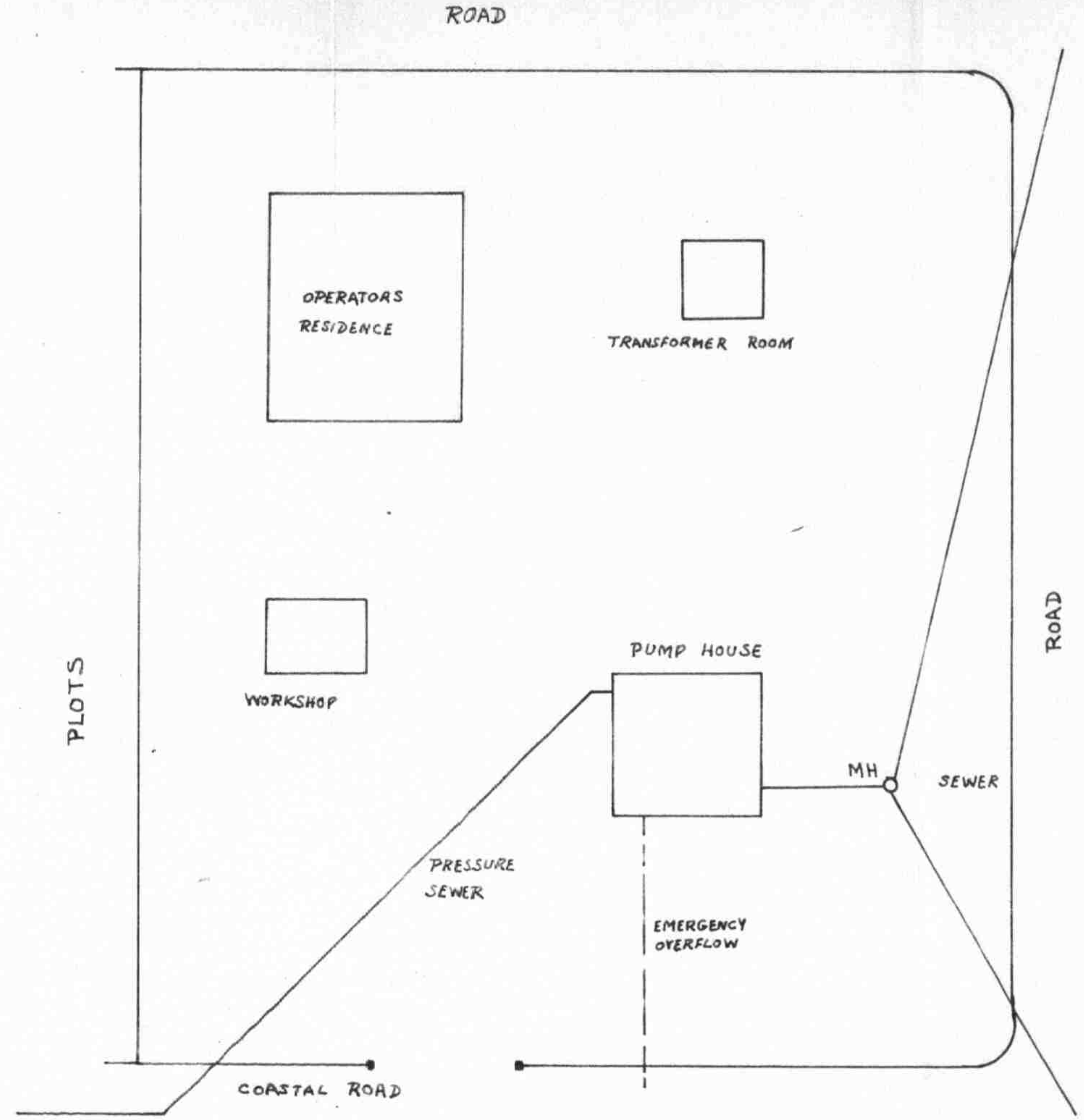
SEWERAGE SYSTEM SAIHAT SAUDI ARABIA
DETAILS OF GRIT REMOVAL SYSTEM
ENGINEER SYED MAZHAR ABBAS
DRAWING NO: 12A SCALE : 1/30 DATE: FEBRUARY 1965

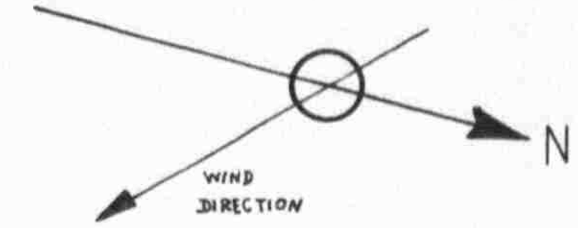


OPERATORS RESIDENC
SCAE: 1/60



SECTION AA

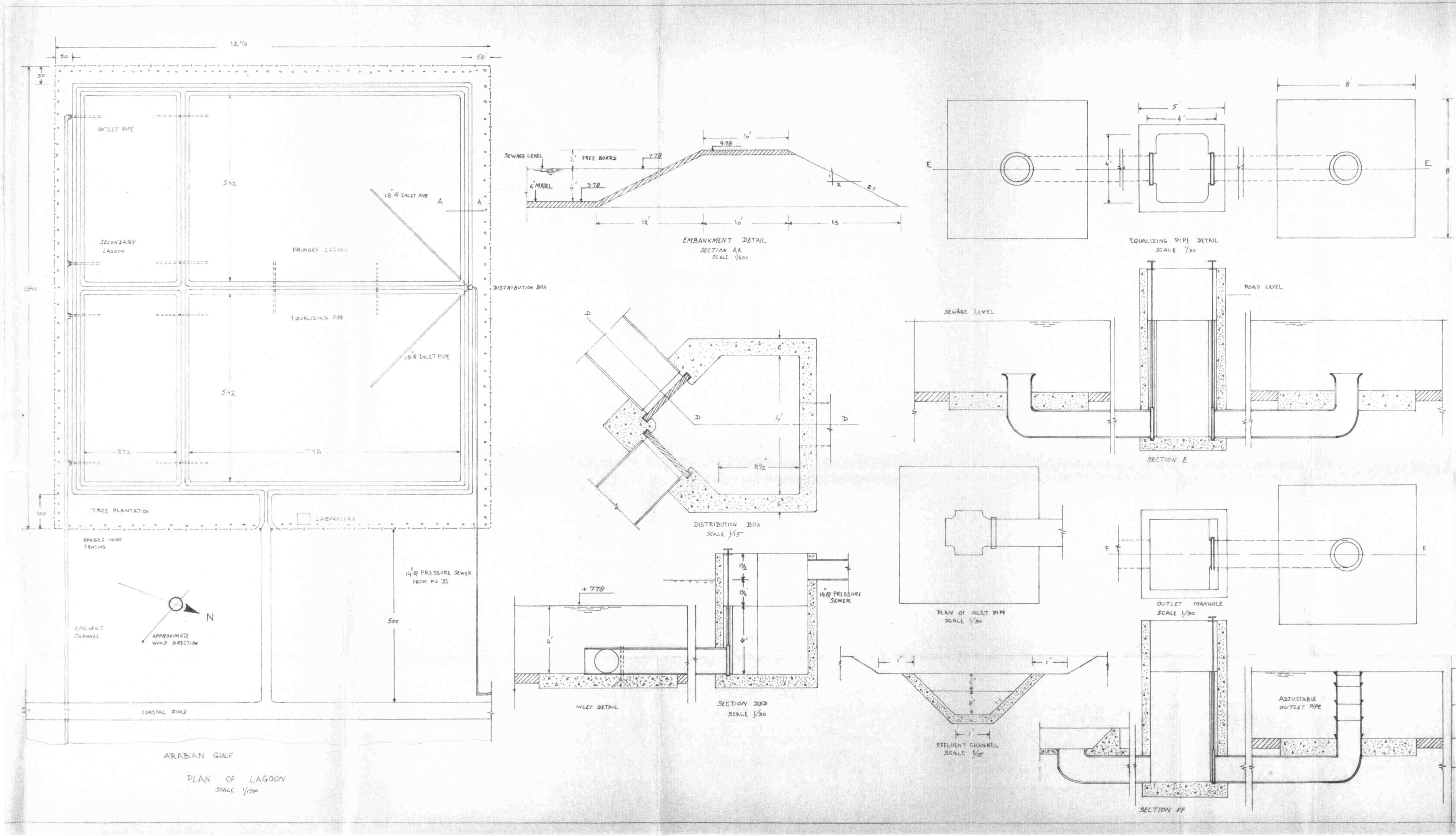




SITE PLAN OF PUMPING STATION
SCALE 1/240

NOTE :
THE SITE PLAN FOR THE THREE PUMPING STATIONS WILL BE SAME EXCEPT FOR THE FOLLOWING CHANGES:
1. TRANSFORMER ROOM PROVIDED IN PS II AND PS III.
2. WORKSHOP PROVIDED IN PS III ONLY.
3. FOR PS II AND PS III THE PUMP HOUSE WILL BE PROVIDED IN WESTERN PORTION WITH WORKSHOP WHILE OPERATORS RESIDENCE AND TRANSFORMER ROOM IN THE EASTERN PORTION OF THE AREA.

SEWERAGE SYSTEM SAIHAT SAUDI ARABIA	
SITE PLAN OF PUMPING STATIONS AND OPERATORS RESIDENCE	
ENGINER SYED MAZHAR ABBAS	
DRAWING NO: 13 DATE FEBRUARY 1965	SCALE: AS SHOWN

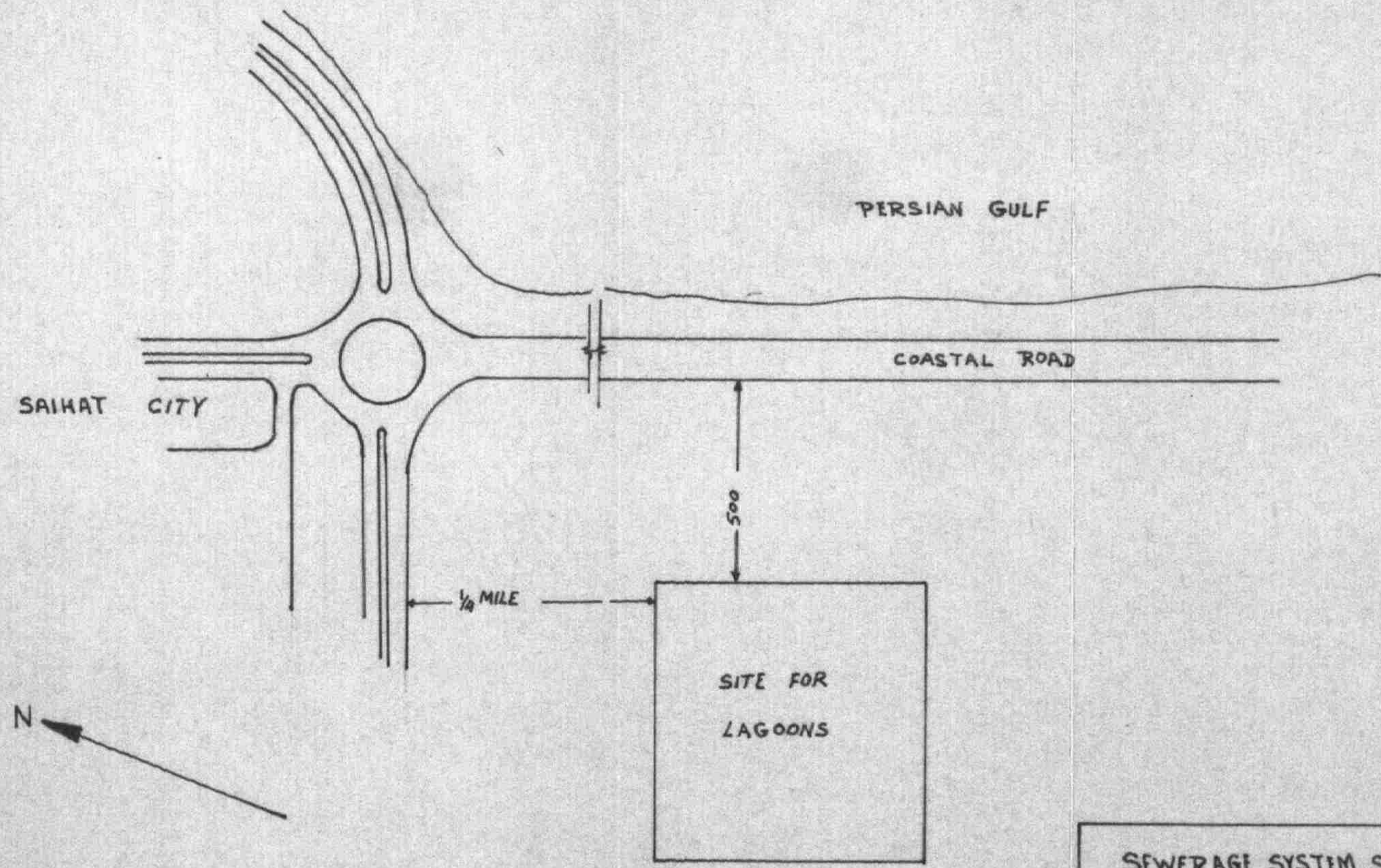


SEWERAGE SYSTEM, SAIHAT, SAUDI ARABIA

PLAN OF LAGOONS AND DETAILS

ENGINER SYED MAZHAR ABBAS

DRAWING NO	14
SCALE	AS SHOWN
DATE	FEBRUARY 1965



SEWERAGE SYSTEM SAIHAT SAUDI ARABIA

LOCATION PLAN FOR LAGOONS

ENGINEER SYED MAZHAR ABBAS

DRAWING NO : 15
SCALE : NOT TO SCALE
DATE : FEBRUARY 1965

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Unpuhblished Material

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ADDENDUM

This project report was submitted before the defense committee on 22nd February 1965. The committee accepted the project for the degree of Master of Engineering with major in Sanitary Engineering, with minor corrections which have been made. However, the committee criticised the project as presented with regard to the following points which, in their, opinion, require further study and modifications :-

(1) Drawings No. 12 and 13. Better arrangements in drawings in accordance with general practice and for convenience of reading the plan.

(2) Drawing No. 12. Changing the position of control panel to position near door and operator for convenience.

(3) More complete study of disposal of sewage by dilution method to better justify the selection made.

(4) Design of piping system in the lagoon for economy in construction and maintenance.

(5) Water hammer in pressure sewer for checking the strength of pipe and adopting a suitable class of pipe and its size.

The above points were not required to be redone but for the sake of any one who may read this project, the committee directed that the above points be included in the project as an addendum.