PRELIMINARY DESIGN
OF
WATER SUPPLY & SEWERAGE SYSTEMS
FOR
KHAIRPUR MIR'S W. PAKISTAN

By

Shaikh, Allah Bux

Beirut,
June, 1967

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A Thesis

Submitted to the School of Engineering
American University of Beirut in partial
fulfilment of requirements for the degree
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Advisor:

Prof. G.M. Ayoub

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June, 1967.

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CHAPTER I

INTRODUCTION

1.1. Scope of the work

The purpose of this report is to deal with the design aspects of the water supply and sewerage systems for Khairpur Mirs W. Pakistan.

The two subjects will be dealt with consecutively or separately in the following chapters as deemed fit.

With regards to the water supply system, it is to say that there is an existing water supply system which being inadequate is being supplemented by a new proposed scheme, execution of which is in progress and being carried out by W. Pakistan P.H.E.D. So far the distribution system has been completed and now the final selection of source is to be tackled.

The aim of this report will therefore be to put forth certain proposals concerning this selection and to design the treatment process suggested. The design of the distribution system etc. will not be dealt with in this report.

Regarding the sewerage system, it is to say that this scheme is yet not dealt with by the authorities and hence this report will include a comprehensive study of the subject.

A typical layout for the system will be included, however, a detailed design for one trunk sewer will be carried out.

A discussion on the different types of possible treatment processes will be included while a detailed discussion on the most suitable one will be presented.

1.2. Location

Khairpur town is situated on the main railway line between Lahore and Karachi and is about 280 miles North of Karachi. It is also about 16 miles south of Rohri and Sukkur and lies on the main national highway running between Karachi - Peshawar.

Its latitude is 27° - 35' North and longitude 68° - 48' East.[1] Fig. 1.1. shows the geographical location of Khairpur.

1.3. History

Khairpur town was the capital of Ex-Khairpur State in the former province of Sind.

Khairpur state, of which His Highness Mir Ali Murad Khan Talpur was ruler, forms a narrow strip of country, the western end of which is bordered by River Indus. This state was created soon after the conquest of Sind by the British in year 1849 and was later on merged in one

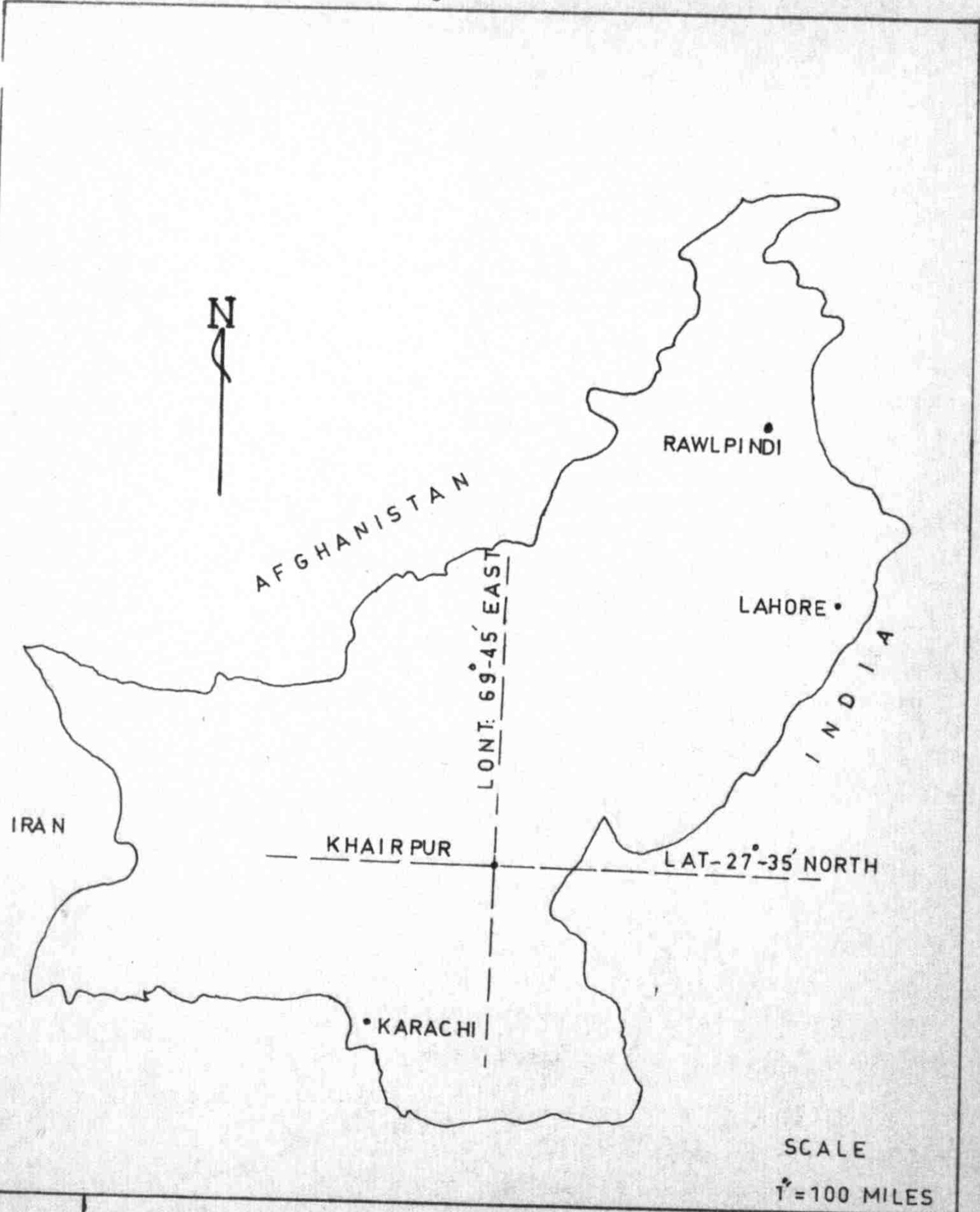

FIG. 1·1

LOCATION MAP

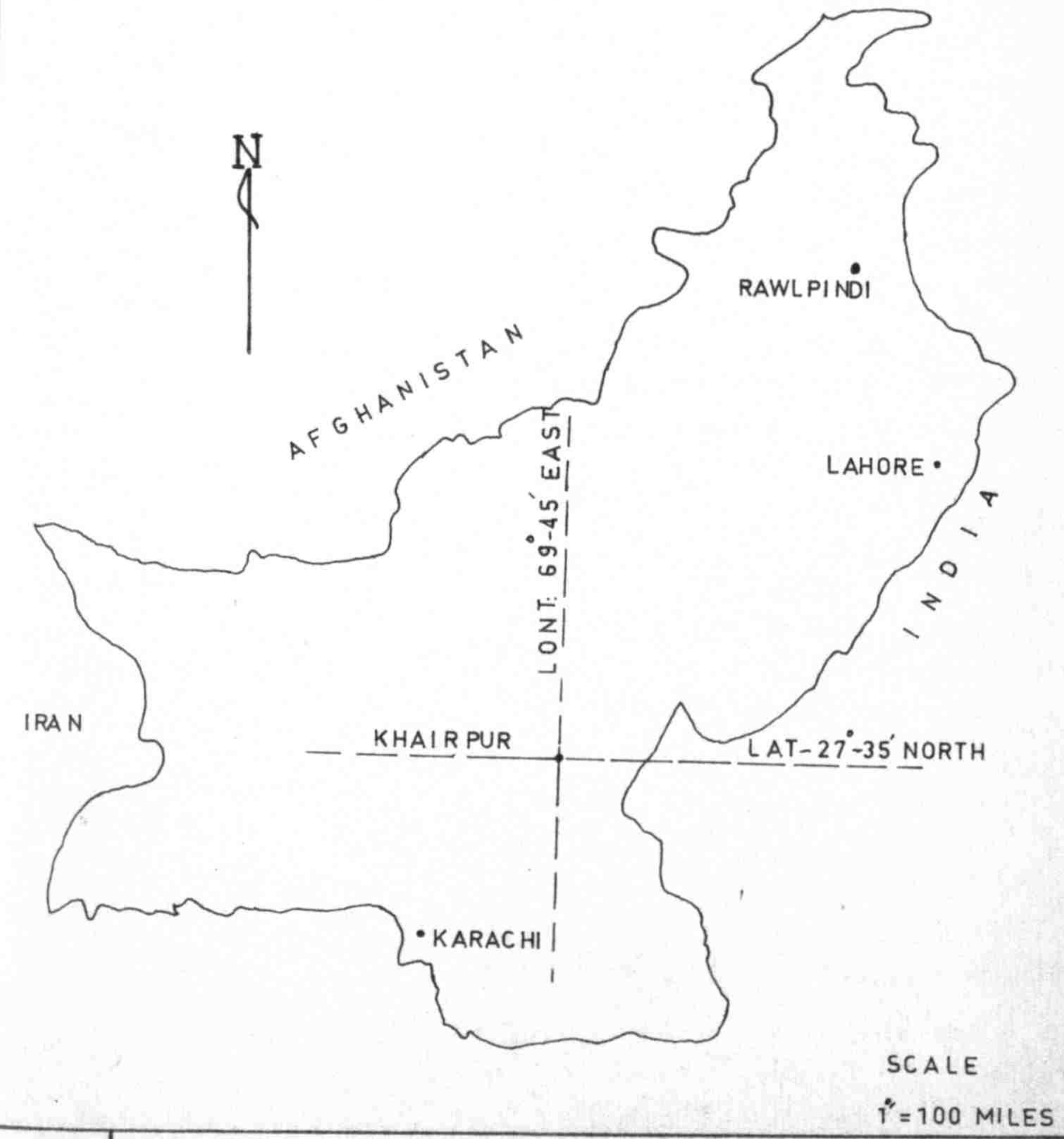

FIG.1·1 LOCATION MAP

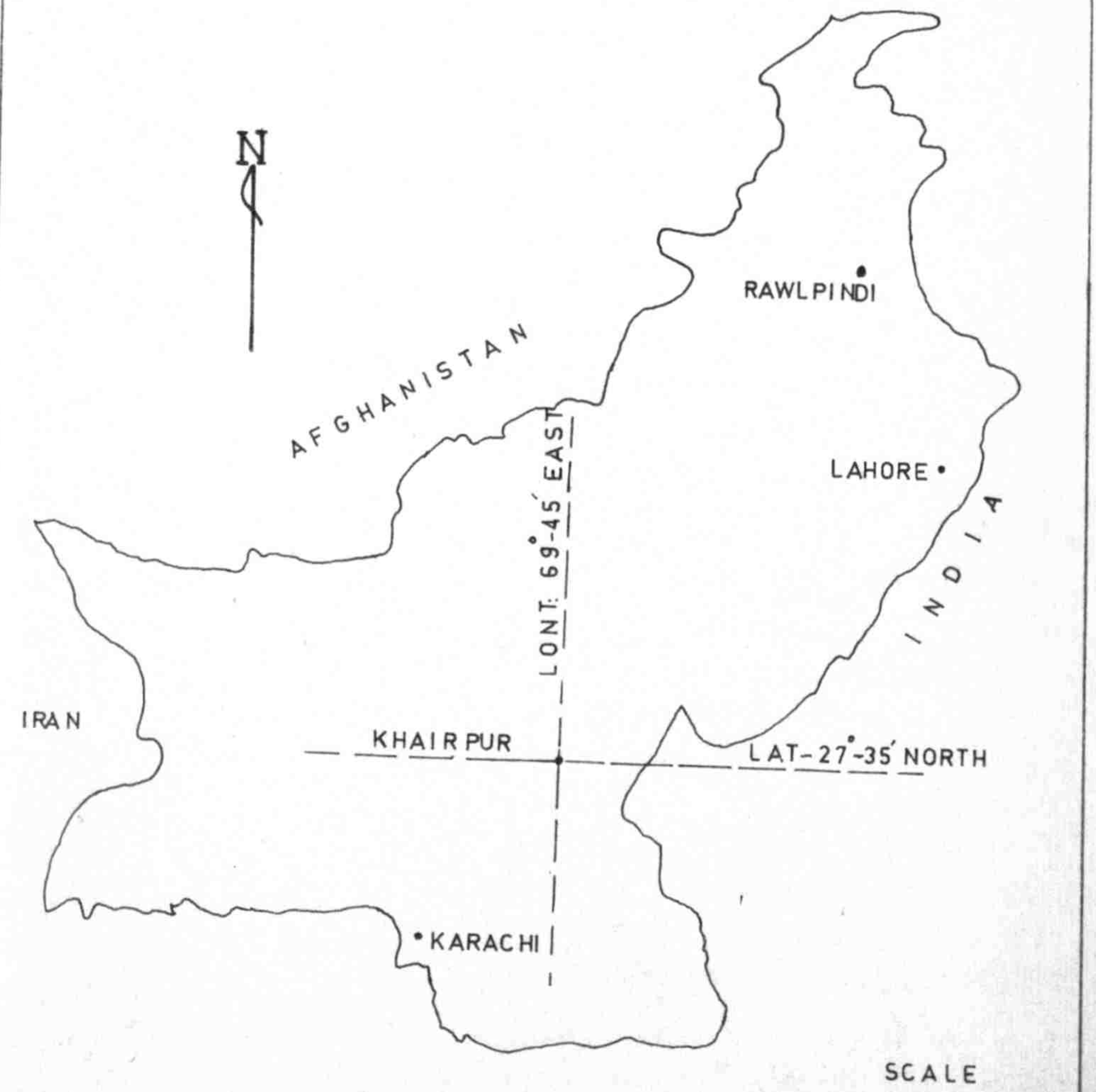

FIG.1·1 LOCATION MAP

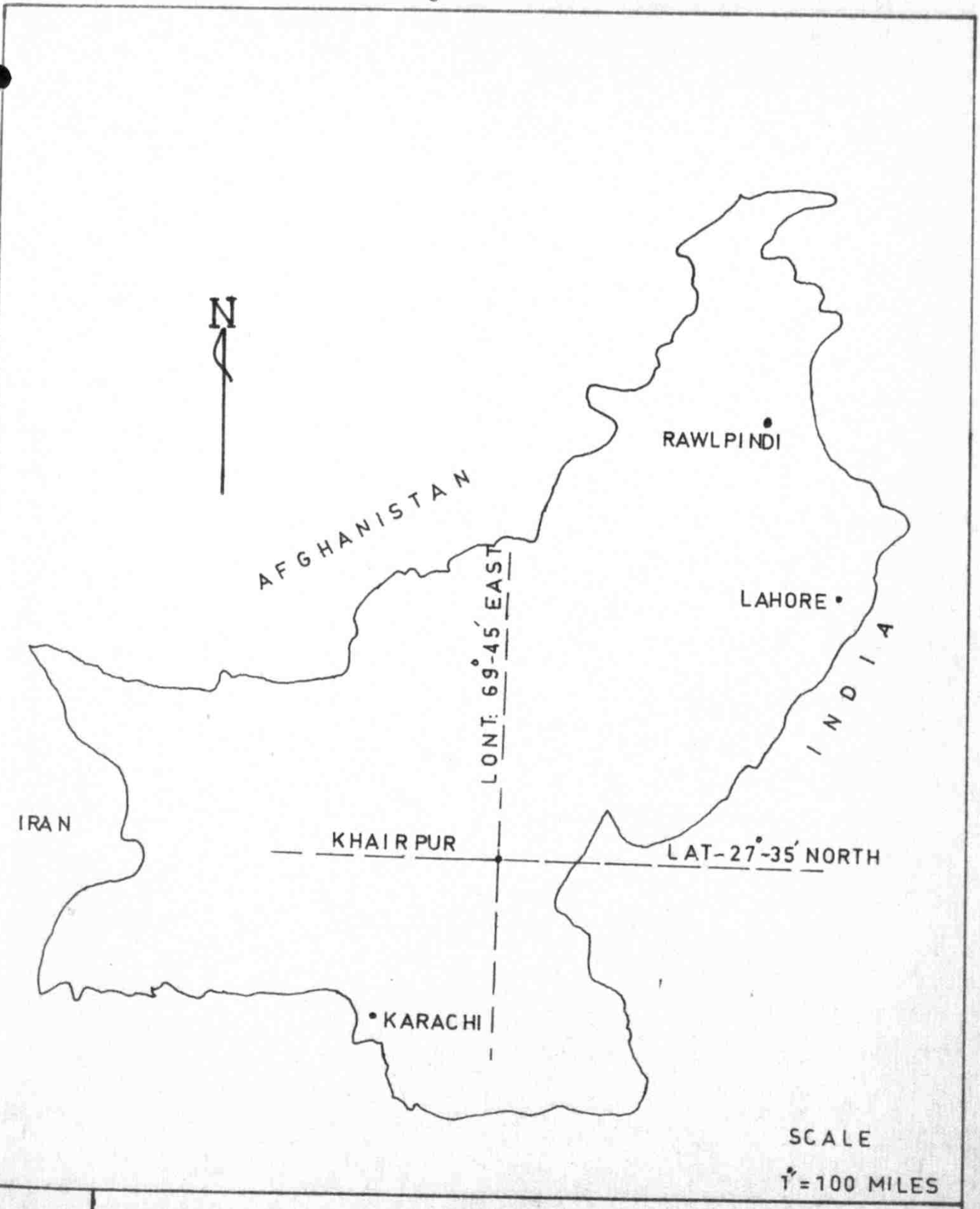

FIG.1.1 LOCATION MAP

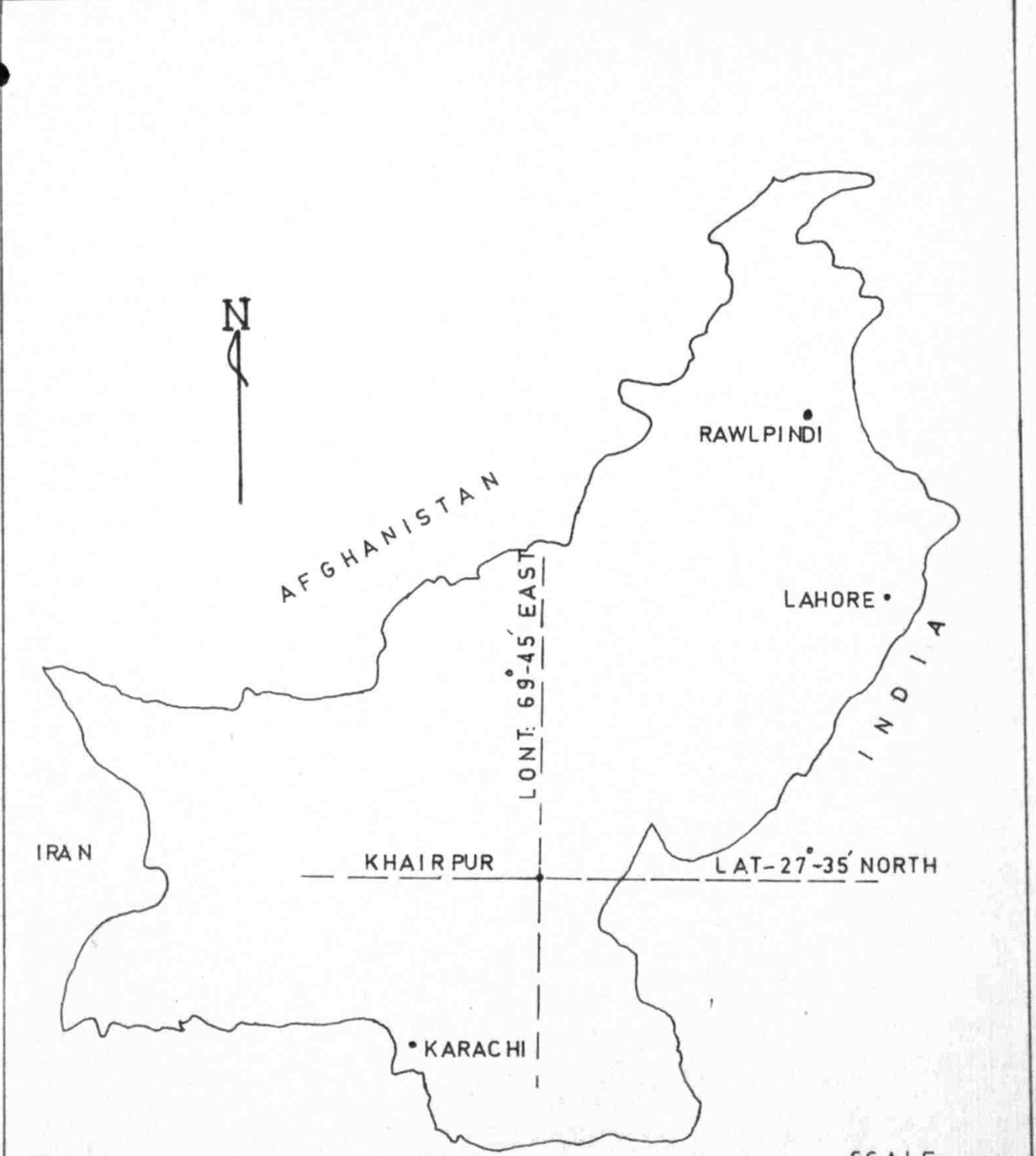

FIG.1·1 LOCATION MAP

unit in 1955.

After the merging of the various provinces and states of West Pakistan into one unit, Khairpur has been formed as the headquarters of Divisional Commissioner - Khairpur Division and thus has started gaining importance.

There has been considerable industrial development in the town. Textile silk and rayon industries being particularly important. It has also got a number of cottage industries and is famous for the manufacture of leather goods.

Recently the West Pakistan Industrial Development Corporation, W.P.I.D.C. has established a small industrial estate area in the town to encourage small industries.

The soil of this area is fertile and produces mainly wheat, rice, cotton, etc. Date palms are also grown in large numbers and are famous of its kind.

It has a number of institutions, public utility buildings such as: colleges, schools, hospitals, tourist resthouses, gardens and playgrounds.

1.4. Communications

Communications are maintained by highways and a railway line. The Karachi-Peshwar main railway line passes through the city.

The main national highway passes through the town which on the northern end connects it with nearby

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towns such as Sukkur Rohri - Pano Akil etc. and on the southern end with Ranipur, Gambat, Kotdigi, etc. These fall on the main road or are connected through branch roads to this main road. Communications of Khairpur with its nearby towns is shown in Fig. 1.2 and with the whole of west Pakistan is shown in Fig. 1.3.

1.5. Living Quaters

The houses in Khairpur are mostly single storeyed and brick built. It consists mainly of unplanned building blocks in the center, so called the main town, however properly planned blocks are constructed on Eastern and Western sides of Mir Wah Canal in Mumtaz Coloney and K.T.M. Coloney respectively. The main roads of the town are 30' - 40' wide and are metalled, while interlinking lanes and streets are brick-paved.

1.6. Topography

The town of Khairpur lies on a flat plain. The ground has a very gentle slope from East towards the West. Khairpur east feeder, a cut from river Indus feeds one of the largest canals in the area so called Mir Wah Canal which runs mostly in embankment, nearly through the center of the area. The maximum water elevation exceeds the ground

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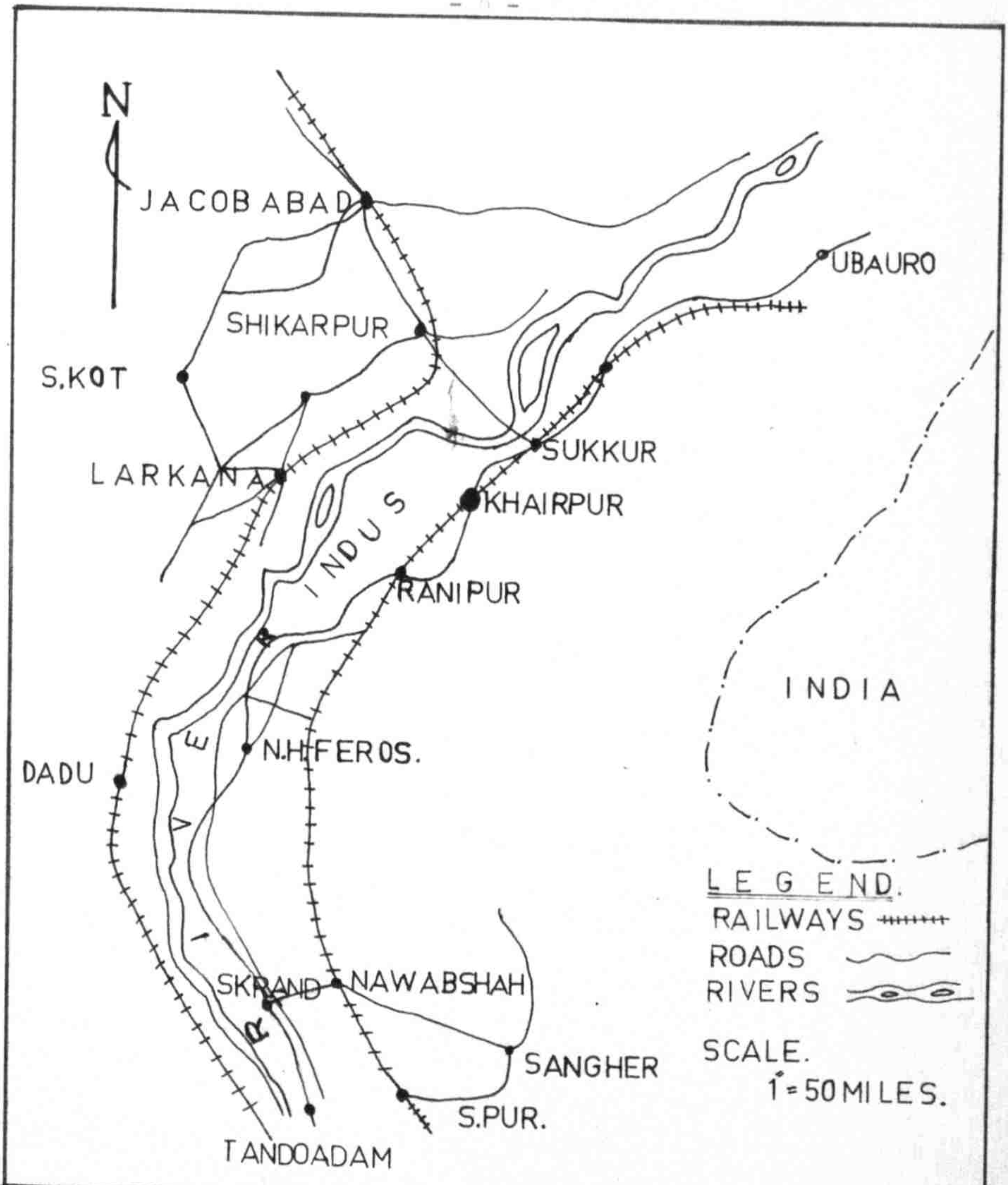

FIG: 1·2 KHAIRPUR COMMUNICATION MAP.

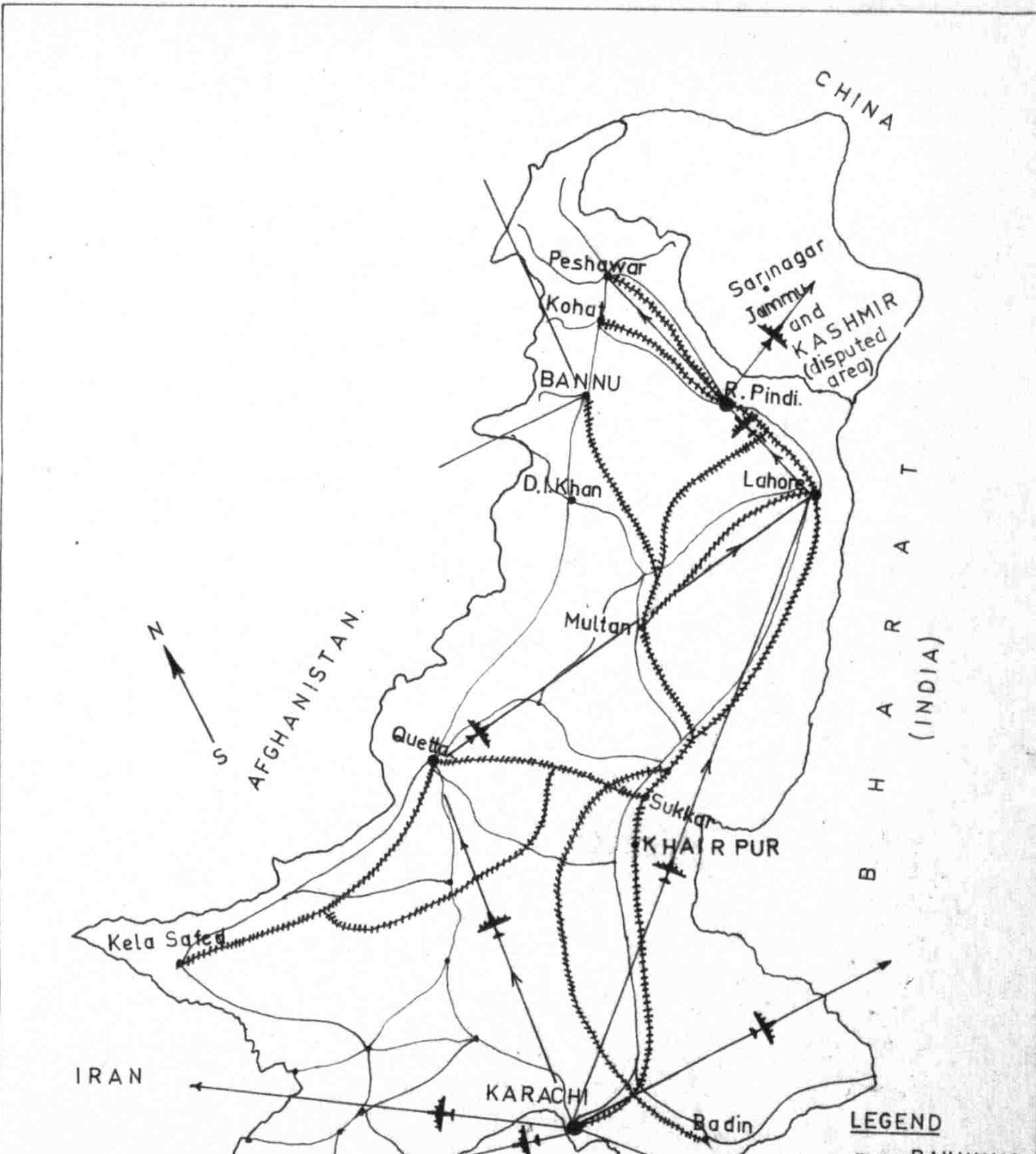

FIG. 1·3 LOCATION AND COMMUNICATION MAP

elevation of most of the inhabited areas of the town.

A contoured map of the town is shown on Drawing No. 1.

1.7. Climatic Conditions.

1.7.1 Temperature

No recent statistics on temperature are available, however an old record of 10 years from 1924-1934 gives the maximum and minimum temperatures recorded during various years. These are given below in Table 1.1.

TABLE 1.1[2]

MAXIMUM AND MINIMUM TEMPERATURES FOR KHAIRPUR (1924-34)

YEAR	MAX.TEMP. °F	MIN.TEMP. °F	YEAR	MAX.TEMP. °F	MIN.TEMP. °F
1924-25	120	38	1929-30	118	46
1925-26	119	43	1930-31	110	47
1926-27	117	37	1931-32	119	51
1927-28	117	38	1932-33	117	50
1928-29	119	45	1933-34	115	45

The maximum temperature is not any of the means, but the actual highest recorded in one particular year, so also the minimum.

In a report for Sukkur, a city 14 miles north of Khairpur and with similar climatic conditions, a statement on the subject reads as under :

"The maximum summer temperature goes as high as 118°F to 120°F. The winter temperature rarely falls below 40°F. However, the day time temperature seldom falls below 70°F even in winter".(3)

The two data given above show a closer similarity.

1.7.2 Rainfall

Rainfall in the area is very scanty and unreliable. The greater portion of it is received during the months of July and August, while there are a few winter showers re-received in December, January and February. Very rarely the early summer showers are received in May also.

From a 12 years record as given in Table 1.2 and shown in Fig. 1.4 average annual temperature is found to be about 1.3 inches with a minimum of 0.5 inches and a maximum of 3.0.

TABLE 1.2(1)

ANNUAL RAINFALL AT KHAIRPUR 1948-59

Year	Rainfall (inches)	Year	Rainfall (inches)	Year	Rainfall (inches)
1948	2.25	1952	0.5	1956	1.45
1949	1.25	1953	0.9	1957	1.02
1950	2.6	1954	0.75	1958	0.50
1951	0.9	1955	3.0	1959	1.02

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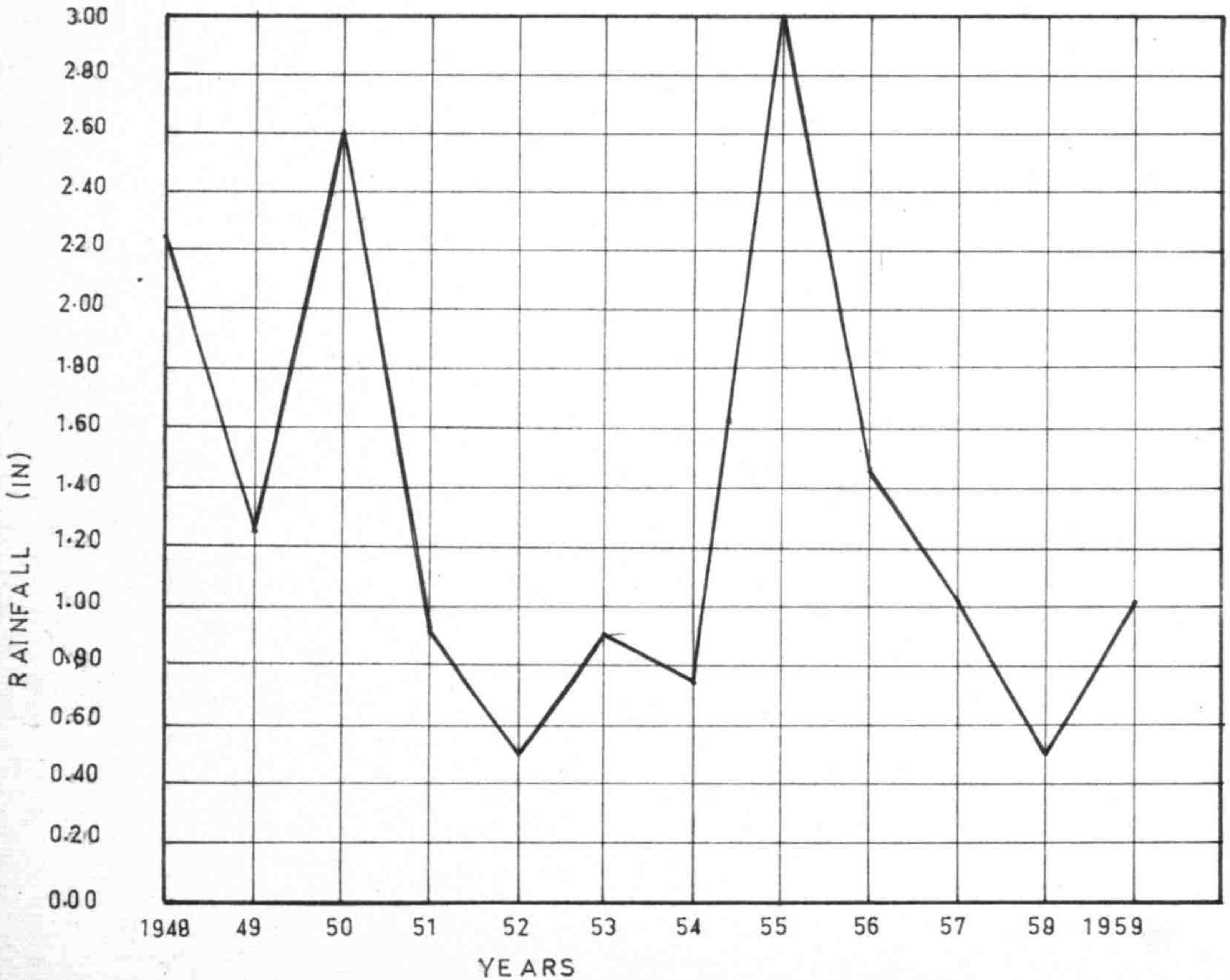

FIG. 1.4 ANNUAL RAINFALL

To get a closer view of annual rainfall distribution a statement of daily rainfall at Sukkur, which, as already said, has similar climatic conditions, for three years from a data of seven years is reproduced in Table No.1.3.

T A B L E 1.3[3]

DAILY RAINFALL AT SUKKUR FOR THE YEAR 1960

Date	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
5th	-	-	-	-	-	-	0.10	-	-	-	-	-
6th	-	-	-	-	-	-	1.15	-	-	-	-	-
10th	-	-	-	-	-	-	2.0	-	-	-	-	-
16th	-	-	0.44	-	-	-	-	-	-	-	-	-
18th	-	-	-	-	-	0.29	-	-	-	-	-	-
27th	-	-	0.63	-	-	-	-	-	-	-	-	-
29th	-	-	-	-	-	-	-	-	-	-	-	0.15
30th	-	-	-	-	-	-	-	-	-	-	-	0.22
Total	-	-	1.07	-	-	0.29	3.25	-	-	-	-	0.37

FOR THE YEAR 1961

Date	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
1st	-	0.36	-	-	-	-	-	-	-	-	-	-
10th	-	-	-	0.30	-	-	-	-	-	-	-	-
13th	-	-	-	0.20	-	-	-	-	-	-	-	-
17th	-	-	-	-	-	-	0.10	-	-	-	-	-

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20th	-	-	-	-	-	-	0.13	-	-	-	-	-
22nd	-	-	-	-	-	-	0.12	-	-	-	-	-
23rd	-	-	-	-	-	0.05	0.20	-	-	-	-	-
24th	-	-	-	-	-	0.14	0.15	-	-	-	-	-
25th	-	-	-	-	-	-	0.02	-	-	-	-	-
28th	-	-	-	-	-	-	-	0.15	-	-	-	-
29th	-	-	-	-	-	-	-	0.09	-	-	-	-
30th	0.20	-	-	-	-	-	-	-	-	-	-	-
Total	0.20	0.36	-	-	0.50	0.19	0.72	0.24	-	-	-	-

FOR THE YEAR 1962

Date	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
11th	-	-	-	-	-	-	-	0.20	-	-	-	-
12th	-	-	-	-	-	-	-	-	0.05	-	-	-
14th	-	-	-	-	-	-	0.55	-	-	-	-	-
15th	-	-	-	-	-	-	0.33	0.75	-	-	-	-
16th	-	-	-	-	-	-	-	0.15	-	-	-	-
17th	-	-	-	-	-	-	-	0.68	-	-	-	-
Total	-	-	-	-	-	-	0.88	1.78	0.05	-	-	-

1.7.3 Winds.

The winds generally blow from north and north-east from October to February and from the South and South-east from March to September.

1.8. Health

The health of the community is generally below normal largely because of inadequate and dilapidated water supply and sanitary conditions. Dysentry, diarrhoea and enteric fever are common diseases in the town.

A statement of mortality and morbidity over a period of three years is given in Table 1.4

TABLE 1.4[1]

MORTALITY AND MORBIDITY CASES AT KHAIRPUR

Year	Morbidity	Diarrhoea	Enteric Fever	Mortality
1960	7461	6503	300	58
1961	5485	5478	618	85
1962	2972	2959	330	54

1.9. Geological Conditions

Geologically, Khairpur town lies in a plain composed of alluvial silt deposits which are of a considerable thickness. Fig. 1.5 shows the stratical exploration chart of a trial boring done at new water works located on the west of Mir Wah canal.

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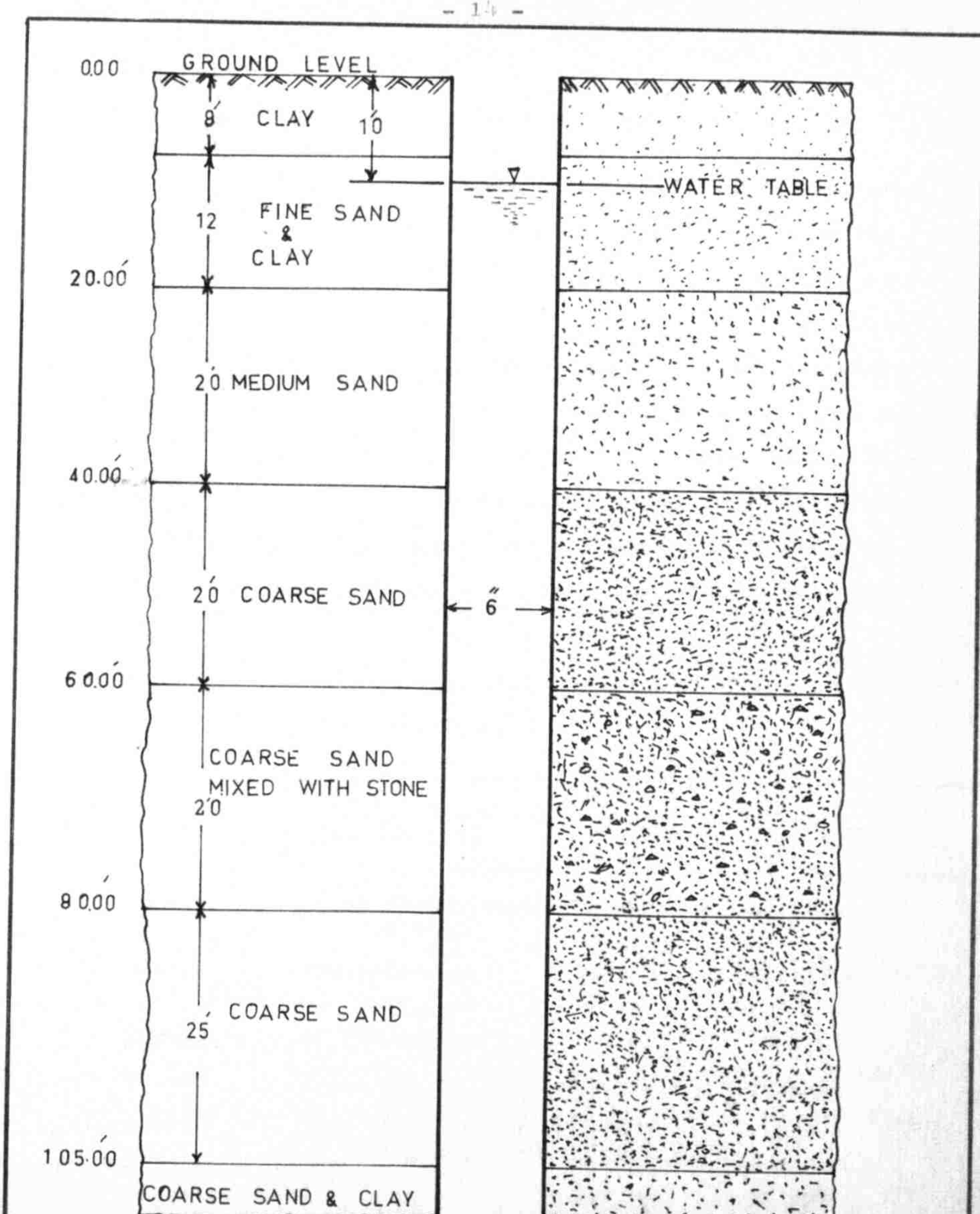

FIG. 1.5 STRATA CHART SCALE. 1cm= 5'

1.10. Ground Water Conditions

The ground water level is very high in this area and varies from 8 to 10 feet below ground level near the Canal and 10 to 12 feet below ground level away from the Canal. The ground water, except for water that seeps from the canal along its side, is generally brackish and unfit for human consumption. A water analysis report for the two samples taken from the two tube wells at water works Khairpur on the bank of Mir Wah canal as given under reference C/966 of 31.1.1961 by the Building and Roads Research Laboratory, Lahore W. Pakistan, is reproduced in Table 1.5.

TABLE 1.5

WATER ANALYSIS REPORT

TESTS	MAXIMUM LIMITS FOR DRINKING WATER	RESULTS	
		TUBE WELL NO. 1	TUBE WELL NO. 2
Colour	20 Platinum Cobalt Scale	Colourless	Colourless
Odour Hot/Cold	Objectionable	Odourless	Odourless
Turbidity	10 PPM Silica	N I L	N I L
Taste	Pleasant	Pleasant	Pleasant
PH	10.6 (Depending on alkalinity)	8.0	8.2
Dissolved Solids		397 PPM	288 PPM
Insoluble matter		N I L	N I L
Total	500 PPM (1,000 PPM for special cases)	397 PPM	288 PPM

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Chlorides	412 PPM	67 PPM	54 PPM
Sulphates	250 PPM	-	-
Hardness: Temporary		53 PPM	49 PPM
Permanent		29 PPM	32 PPM
Total	320 PPM	82 PPM	81 PPM
Alkalinity			
Carbonates (as $CaCo_3$)	120 PPM	108 PPM	103 PPM
Bicarbonates	-	N I L	N I L
Total	Should not exceed the hardness by more than 35 PPM	108 PPM	103 PPM
Sulphurated Hydrogen	Traces objectionable	N I L	N I L
Nitrates	Traces objectionable	N I L	N I L
Nitrites	20.0 PPM	N I L	N I L
Iron	9.0 PPM	N I L	N I L
Other Metals	Lead 0.1 PPM	N I L	N I L
	Copper 3.0 PPM	N I L	N I L
	Manganese 0.1 PPM	N I L	N I L
	Zinc 15 PPM	N I L	N I L

Opinion: Both the samples are fit for drinking purposes.

1.11. Financial Position

The financial position of the Municipal Committee is sound and it will further improve with more development of industries and with the completion of water supply and sewerage systems. The income and expenditure of the Municipal Committee for a 10-year period between

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1952-62 is given in Table 1.6.

TABLE 1.6[1]

INCOME AND EXPENDITURE OF MUNICIPAL COMMITTEE KHAIRPUR MIRS

YEAR	INCOME		EXPENDITURE	
	PAK RUPPEE	EQUIVALENT LL.	PAK RUPPEE	EQUIVALENT LL.
1952-53	111,143.00	73,360.00	125,940.00	83,120.00
1953-54	145,207.00	95,840.00	155,421.00	102,580.00
1954-55	229,316.00	151,350.00	150,103.00	99,070.00
1955-56	213,381.00	140,830.00	179,852.00	118,700.00
1956-57	248,617.00	164,090.00	175,413.00	115,770.00
1957-58	272,809.00	180,050.00	460,553.00	303,960.00
1958-59	322,817.00	213,060.00	569,708.00	376,010.00
1959-60	512,180.00	338,040.00	650,030.00	429,020.00
1960-61	611,996.00	403,920.00	704,976.00	465,280.00
1961-62	758,333.00	500,499.00	666,930.00	440,175.00

Note: 100 Pak. Ruppee are approximately equal to 66 LL.

Exact figures for recent years are not available, however, it is learnt that the figures have approximately doubled the figures in years 1961-62, which shows a greater stability of the Municipal Committee.

CHAPTER 2

POPULATION ESTIMATES - PRESENT AND FUTURE

Design and management of water supply and waste water disposal systems require a knowledge of the quantities of water needed and waste water produced. The determination of the probable figures is important because it fixes the capacity and sizes of the parts of the systems. This involves obtaining information as to the number of people who will be served as determined by forecasts of population and the number of years so called the "period of design", during which the proposed systems and its component structures and equipments are to be adequate.

In the following, therefore, such population figures and design period will be determined and fixed.

2.1. Design Period

Design period is the length of time the system will serve the community before it must be abandoned or enlarged for reason of inadequacy.

The period into future for which the estimate of demand for water and waste produced is to be made depends, to some extent, on the portion of the works being designed and how it is to be financed. If the structure is long lived and will benefit future generations, it will be unfair to the present generation to put the full financial burden upon it. On the other hand, it is unwise and uneconomical to build for too short a period.

However, the following few factors should be given due considerations:[4]

a. The useful life of the structures and equipment used.
b. The ease, or difficulty, of extending or increasing the works.
c. The anticipated rate of growth of the population, with due regard to increases in industrial and commercial needs.
d. The change in the purchasing power of money during the period of retirement of loans.
e. The rate of interest that must be paid on loans.
f. Financial status of the sponsoring authority.
g. The performance of the works during their early years when they are not loaded to capacity.
h. The maintenance cost which tends to increase with the longer use of the structure.

The longer the useful life (a), the greater the difficulty of extensions, (b), the smaller the rate of growth (c), the greater the likelihood of inflation (d), the lower the rate of interest (e), sufficiency of

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funds (f), better the early performance (g), and lower the maintenance cost (h), the further into the future can the design be projected with economic justification. The lengths of design periods often employed in practice are indicated in Table 2.1.

TABLE 2.1[4]

DESIGN PERIODS FOR WATER SUPPLY AND SEWERAGE STRUCTURES

TYPE OF STRUCTURE	SPECIAL CHARACTERISTICS	DESIGN PERIOD-YEAR
a. Water Supply		
Large dams and conduits	Hard and costly to enlarge	25-50
Wells, distribution systems & filter plants, etc.	Easy to extend	20-25
Pipes more than 12 in. in dia.	Replacement of smaller is more expensive	20-25
Pipes less than 12" in dia.	Requirements may change fast in limited area	Full Development
b. Sewerage		
Laterals and Submains less 15".	Requirements may change fast in limited area	Full Development
Main sewers, outfalls, and intercepters.	Hard and costly to enlarge	40-50
Treatment works	When growth and interest rates are less	20-25

In practice normally the design period is taken between 25 to 50 years. For this project, the design period will be taken as 40 years for the reasons listed below. The scheme will be completed in two phases of 20 years each.

a. It is felt that the present Municipal boundaries will be fully saturated with people such that further increase will require additional outside areas.

b. Most of the components of the project like tube wells, filter plants, and machinery etc. have a useful life of 20-25 years, as such their performance during the phase period will be better and less maintenance costs will be required. Also the sewage treatment plant is easy to extend as such the phase selected seems to be justified.

c. The materials like sewers and water distribution pipes have a useful life of even more than 40 years.

d. The rate of interest to be paid for loans is about 5% which is on lower side.

e. Financial position of Municipal authorities seems to be sound. Moreover one third of the capital cost is to be given by the Government as an aid to the Committee.

f. The anticipated rate of population growth is not high so that it does not necessitate shorter design periods.

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2.2. Past Population and Source of Data

The source for population figures in Pakistan after the integration is the individual Bureau of Census for the two provinces which makes decennial counts and publishes reports covering its statistics. The first enumeration was done in year 1951. However, the population figures for last six decades as published in the feasibility report on water supply scheme of Khairpur Mirs are available and are given in Table 2.2 and graphically show in Figure 2.1.

TABLE 2.2[1]

POPULATION OF KHAIRPUR 1911-61

YEAR	POPULATION	%INCREASE (+)/DECREASE (-) PER ANNUM
1911	14,989	-
1921	15,740	(+) 0.65
1931	11,582	(-) 2.65
1941	17,510	(+) 5.1
1951	18,186	(+) 0.38
1961	34,144	(+) 9.00

From the above figures it is deduced that the population growth rate taken by the town was quite low except for the decades ending year 1931 and 1961. These

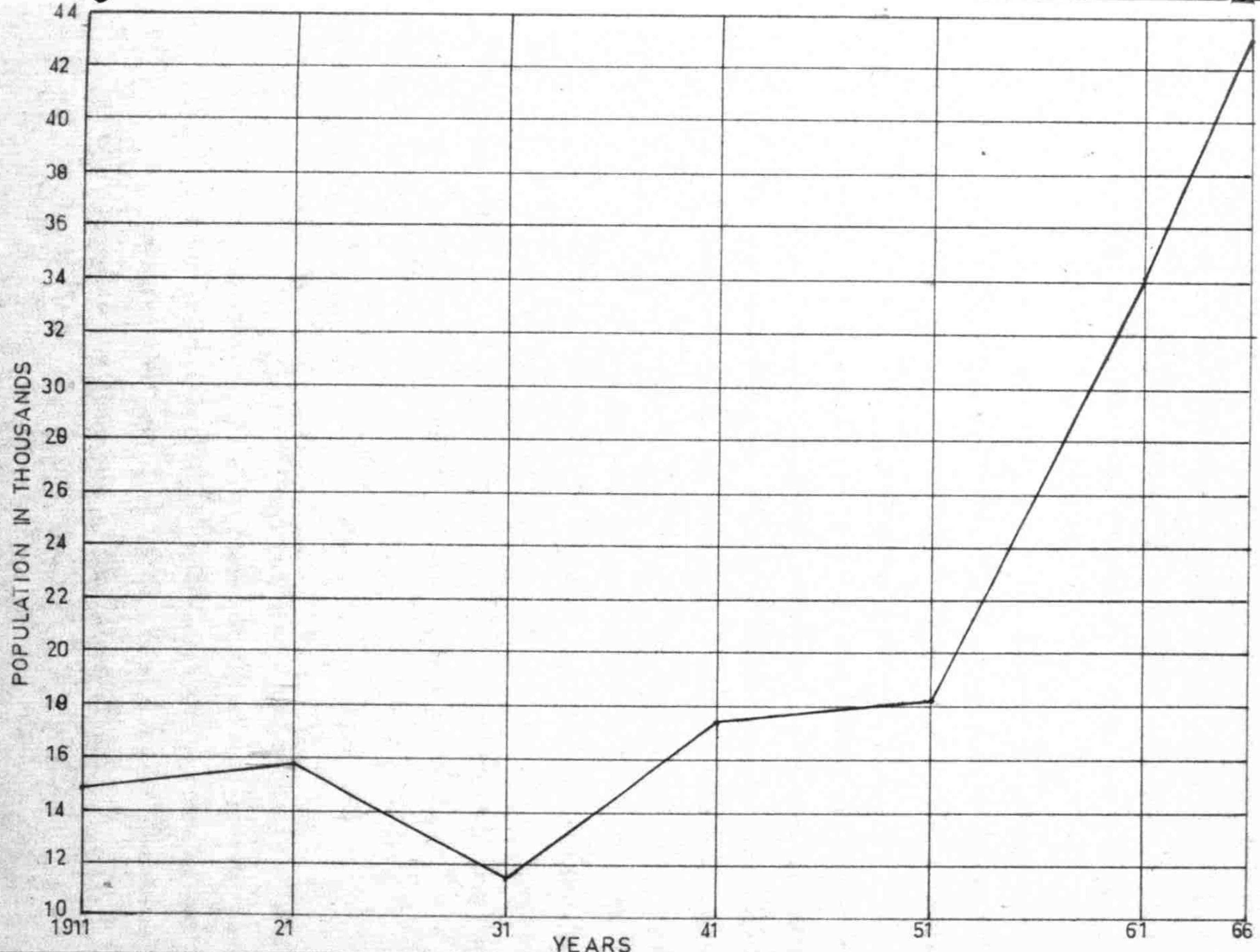

FIG. 2·1 GRAPH FOR PAST POPULATION

two decades have shown quite a significant divergence.

In 1931 the population has decreased by about 27% of that in 1921, whereas in 1961 it has increased by bout 90% of that in 1951.

The reasons for decrease in the decade ending year 1931 is said to be due to several causes viz. influenza, forced labour, the Great War and the State political troubles, when migration took place.[2]

Sharp rise in the decade ending year 1961 is attributed to political, industrial and commercial changes that took place in the area. Another political change had also occured in the year 1947, the Indo-Pakistani integration, but this did not effect much as the major occupancy in the area was of Moslems, this is said to be 83%,[2] and also the number that moved out was nearly equalled by those moving in and occupying the vacated buildings.

No such abrupt changes are, however, expected now, as the conditions have resumed to normal.

2.3. Population Forecasts

Population forecasting is essentially a matter of judgement. Although "Laws of biological growth" have been mathematically presented, yet the success of any forecast "method" must be in its ability to enhance or augment the judgement of the forecaster.

Population change can occur in only three ways (a) by birth (gain),(b) by death (loss) or (c) Emigration of migration (gain or loss).

The factors causing births, deaths and movement are virtually infinite in their variety. However, a rational forecast demands to determine these factors and their degree of importance. The usual approach is to determine the effects and trends of pertinent factors in the past, and then to estimate the probable deviations of these in the future.

Certainly, this is difficult, as such data and resources may require the use of assumptions, and, forecasts do contain certain assumptions whether stated or not.

In addition to such special assumptions several basic assumptions are implicit. V.B. Stanbery has summarized these as: (5)

a. The form of Government and the political, economic and social organisations will remain unchanged.
b. No all-out war, internal revolution, Nation wide devastion, will occur.
c. No large scale epidemic, destruction by military action, fire, earthquake, or other disaster will occur in the area or in its neighbourhood.

Graphic extra-polation and projections are among the simplest and common methods usually adopted in population forecasting, common being Arithmetic, Geometric projection and vizual least square regression.

Comparative method in which projections are done from the plots of larger areas whose earlier growths have exhibited characteristics similar to that area under study and has reached the present population figures of area some time in the past.

Mathematical equations based on indentifiable mathematical relationship have also been formulated for the purpose.

However, the basic weakness in all the above methods is their implicit assumption, that, relationship that has existed in the past will continue to exist in the future, with the same intensity.(5)

Other methods like "Ratio and Correlation methods", which assume that the rate of population growth for most areas is related to the rate of growth of state or national population, has shown success in some cases but unfortunately such statistics are lacking in our case.

Still another method so called "Component method", which involves summation of separate but related projections of natural increase and net migration, could be used but again we are handicapped of the available data.

"Employment Forecast method" could also be best used if future employment of an area could be fore-casted accurately. This sort of forecast which is purely based

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on commercial and industrial activity, itself is least predictable element of the population change.

The previous statistics cannot be used due to the fact that great variations are noted in the growth trend and are found to be due to historical incidents, political changes, and the sudden industrial development in the area.

As such, perhaps the graphical-mathematical extra-polation can only be the ending solution.

A review of arithmetic projection reveal that in the long run, a constant assumed rate of increase results in quite a large figure or so called the over estimates, still larger figures are reached with the same increase rate by geometric projections.

The actual growth follows somewhere in between the two as studied by Pearl and Reed in the development of logistic or S-shaped curve, bounded by an upper limit. This curve is characteristic of all forms of life growths within a limited space, early growths taking place at an increasing rate, later growth taking place at a decreasing rate as a saturation value or upper limit is approached.[4]

2.4. Present Population

Determination of the current population too involves projection from previous census. However, the forecaster here is helped by the fact that growth has already taken place as such he can measure the success of the method he has used, it is other question that such enumeration becomes expensive and time-consuming for most of the sanitary projects hence it is not usually practised.

It is sufficient and satisfactory to add to the census year population a sum equivalent to the estimated population increase during the equivalent time period. The reasonable rate of increase may be approximated and assumed for the purpose.

Fortunately, in our case, the local enumeration as conducted by the M.C. Authorities for the year 1966 is available, to this the approximate increase, to arrive at the figures for present, can be easily added. The population for the year 1966 is 43,200 souls.[6] Now the population as per 1961 census was 34,144 souls, this shows an increase of about 26% over a period of 5 years or 5.2% per year. For our purpose we can easily adopt a 4% increase for the year 1967 as the increase seems to adopt a decreasing trend in comparison to figures for year 1951 and 1961. With this increase the present population for the year 1967 comes to be 44500 souls.

2.5. Future Population

In estimating the future population an upper limit or so called the saturation limit has been fixed as 25 years hereafter, ie. in year 1992. The percentage rate of increase has been considered as 2.5% per year with necessary decrease after saturation limit. Estimates have been done on the basis of arithmetic and geometric increase and ultimately an average of the two has been taken.

Figure 2.2 shows three curves. Curve (A) represents the population increase based on arithmetic annual increase, future population are given in Table 2.3.

Curve (B) represents the population increase based on geometric annual increase, future population are given in Table 2.4.

Curve (C) represents average of the two, future populations are given in Table 2.5.

Thus following the discussion in section 2.3, curve (C) represents the future population.

T A B L E 2.3

FUTURE POPULATION ACCORDING TO ARITHMATIC PROJECTION

YEAR	POPULATION
1967	44,500
1977	55,500
1987	66,600
1992	72,200

1997	77,500
2007	85,000

TABLE 2.4

FUTURE POPULATION ACCORDING TO GEOMETRIC PROJECTION

YEAR	POPULATION
1967	44,500
1977	57,000
1987	72,800
1992	82,300
1997	91,000
2007	103,000

TABLE 2.5

FUTURE POPULATION ACCORDING TO CURVE (C)

YEAR	POPULATION
1967	44,500
1977	56,000
1987	70,000
1992	77,500
1997	84,500
2007	93,000

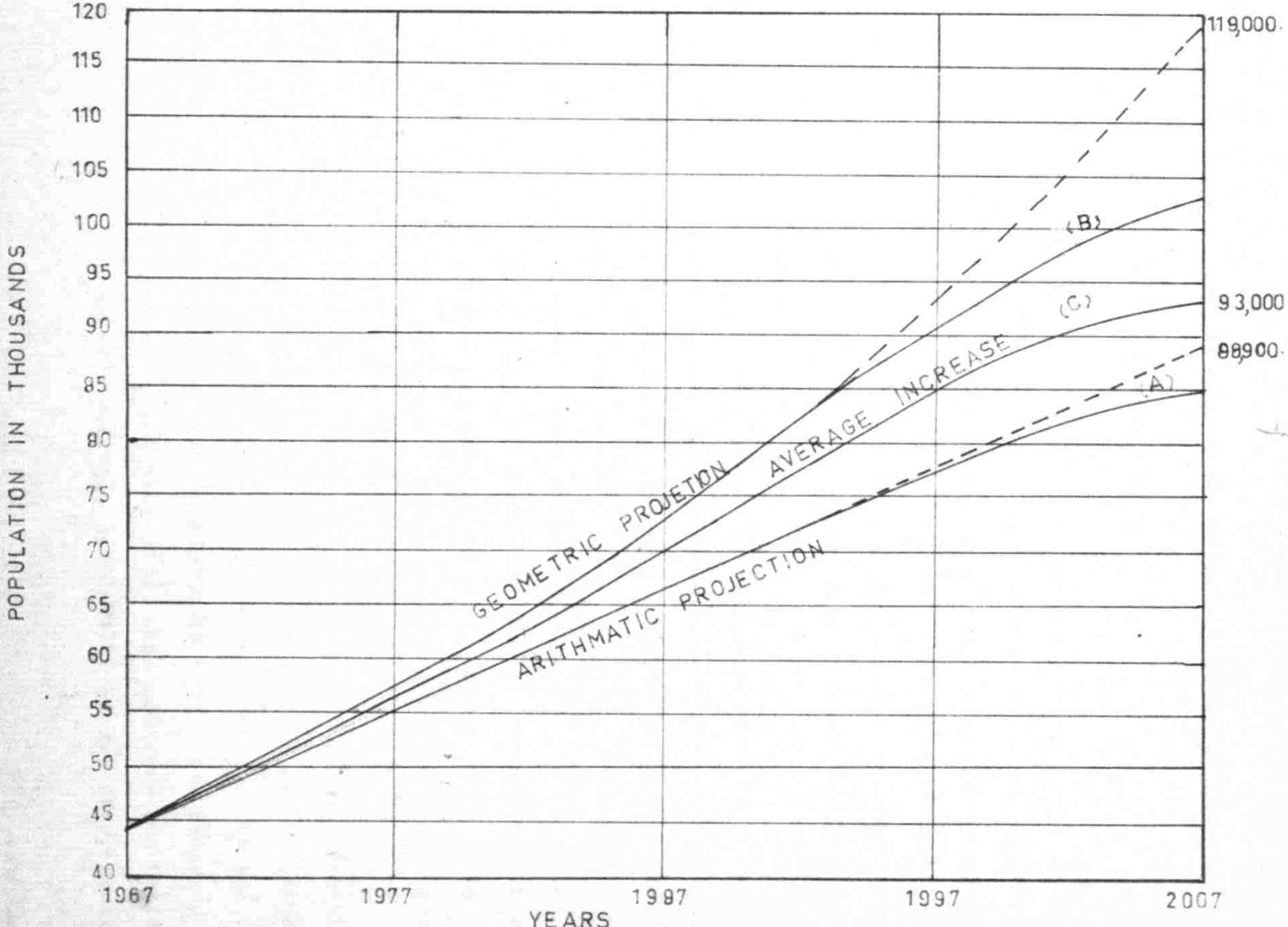

FIG. 2.2

POPULATION GROWTH CURVES.

2.6. Direction Of Development

2.6.1 Present Direction

As already stated, the town of Khairpur is very poorly planned particularly the main town which consists of old buildings and zigzag narrow streets. However, good planned residential areas have, and are being developed on the left bank of Mirwah Canal and eastern parts of the town like Mumtaz Coloney. The old buildings in the main town are, however, being gradually replaced by new ones.

The tendency of buildings at present is comparatively more in the eastern portion of the town than in the western because of the following reasons :-

a. Nearly all the main roads including main "National Highway" passes through that part.

b. Nearly all the Government as well as non-government offices are located in that portion of the town.

c. The commercial institutions including the big grain market are also located in that part.

d. Most of the western part is water logged particularly the luqman area on the other side of railway line.

e. The portion in between railway line and Mir wah Canal is rapidly being occupied by industrial sectors, gardens and public buildings like colleges and hostels. There are very few residential blocks at present, more

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are being built but comparatively at lower rate.

Since after becoming the Divisional headquarter there has been a tendency for a number of head offices in the various districts under jurisdiction to shift to Khairpur and as such the number of public buildings etc. have been and are being built in the town.

Industry and agriculture are the two main prospering and developing fields in the town.

2.6.2 Future Direction

Khairpur is one of the important and biggest grain market in the division.

Recently a big agricultural work-shop has been established in the area in order to enhance and encourage the adoption of new modern techniques of agriculture and also to develop through proper shaping and ploughing etc. the thousands of acres of the barren lands all around for good agricultural use.

A second big scheme of "water logging and salinity control" is also under administration for the division, the completion of which will end the problem of high water table, seepage, and brackish water, through the installation of thousands of tube-wells and leaching of under ground reservoirs. This will finally result in the land reclamation and subsequent increase in agricultural use.

From the above it is, therefore, deduced that one of the main directions of development in future is expected to be agriculture and thus subsequent trade and commerce.

The second direction seems to be industry. Industry has developed very much in the area, industries like tannery, silk, cotton textile mill and R.C.C. pipe factory are existing in the area.

Recently Government has established a small industrial estate in the town to localize and encourage the small and cottage industries.

These efforts and the present trend clearly dictate that the town will have a very good hold over this field of development in ftuure to come. Major industrial establishment will take place in the western portion of the town along and near the railway line.

It is, therefore, esteemed that present area will be completely built-up in a period of 40 years hereafter with major industrial and residential establishments in western and eastern portions of the town respectively and thereafter M.C. will have to extend its present limits to accommodate further growth.

A master plan for the city is under consideration too.

2.7. Population Densities

2.7.1 Present Population Densities.

Estimates of the density of the population and the nature of occupancy of the various portions of an area are required in the design of a water distribution and sewage collection systems within the community.

Accordingly the town of Khairpur has been divided into three zones as detailed below. The "Luqman area" has been left as it forms a separate drainage district and is outside the limits.

a. Zone A (Main town Zone)

This zone consists of major dwelling and commercial parts of the town. It has a number of unplanned houses and narrow streets. The old houses are by and by being replaced by new buildings. The families are of multiple family type and the area is densily populated. Themain grain market and Commissioner's office, etc. fall in this area.

b. Zone B (Mumtaz Coloney Zone)

This zone consists in part of the few old occupied unplanned buildings in Burghari and the rest as a newly developed planned coloney. The area is very splashly populated, however, the present trend of occupancy and building seems to increase in this part of the town.

c. Zone C (Industrial area Zone)

This zone which forms a strip in between the

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railway line and Mir Wah Canal is mostly occupied by industrial blocks and few dwelling sectors like: Banarsi Coloney, etc. The Civil Hospital falls also in this zone. The tendency of residential occupancy in this area is comparatively less. It is somewhat away from the main busy center of the town.

The present population densities in various zones are given in Table 2.6 and shown in Drawing No.2.

TABLE 2.6

PRESENT POPULATION DENSITIES

ZONE	AREA IN ACRES	EST.POPULATION DENSITY (PERSONS PER ACRE	NUMBER OF PERSONS
A	528	46	24,330
B	447	29	12,990
C	477	15	7,180
TOTAL	1452		44,500

2.7.2. Future Population Densities.

The increase, in population, commerce, industry, new constructions, facilities, and many more favourable factors in a limited area cause a proportional change in the population density of that area.

From the present trend of increase and expected future, it is deduced that the present limits will be completely saturated in a period of 40 years hereafter.

With due consideration to all these and various other factors the expected population densities at the end of 40 years' period ie. by the year 2007, have been calculated and are given in Table 2.7 and shown in Drawing No. 2.

TABLE 2.7

FUTURE POPULATION DENSITIES

ZONE	AREA IN ACRES	EST.POPULATION DENSITY (PERSONS PER ACRE	NUMBER OF PERSONS
A	528	72	38,000
B	447	75	33,500
C	477	45	21,500
TOTAL	1452		93,000

CHAPTER 3

WATER CONSUMPTION AND EXISTING WATER SUPPLY AND DRAINAGE CONDITIONS

3.1. Existing Water Supply Conditions

Khairpur has an existing water supply plant which was constructed in the year 1925. The distribution system based on this supply was spread over in Zone A only, the rest of the two zones viz: Zone B and C had to resort to their own individual sources.

The enormous increase in population until now and occupancy of the zones B and C called for a need to realize that the supply has on one hand not only absolutely neglected the two zones but on the other hand is even insufficient for the zone it was supposed to serve. This need was shaped into a proposal to provide for the additional requirements and to this effect a scheme is approved by the Government which provides for the necessary distribution net work in the three zones and subsequent additional supply.

The existing supply consists of two shallow tube wells capable of a discharge of about 10,000 gallons/hour, and an overhead reservoir of 30,000 gallons capacity and a distribution network in Zone A only. No treatment except chlorination is given to tube well water.

None of the industries at present is being supplied by this plant, the water is purely used for domestic purposes in houses and other sanitary purposes in few public buildings in the area. Domestic requirements in the other two zones are met through shallow insanitary hand pumps.

From the various components of the approved new scheme; part of the distribution system is completed and the rest is being completed. About two experimental tube wells have been installed and one over head reservoir of 100,000 gallons capacity is scheduled to be completed now. The new distribution system has been linked to an old one, pipes have been replaced wherever found necessary. The existing supply is not switched to all newly laid lines because of insufficiency in quantity of water, and is checked through junction sluice valves. The final switching over is awaited for the final decision on the supply source, which, in other words, in part is the aim of this report.

3.2 Water Consumption

3.2.1 General.

Water consumption is purely a relative term, amount is variable according to its use, like: domestic, industrial, commercial and public use, and the various factors, like: standard of living of the community, quantity and quality available, adequacy of source, climatic conditions, cost, metering, reliability and efficiency of the system, drainage conditions of the area and available pressure, under which it is being used.

A survey of American cities has given average daily per capita consumption figures ranging from 35 to 528 gallons.[7] From this example a wide variety in rate could be well judged. There are no hard and fast rules to fix this figure. It mainly depends on the local conditions and requirements. However, some guiding figures are given below. Table 3.1 gives the approximate percentage sharing of various purposes served by a supply in a community.

TABLE 3.1[7]

CONSUMPTION OF WATER FOR VARIOUS PURPOSES

USE	GALLONS PER CAPITA PER DAY	PERCENTAGE OF TOTAL
Domestic	60	40.0
Industrial	32	21.3
Commercial	21	14.0
Public	15	10.0
Loss & Waste	22	14.7
TOTAL	150	100.0

Table 3.2 shows the quantities delivered in North American Communities.

T A B L E 3.2[4]

Quantity g.p.c.p.d.

CLASS OF CONSUMPTION	NORMAL RANGE	AVERAGE
Domestic	15-70	35
Commercial & Industrial	10-100	30
Public	5-20	10
Water unaccounted for	10-40	25
TOTAL	40-230	100

3.2.2 Present Water Consumption.

There are no exact statistics for water used at present, however, it is learnt that the two tube wells of existing water works are run for 12 hours in summer months and for 8 hours in winter months. The discharge of each tube well is said to be 10,000 gallons per hour. These figures give a figure of 240,000 gallons/day in summer and 160,000 gall/day in winter, giving an average per capita figure of about 20 g.p.c.p.d., when the connections are said to be about 50% in Zone A.

However, the new proposed scheme which was first designed on the basis of 15 g.p.c.p.d. and later on revised and based on 30 g.p.c.p.d. for domestic use excluding commercial and industrial use, can be a fair guide in assuming these figures.

All the big industrial units including industrial estate and civil hospital which form the major sources of water consumption have their own individual water supplies. The proposed scheme will not, therefore, cater for this use.

The specific water consumption figure, which is the total water use plus losses divided by the number of population, as assumed here to be 20 g.p.c.d. is, therefore, assumed to account for minor industrial use by small cottage industries, public buildings, gardening and losses, etc.

3.2.3 Future water Consumption.

The amount of water consumed per person tends to vary with passage of time and increases in direct proportion to the increase in standard of living.

Present trend of development and similar expectations in future clearly demonstrate that an improvement in the living standard of people and thus the subsequent increase into a need for water is likely to occur in future.

Similar conditions have occured in Sukkur, a city 14 miles north of Khairpur, as already mentioned. This city was nearly in a similar state of conditions about 16 years before. Industrial and other improvements in the area has caused a remarkable change in every phase. Water which was consumed at the rate of 20 g.p.c.p.d. has shoted upto a figure of 35 g.p.c.p.d. by now. Similar changes are expected to occur in Khairpur thus the present demand is expected to shoot upto 35 g.p.c.p.d. after about 20 years hereafter ie. by the year 1987.

Ultimate demand is in no way expected to exceed 50 g.p.c.p.d. This is the maximum ultimate figure adopted for a period of 40-50 years hereafter in the design of urban water supplies of other communities in the area. For our purpose a figure of 45 g.p.c.d. will be a reasonable one to adopt. This figure is to cover all domestic, commercial, small industrial, and other miscellaneous uses as already described.

The total daily water consumption is calculated by the product of the estimated number of population for the different years of the design period and the specific water consumption for that year.

The number of consumers using water from the distribution system, which is said to be 50% in Zone A, might reach in the beginning to 40% on average in three Zones. However, water will also be taken by rest of the consumers through standposts, but, of course, not equivalent to designed per capita figure.

Therefore, a figure of 50% and 90% for total water consumption for the present and for the year 1987 respectively can be a fair approximation. Such percentages have been taken into consideration while calculating the total water consumption given in Table 3.3 and shown graphically on figure 3.1.

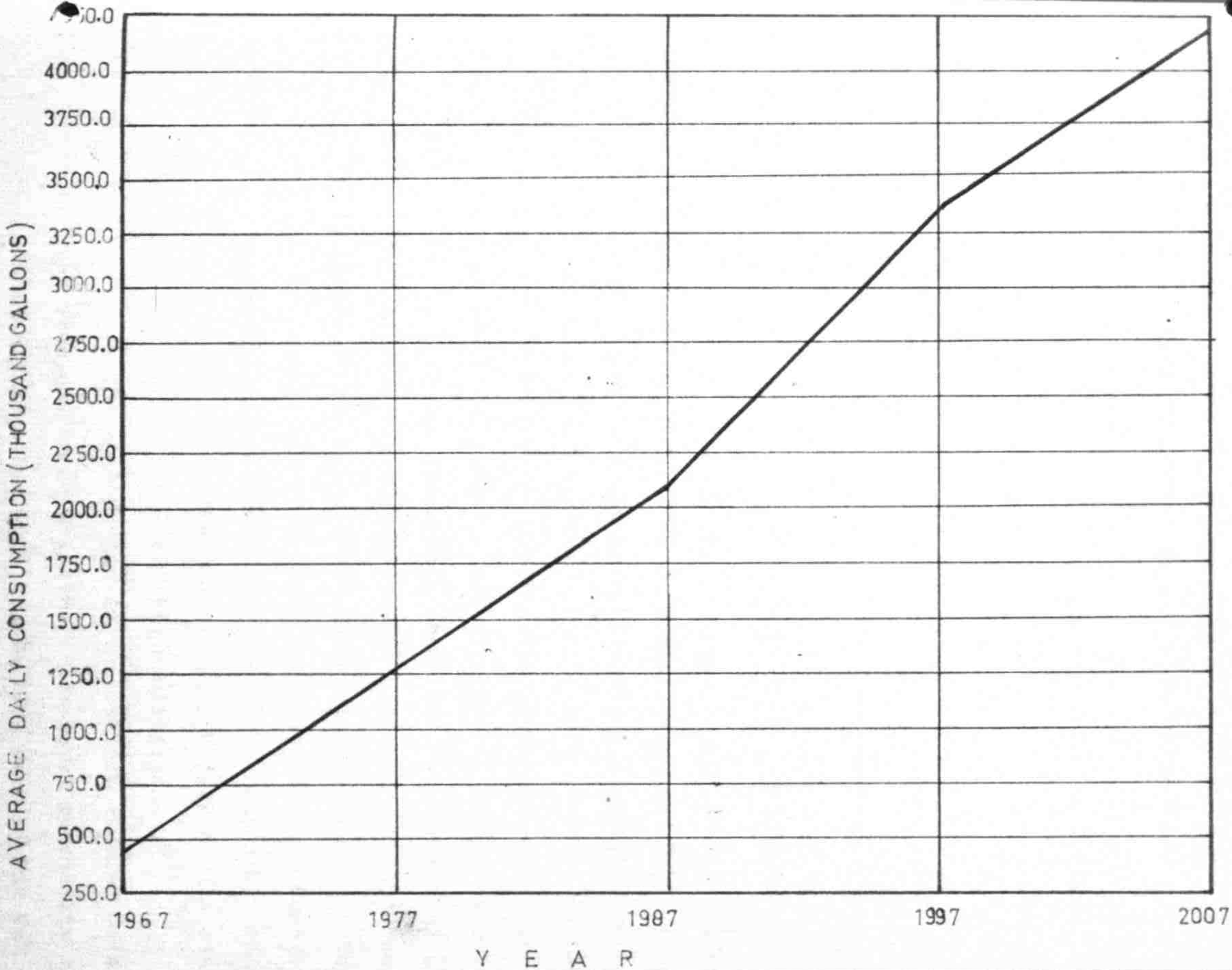

FIG. 3-1 TOTAL WATER CONSUMPTION

TABLE 3.3

ESTIMATED AVERAGE DAILY WATER CONSUMPTION

YEAR	AVERAGE DAILY CONSUMPTION (THOUSAND GALLONS)
1967	445,
1987	2205,
2007	4185,

3.3. Existing Drainage Conditions

In general the drainage conditions of the town are very poor. The system consists of open drains in some parts of the town and in the rest the waste water from kitchens and baths, etc. collects into cesspits in front of each door. The stagnant water in the cesspits emit foul smell and pollutes the atmosphere. These cesspits are further the breeding centers for flies and mosquitos. The cesspits which are in certain areas formed by the ditches in low lying areas and collect water from the house cesspit overflows or direct discharge of some of the house drains assume some time such unhealthy conditions which can easily jeopardize the life of the inhabitants besides the asethetic manifestation.

The open drains in the western side of Main town area discharge into the extra-mural open drain terminating into a big sump on the left bank of Mir Wah Canal near road bridge. The sullage water from this sump is pumped into the Canal for its final disposal. This, of course, is dangerous for people downstream.

Another main surface drain serving part of Mumtaz Coloney also, terminates into a sump located on the Eastern side of Main National highway. The sullage water from this sump is spread overthe sewage farm and is really a cause of nuisance because of being located in the direction of wind.

Further more, the sumps which were constructed long before, are not bein maintained properly and are liable to pollute the underground waters.

The drainage water of the industrial area zone, which mainly consists of industrial wastes is also collected through open drains and discharged into low lying barren lands situated in the southern part of the town and on the eastern side of railway station. These cesspits or ditches as they may be called are less injurious to inhabitants as being remote and situated against the wind direction.

The above detailed conditions will be more enhanced with the more use of water which is to follow the complete proposed water supply system for the town. It is, therefore, high time to call for a comprehensive sewerage schme for the town.

CHAPTER 4

WATER QUANTITY, QUALITY AND SOURCE SELECTION

An estimate of the amount of water that may be used for various specific purposes is among the first steps to be taken in the design and management of water works for a community.

Probable figures as "Specific Water Consumption", and total water consumption, were estimated in Chapter 3. The amounts specified therein are esteemmed to cover, as described, all the various needs of the Community except fire. This will, however, be mentioned below.

Not only, the quantity but also the quality of water to be supplied to a community, requires due attention.

Water has a direct relationship with human health and adaptability. A water, which is not fit for municipal use for reasons of tastes, odours, asethetic, or health hazards, will in no way be accepted.

Thus the success of a water works management lies in the extent to which it is able to meet with the quantitative and qualitative needs of a community.

In the following, therefore, a quantitative and qualitative review of the requirements and the available supply sources will be presented. In light of this review proper source selection will also be made.

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4.1. Water Quantity Required

Table 3.3 shows the estimated total daily water consumption from the year 1967 through 2007.

The figures mentioned therein are average. However, variations are to be expected in demand over the various hours of the day, various days of the week and various months of the year. But for design purposes average figures are to be taken. For comparison sake, these variations are expressed as ratios to the average demand as given in Table 4.1.

TABLE 4.1[4]

NORMAL RANGE OF MAXIMUM TO AVERAGE DEMAND

RATIO	NORMAL RANGE	AVERAGE
Maximum day: average daily	(from 1.2 to 2.0):1	1.5:1
Maximum hour:average hour	(from 2.0 to 3.0):1	2.5:1

The quantities of water estimated, as already stated, satisfy all other specific purposes like domestic, industrial, commercial, public, waste, and miscellaneous except fire. This phase of demand is, of course, to be given due consideration.

Although the volume of water used in quenching fires is relatively small, the rate at which it must be supplied is very great. The rate is determined by factors such as the bulk, congestion, fire resistance, and content

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of buildings.

Required fire flow is a function of population and the general structural conditions of the district as fixed by the National Board of Fire under writers. The requirements vary between 1,000 g.p.m. for 1,000 persons and 12,000 g.p.m. for 200,000 persons, with a maximum of 20,000 g.p.m.[8] There must be enough water to provide for a 5-hour fire for towns of less than 2,500 persons and a 10-hour fire for larger cities. A minimum limit of fire demand is the amount and rate of supply that are required to extinguish the largest and probable fire that could be started in a community. A minimum of 4 streams each of 175 g.p.m. in low risk district, and 250 g.p.m. in high-risk districts are required.[8]

There are some empirical formulas for calculating rate of fire demand. An allowance based on such calculations is recommended to be for peak fire demands in water works design. Such formulas are given in Table 4.2.

TABLE 4.2[8]

EMPIRICAL FORMULAS FOR RATES OF FIRE DEMAND

NAME OF AUTHORITY	FORMULA: Q=DEMAND, GPM P=POPULATION, THOUSANDS.	GPM FOR 100,000 POPULATION
Kuiching (on basis of fire streams, of 250 g.p.m.) -------	$Q = 700\sqrt{P}$	7,000

John R. Foreman --- $Q = 250 \left(\frac{P}{5} + 10\right)$ 7,500

National Board of
Fire under Writers --$Q = 1,020 \sqrt{P} \left(1 - 0.01 \sqrt{P}\right)$ 9,180

As regards the pressure to be maintained at the hydrant should, in general, not be less than 20 psi where mobile fire pumpers are used and 75 to 90 psi otherwise.(8)

From the above it can be deduced that sufficiently large quantities should always be kept available for the purpose. The quantities should be available in storage which may be elevated or surface built. When reserve storage is elevated quantity can be estimated from the following formula:-

$$\text{Reserve storage (gal)} = \left[F(\text{gpm}) - P(\text{gpm})\right] T(\text{min})$$

where,

F= Fire demand

P= Reserve fire pumping capacity

T= Duration of fire.

Reserve storage as calculated above is to be added to the capacity calculated for an equalizing or distribution reservoir.

Since elevated storages become expensive as such in some system it may be more economical to provide for fire reserve in a surface level reservoir and pump into the distribution system during emergency.(8)

A study of Khairpur town reveal that the town is not susceptible to frequent fires. Moreover, the industrial area is located quite far from the densily populated area viz. Main town. The houses are mostly single storeyed and brick built. At present, water lorries carrying sufficient water in tanks, and trolley pumps, are used for the purpose. It, therefore, will be an unnecessary financial burden for the Municipal Committee to provide for such a large storage particularly in the elevated resevoir and a reserve fire pumping capacity and keep it ready for an occurance which is rare to occur.

However, since the water supply of Khairpur is to be based in part on the tube wells, as will be described in the subsequent chapters, it is, therefore, not necessary to provide for such a reserve, underground reserve will always be available and can be pumped through normally working pumps or standbys if a fire break-out occurs when these are not in operation. The pressure thus obtained at a fire hydrant will be sufficient, however, a fire pumper may be used to provide for additionals if so required.

From the above, it is, therefore, deduced that no additional supplies will be required for fire fighting. The total quantity required and thus to be collected will be as given in Table 3.3.

4.2. Water Quality Required

One of the main objects in water works practice

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is to provide for the consumer a water of good physical quality, free from unpleasant taste or odour and contains nothing which may be detrimental to health. The statement can be put into a more technical form, saying that, to meet the general sanitation requirements, water supplies must possess the two interwined attributes: wholesomeness and palatability. To achieve wholesomeness, water must be

(a) uncontaminated and hence unable to infect its user with a water borne disease,

(b) free from poisonous substances and

(c) free from excessive amounts of mineral and organic matter.

To be palatable, water should be free, or significantly so, from colour, turbidity, taste and odour, of moderate temperature in summer and winter, and aerated.

In the above paragraphs, a general criteria to weigh a water to be qualitatively acceptable has been set forth in words. However, specific water quality criteria for each of the requirements have been set by the various authorities working in this field. These standards, as they are called, express the limits in quantitative terms in light of which, a given sample can be weighed.

A sample is said to be qualitatively acceptable if it does not cross the boundary set forth by these standards.

Limits are not supposed to be a binding for a community, to adopt, which has no other alternative. However, it is always advisable to resort to the source which is the best and economical among those available. While making a decision these limits should, however, be borne in mind and followed as a guide if not decisive. The limits for toxic and poisonous materials are not to be exceeded. also the limits set for interstate carriers are to be abided by those who come under jurisdiction.

The two agencies are prominant in this field viz. World Health Organization (W.H.O.) and United States Public Health Service (U.S.P.H.S.). The limits given by the two differ in certain cases, however, for toxic substances they do agree with one another.

4.2.1 Drinking water Standards.

Table 4.3 shows the comparison of chemical constituents in the drinking standards as given by the two agencies.

A review of the above Table shows that two standards prescribe the same limits for toxic substances.

Limits as prescribed by W.H.O. in International standards for physical quality of drinking water are given in Table 4.4.

TABLE 4.3[9]

COMPARISON OF CHEMICAL CONSTITUENTS IN THE DRINKING WATER STANDARDS OF OF THE W.H.O. AND U.S.P.H.S.

CONCENTRATION IN MILIGRAM PER LITER(MG/L)

CHEMICAL CONSTITUENT	WHO INTERNATIONAL (1963)* PERMISSIBLE LIMIT	WHO INTERNATIONAL (1963)* MAXIMUM ALLOWABLE	WHO EUROPEAN (1961) RECOMMENDED LIMIT	WHO EUROPEAN (1961) TOLERANCE LIMIT	USPHS (1962) RECOMMENDED LIMIT	USPHS (1962) MAXIMUM ALLOWABLE
Alkylbenzene Sulfonate	0.5	1.0	-	-	0.5	-
Ammonia (NHo)	-	-	0.5	-	-	-
Arsenic	-	0.05	-	0.2	0.01	0.05
Barium	-	1.0	-	-	-	1.0
Cadmium	-	0.01	-	0.05	-	0.01
Calcium	75	200	-	-	-	-
Carbon Chloroform extract	0.2	0.5	-	-	0.2	-
Chloride	200	600	350	-	250	-
Chromium (hexavalent)	-	0.05	-	0.05	-	0.05
Copper	1.0	1.5	3.0	-	1.0	-
Cynide	-	0.2	-	0.01	0.01	0.2
Fluoride	1.0	-	1.5	-	0.8 - 1.7**	1.6 - 3.4**
Iron	0.3	1.0	-	0.1	0.3	-
Lead	-	0.05	-	0.1	-	0.05

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Manganese	50	150	125***	-	-	-
Magnesium & Sodium Sulphate	500	-	-	-	-	-
Manganese	0.1	0.5	0.1	-	0.05	-
Nitrate (as No_3)	-	45	50	-	45	-
Oxygen dissolved (min)	-	-	-	5.0	-	-
Phenolic Compounds (as phenols)	0.001	0.002	-	0.001	0.001	-
Selenium	-	0.01	-	0.05	-	0.01
Silver	-	-	-	-	-	0.05
Sulphate	200	-	250	-	250	-
Total Solids	500	1500	-	-	500	-
Zinc	5.0	15.0	5.0	-	5.0	-

* : Original contents of the Table under these columns were taken from 1st edition of 1958, here these have been replaced from latest edition of 1963. (10)

** : Limits vary inversely with mean annual temperature.

*** : If sulphate content is 250 mg/L, magnesium should not exceed 30 mg/L.

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TABLE 4.4[10]

STANDARDS OF PHYSICAL QUALITY FOR DRINKING WATER

ITEM	MAX. ACCEPTABLE CONCENTRATION	MAX.ALLOWABLE CONCENTRATION
Colour	5 units	50 units
Turbidity	5 units	25 units
Taste	unobjectionable	-
Odour	unobjectionable	-
P.H. range *	7.0 - 8.5	Less than 6.5 or greater than 9.2

* This item actually falls under chemical quality.

With respect to bacteriological criteria, the WHO international standards are approximately equal to the USPHS standards in requiring that the arithmetic mean coliform density should not exceed one organism in 100 ml. of water.[9]

Regarding radiological requirements neither WHO nor USPHS standards give permissible limits for short time-use or for small populations, however, limiting values are tentatively established by WHO in international standards as maximum acceptable limits in drinking water as supplied to consumers for life-time use for large populations as follows:[10]

Strontium - 90	30 picocurie/liter
Radium - 226	10 picocurie/liter
Gross beta concentration (in absence of Strontium-90 and alpha-emitters)	1000 picocurie/liter

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Note: Picocurie is equal to micro microcurie.

4.2.2 Raw Water standards.

In the development of standards for raw waters to be used as sources of public water supply, major attention has been limited to components affecting health, define pollution to indicate the need for treatment.

A study on the subject reveal that different countries have followed different standards for their raw water sources.

Table 4.5 given below represents a summary of current thought on the subject and is established from the general pattern which these standards tend to follow.

TABLE 4.5[9]

RANGES OF PROMULGATED STANDARDS FOR RAW WATER SOURCES OF DOMESTIC WATER SUPPLY

CONSTITUENT	EXCELLENT SOURCE REQUIRING DISINFECTION ONLY	GOOD SOURCE REQUIRING USUAL TREATMENT AS FILTERATION AND DISINFECTION	POOR SOURCE REQUIRING SPECIAL TREATMENT AND DISINFECTION
BOD (5 day) mg/L Monthly average	0.75-1.5	1.5-2.5	over 2.5
Maximum day, or sample	1.0-3.0	3.0-4.0	over 4.0
Coliform MPN per 100 ml Monthly average	50-100	50-5000	over 5000

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Maximum day, or sample	Less than 5% over than 100	Less than 5% over 5000	Less than 5% over 20000
Dissolved oxygen mg/l average	4.0-7.5	4.0-6.5	4.0
% saturation	75% of water	60% of water	-
PH	6.0-8.5	5.0-9.0	3.8-10.5
Chlorides,max,mg/L	50 or less	50-250	over 250
Fluorides,mg/L	less than 1.5	1.5-3.0	over 3.0
Phenolic compounds max, mg/L	N o n e	0.005	over 0.005
Colour units	0-20	20-150	over 150
Turbidity units	0-10	10-250	over 250

4.3. Water Quantity Available

The source of water determine the nature of the collection, purification and distribution works.

Municipal supplies may be drawn from a single source or from a number of different ones. The water from different sources is ordinarily mixed before distribution to the community provided that the component waters are safe and suitable in quality.(4)

Both ground and surface waters are the possible sources of public water supply for the Khairpur town.

Without sufficient data required for computations, it is exceedingly difficult to estimate exactly the amount of ground water that is available.

Data usually required include porosity of the soil, the geological formation, the slope of water table, the rainfall, extent of acquifer yielding the supply, velocity of moving water, permeability of the soil, direction of the flow, boundary conditions, and detailed results of pumping tests from test wells driven for exploratory purposes.

The quantity that can be gathered from a surface source vary directly with the size of catchment area, or water shed, and with the difference between precipitation and losses if the supply source is an impounded reservoir, and the total discharge of stream if the supply source is a river or stream.

4.3.1 Ground water quantity.

No records on porosity and permeability of soil and hydrautic gradient and many other things required to estimate the flow, are available for Khairpur.

However, certain results, as obtained from test tube wells installed along the Mir Wah Canal for exploratory purposes and from the tube wells of existing water works are available. These results can serve as a fair guide in estimating nearly accurate if not exactly accurate amount of ground water supply that can be obtained from the source.

The test tube wells are driven to a depth of 100 feet below ground level and equiped with 6" dia cook's

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strainer 30 feet in length. From number of tests it has been deduced that an average water yielding capacity or specific yield expressed in terms of gallons per square foot of cross sectional area of strainer per hour per foot of depression head or draw down comes to be 12.[1] The stabilized water level reached at a maximum pumping rate was recorded as 12 feet below static water level in the well. Two test tube wells are installed and it is found that there is no influence of one's working over the other. The distance between the two is 250 feet which shows that the radius of influence for either of the tube wells falls within a length of 125 feet on either side. The above figures have been verified with the results from existing tube wells and seem to show a better similarity.

From the above figures for a 12" dia. strainer of 40 feet in length the discharge per hour comes to be:

12 x ‾ x 1 x 40 x 12 = 18,090 gallons per hour.

Adopting 16 hours working in a day

Discharge per tube well = 18,090 x 16 = 289,000 gal/day

Discharge from the existing tube wells= 10,000 gal/hour

Adopting similar 16 hour working in a day

Discharge per tube well = 16 x 10,000 = 160,000 gal/day

<u>4.3.2 Surface water quantity.</u>

The possible source of surface water can be "Mir Wah" Canal passing through the town area. It is 60 miles in length, taken from the main feeder called "Khairpur

East Feeder" which is 13 miles in length and has a bed width of 82 feet at head works.[3] This feeder is a cut from Sukkur Barrage, at Sukkur, built on mighty Indus River.

No records for Mir wah canal are available, however records of the discharges for the Feeder for year 1965-66 are given in Table 4.6. The maximum closure period for the Canal is said to be 15 days.[1]

4.4. Water Quality Available

Regarding the quality of ground water supply, results of test analysis for samples taken from the two tube wells installed along the bank of Mir wah Canal, are given in Chapter I under Table 4.1.

A critical review of the contents of this Table in light of the standards as described in Section 4.1.2 for drinking waters clearly confirm the suitability of such waters for the purpose.

Regarding the quality of surface water, no results of any test for the samples taken from the Mir wah Canal are available, however results of a test analysis for water samples of Khirthar Canal near Jacobabad are Available. The original feeding source of this Canal is also Indus River and the soil conditions etc. are similar to those at Khairpur. These results, which can hold good for raw water quality of Mir wah Canal are given in Table 4.7.

T A B L E 4.6[11]

AVERAGE DISCHARGES FOR K.E. FEEDER AT SUKKUR BARRAGE HEAD WORKS DURING THE YEAR 1965-1966.

YEAR		JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER
1965	Average Q (Q* for January)	1,639	1,860	1,736	1,992	2,261	2,784	2,648	2,871	2,594	2,677	2,042	1,873
1965	No. of Days (Days** for January)	16	28	31	30	31	30	31	31	28	31	30	26
1966	Average Q (Q* for January)	1,446	1,329	1,569	2,145	2,289	2,436	2,641	2,765	2,662	2,453	2,387	2,062
1966	No. of Days (Days** for January)	21	28	31	30	31	30	31	31	28	31	30	26

* Average represents average discharge in cubic feet per second per day.

** Number of days feeder was flowing.

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TABLE 4.7[12]

TEST RESULTS OF WATER ANALYSIS OF A SAMPLE OF KHIRTHAR CANAL

CONSTITUENT	RESULT	REMARKS
Free Carbonic Acid	Absent	-
Chlorides (Nad)	48 mg/L	-
Nitrates	Present	Undesirable
Sulphurated Hydrogen	Absent	-
Nitrites	Absent	-
Lime	22 mg/L	-
Sulphates	110 mg/L	-
Total Solid Matter	265 mg/L	-
Temporary Hardness	118 mg/L	-
Permanent Hardness	48 mg/L	-
Total Hardness	166 mg/L	-
Free Ammonia	0.03 mg/L	-
Albumnoid ammonia	0.2 mg/L	High
Poisonous Metals: Iron, Lead, Copper etc.	Absent	
PH	7.8	
Reaction	Alkaline	
Colour	Colourless	
Taste	Tasteless	
Odour	Odourless	

Opinion: In the sample examined there is slight evidence of contamination from the chemical point of view. The water can be regarded as fit for consumption, after confirmation by a bacteriological examination.

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<u>N.B.</u>: Biological analysis could not be carried out.

4.5. Source Selection

In the above paragraphs the two available sources are discussed qualitatively and quantitatively. From either point of view it seems on the first hand to decide for either of the two. In general, ground waters are, however, more preferred than surface waters, both from economic and health point of view. However, decision regarding the adoption of any source for community water supply of Khairpur needs certain considerations, which are enlisted below.

4.5.1 Surface Water Considerations.

1. Complete treatment of such a supply will require heavy purification costs.
2. Since maximum canal closure period is 15 days as such a provision of 15 days storage is minimum to be provided.
3. Such a heavy storage will require very large storage tanks with greater surface area exposed to sun and thus will consequently result in heavy evaporation losses and in percolation losses, if not lined.
4. A change in bacterial quality particularly, may occur due to seasonal changes in large storage tanks.

4.5.2 Ground Water Considerations.

1. A study of area reveal that ground water quality particularly chloride content varies directly with the distance from the Canal.

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2. A review of Tables 4.7 and 1.5 follows that a change in chloride content from 48 mg/L to 67 mg/L has occured in a water from Canal and a tube well installed on the bank of the Canal.

3. Nearly all the tube wells except those installed on the bank of the Canal, which get direct seepage water from the Canal, have brackish water. This shows that if the total ultimate supply is based on tube wells, it is possible that during the maximum closure period of the Canal, when there is no direct seepage of canal water, a depletion in existing seepage water may occur thus resulting on further pumping, in replenishment with brackish water from distant. This sort of occurance, if present will completely upset the situation and will result in complete failure of the scheme.

4. During investigations carried by Government of W. Pak. "Water logging and Salinity Control" department, it has been found that underground waters are not completely brackish but this brackish or saline water, as it may be called, forms a separate zone extending upto a certain depth below ground. Below this zone there is a zone of good quality water. The water of the two zones are not inter-mixed, because of density difference, and move with different velocities one over the other.

This department as such has a programme scheduled to be completed within the coming 20 years, under which

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through the process of leaching the underground soils, thousands of shallow tube wells will be installed to withdraw this brackish water and dispose into a separate channel leading to its final disposal in sea. This scheme will also result in lowering the water table.

5. It is hoped that with the completion of the programme as detailed under para 4, it will be possible by the end of 20 years hereafter, to tap good quality water from under ground reservoirs, and one will not have to completely depend on Canal seepage waters.

4.5.3 Recommendations.

After a careful study of various factors affecting selection of community water supply source for Khairpur town following recommendations are put forth:

For the first phase of 20 years, it is recommended that supply should be based on both of the sources. Major portion, say about 60%, should, however, be met through tube well and rest from canal water. The canal water should be given the required treatment. The two waters should be mixed and chlorination for disinfection purposes should be given to the mixture in the rising main leading to city reservoir, or in clear water well. The existing supply from the two tube wells should be treated as part of the portion proposed to be supplied by tube wells.

It is also suggested that behaviour of the ground

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water quality, with respect to chloride content particularly, be observed through frequent sampling during the canal closure period. This will help in getting closer view of the effects of the canal closure and pumping over the concentration of chlorides. A direct check over pumping and thus an indirect check on possible contamination of the source from distant brackish water, can therefore, be possible to apply, at the moment, there is an abrupt or considerable change in the concentration. The observations will also give a clue to decide about the adoptability of source in future to work on independent or in collaboration with canal water.

It is also recommended that effects of "water logging and salinity control" programme in the area be observed during this first phase of the scheme running.

It is also recommended to make an extensive study of under ground reservoirs - porosity and permeability of soil and many other things required for the purpose. This will give a clear picture of adequacy and reliability of the source.

Recommendations for the final phase of 20 years are to be based on the results of various observations recommended to be taken during the first phase of scheme running.

During the first phase observations, if it is found that during canal closure no undesirable effects appear on continuous pumping, also if it is found that "water logging and salinity control" measures have been effective in the area, complete supply should be based on tube wells and the purification plant which has also bared its usual useful life should be abandoned. In other case, it is recommended that the extent to which pumping could be continued, without creating undesirable effects, should be determined and the additions for the final phase should be based on new proportions so determined.

However, it is hoped that "water logging and salinity control" programme is to result in its success and it will be possible to resort completely on tube well supplies.

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CHAPTER 5

WATER COLLECTION, PURIFICATION AND DISTRIBUTION

Once a proper source selection has been made the next step that follows is to find out the means and modes in which, the required quantity can be collected, given some sort of treatment to improve its quality if it needs so, and to distribute it among those who need it.

In the following, these three aspects viz. collection, purification and distribution will be dealt in separately.

5.1. Water Collection

Upon two sources viz. ground water and canal water has been decided to base the total water requirements for the community water supply of Khairpur.

The methods of collection of the supply from the two sources differ from each other.

Ground waters can be collected by constructing suitable type of wells and installing suitable type of pumps to draw the water from these.

Surface waters can be collected by constructing suitable in-take conduits or structures depending upon local conditions.

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The two methods will be dealt with, hereunder sub-sections separately.

5.1.1 Ground Water Collection.

From the previous studies it is found that because of the presence of unconsolidated acquifer and shallow water table, a bored tube well upto a depth of 100 feet below ground will have to be bored. The stratification of under-lying stratum as given on Figure 1.5 is based on the classification of soils as to particle size and is given in Table 5.1.

TABLE 5.1 (7)

CLASSIFICATION OF SOILS AS TO PARTICLE SIZE

MATERIAL	SIZE, INCHES
Gravel (stone)	Larger than 0.08
Very coarse sand	0.04 to 0.08
Coarse sand	0.02 to 0.04
Medium sand	0.01 to 0.02
Fine sand	0.005 to 0.010
Very fine sand	0.003 to 0.005
Silt and clay	Finer than 0.003

In the following paragraphs some aspects of tube well construction, completion etc. and installation details including of pumping equipment will briefly be given.

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A) Tube Well Construction.

Bored wells are constructed with hand-operated or power-driven earth augers. These augers are equipped with cutting blades at the bottom. The auger is removed from the bore and emptied when full of excavated material and process is repeated till desired depth is reached. When loose sands are encountered, or the boring reaches water table, which is in our case, a casing is required to lower to the bottom of hole and continue boring inside. A casing maintained to near the bottom of the hole checks caving problems.(13) Casing may be constructed of a corrosion resistant steel pipe. The entire length of casing may consist of individual pipe sections connected by threaded or welded joints. However, threaded joints are preferred. It is important to maintain vertical alignment otherwise great difficulties are encountered in withdrawing casing. To avoid interference with pump installation and operation, it is important to maintain proper alignment in a completed tube well.

B) Tube Well Completion.

The art of well completion aims to provide for ready entrance of ground water into the well with minimum resistance in and around the casing. In unconsolidated formations, which is our case, a casing with perforations is provided to serve the dual purposes of freely admitting water into the well and supporting the outside material.

The perforated casing or well screen as it is called should extend over the permeable depth of aquifer, which is 40 feet in our case as found from Figure 1.5. The rest of the section of the bore should contain a blank casing surrounded and sealed by puddled clay, cement grout or gravel packing. This will help in preventing vertical movement along the exterior of the casing.

Opening may be horizontal or vertical but generally a horizontal louvered opening gives a better control of unconsolidated materials than does a vertical slot.[13] Slots are tapered with the widest space at the inner surface to prevent particles from accumulating within the openings and blocking flow. The size of openings selected should be such as to pass about 70 per cent, or more, of the sand grains in the aquifer, where the uniformity coefficient is high.[8] The total area of the openings in a screen should be such as to maintain an entrance velocity less than necessary to carry the finest particles of sand that is to be excluded by the screen. In general, it should be less than about 0.125 to 0.2 fps.[8] Lifting velocities of sand grains, with a specific gravity of about 2.65, are given below in Table 5.2.

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TABLE 5.2[8]

LIFTING VELOCITIES OF WATER

DIAMETER OF GRAINS, MM	VELOCITY OF WATER, FPS.
Upto 0.25	0.0 - 0.10
0.25 - 0.50	0.12 - 0.22
0.50 - 1.00	0.25 - 0.33
1.00 - 2.00	0.37 - 0.56
2.00 - 4.00	0.60 - 2.60

However, it is recommended to refer to the screen manufacturers for recommending the most satisfactory slot size based on a grain size analysis of a given aquifer.

Materials used for screens include iron or steel, either black or galvanized and corrosion-resistent alloys. However, it is recommended to use silicon red brass which contains copper, silicon and zinc in a proportion of 83, 1, and 16 per cent respectively. This type is found to be commonly used in the area and is comparatively cheaper than other alloys of stainless steel or super nickel, etc.

A gravel screen or envelope surrounding the perforated portion of the casing is provided to increase the effective diameter of the well, act as strainer to keep fine material out of the well, and protect the casing from caving of surrounding formations.

In an unconsolidated formation, which is our case, a gravel pack well will have a greater specific capacity than one of the same diameter not surrounded by gravel.[13] Also to avoid a san-pumping well, a gravel pack is a necessity. A minimum of 6 inches thick pack with sizes ranging from sand upto 1/4 in. gravel is recommended.[13]

The location of strainer, blank casing, gravel pack and under-lying stratum for tube wells at Khairpur are shown in Figure 5.1.

C) <u>Tube Well Development.</u>

To obtain maximum well life, increase in specific capacity and prevent sanding, development of a new tube well is to follow its completion.

Development of tube well aims to the removal of fine sands. In a gravel pack well though it is not necessary yet it, of course, is beneficial.

Development is accomplished by pumping, surging, injection compressed air, back washing, and addition of carbon dioxide.

Development through pumping perhaps is the one, which is easier and most economic than the many other procedures. Here pumping is first started at lower discharge until the well water is clear and thereafter with successive increase in discharge and repetition the pumping is continued till the well capacity is reached. After the water clears at the maximum discharge, the pump should be

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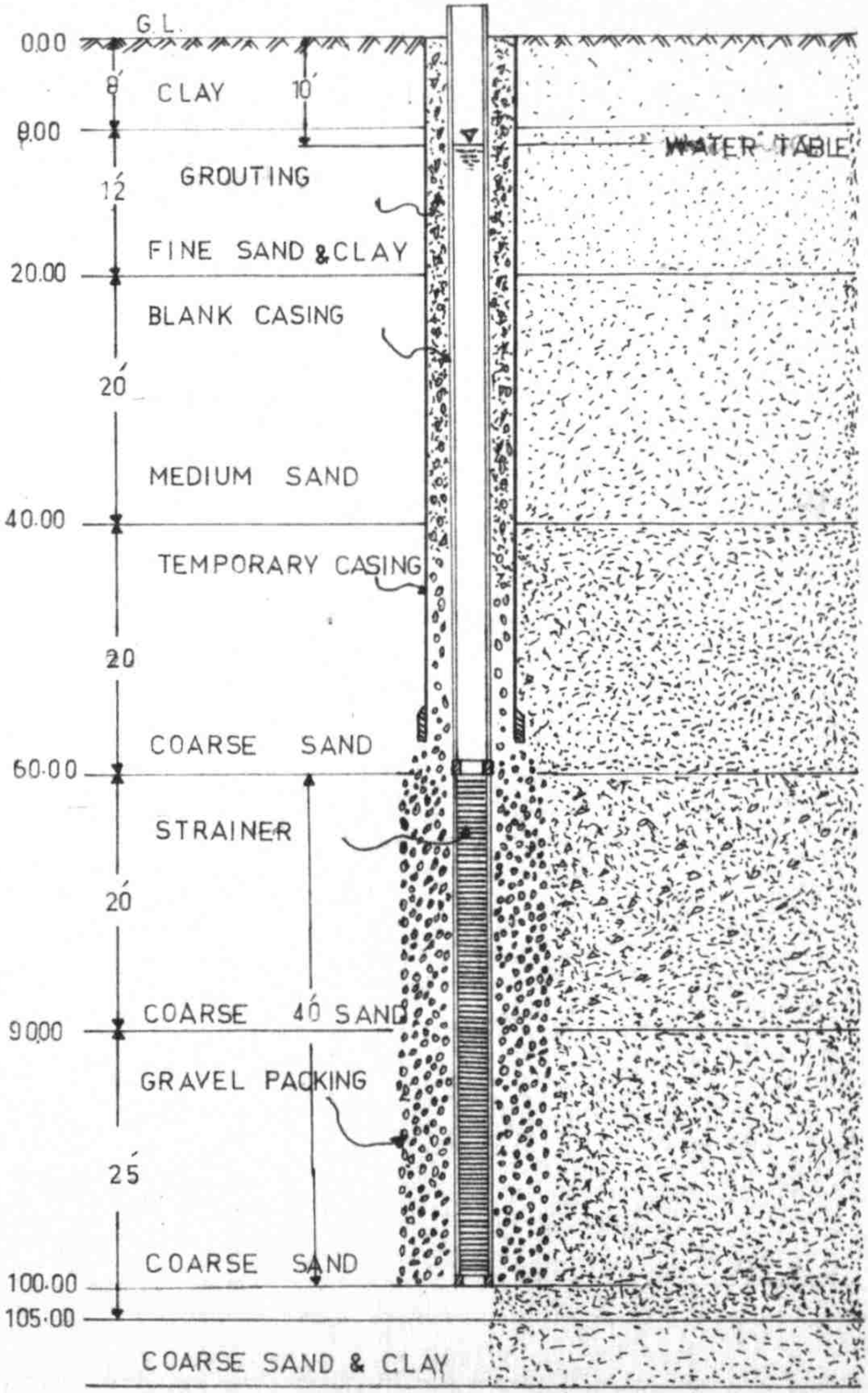

FIG 5.1	A COMPLETED WELL DIAGRAM	Scale 1cm = 5'

shut down and the water level in the well allowed to return to normal; then the entire process is repeated. This iregular pumping results in agitation of fine material surrounding well and is thus pumped out.

D) Interference of Wells.

If wells are placed close together, their radii of influence may over-lap and the wells may interfere with each other and reduce the quantity that may be pumped from each. This may lead to serious problems like depletion of underground reserves or drawing into the aquifer of water contaminated of salt or faecal matter on over pumping.

From the study of two experimental tube wells, which are installed 250 feet apart, it follows that there is no probability of such an over-lap if tube wells are located this much distance apart.

The tube wells will thus be located 250 feet apart.

E) Water Contribution.

In Section 4.3.1 on the basis of 16 hours operation of the plant the discharge from new tube wells and the existing tube wells was estimated as under.

Discharge from existing tube well = 160,000 gal/day.

Discharge from proposed tube well = 289,000 gal/day.

In section 4.5.3 it has also been proposed that for the first phase of 20 years, tube wells will be designed to contribute about 60% of the total supply.

The figures are calculated as under.

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a) <u>1st Phase.</u>

Total water consumption at the end of the period

= 2,205,000 gallons/day.

Discharge from two existing tube wells

= 2 x 160,000 = 320,000 gallons/day.

Adopting 4 new tube wells

Discharge from proposed tube wells

= 4 x 289,000 = 1,156,000 gallons/day.

∴ Total contribution from tube wells

= 1,156,000 + 320,000 = 1,476,000 gallons/day.

∴ Percentage of supply shared by tube wells

$= \frac{1,476,000 \times 100}{2,205,000}$ = approx. 67 per cent.

b) <u>Final Phase</u>

Total supply required = 4,185,000 gallons/day.

Assuming that full supply is being met through tube wells. Also it is assumed that the two tube wells of old water works are no more in use.

∴ Total tube wells required $= \frac{4,185,000}{289,000}$ = approx. 14

Provide 14 tube wells bored to a depth of 100 feet below ground level and equipped with 40 feet length of strainer of 12 inches dia. as shown in Figure 5.1 only 12 tube wells will be installed at new water works and 2 at old water works.

F) <u>Pumping Installations.</u>

Different types of pumps each satisfying certain

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defined conditions are used to draw water from well.

The type of pumps, suitable for the actual conditions of the supply system, from the tube well source to the surface reservoir, are the centrifugal pumps.

The centrifugal pumps are almost universally used in water works. Large and small quantities under different heads and at different distances can be supplied by these. Owing to their relatively high speed, they are lighter and smaller as compared to others. Transmission losses are minimized because of direct coupling with the driver. Smaller space, lower initial and maintenance costs are required. There are different types of centrifugal pumps: the axial andthe mixed flow. Of these two, there are the single and the multi-stage types.

In this report we recommend axial flow single stage centrifugal pumps.

G) Design of Pumping Units.

In order to select the proper units, certain definite specifications and data are supposed to be known.

The first factor to be known is: the capacity or quantity of water to be pumped. The quantity to be pumped from each well has been found to be 18,090 gallons per hour or 300 g.p.m.

The second important factor is the head against which this capacity of water is to be pumped. The total head includes many different components.

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a) The static discharge head.

b) The static suction lift.

c) The maximum draw down.

d) The head due to pipe line losses.

The figures required for the first three items can be directly measured but for the fourth the following computations are required :

The necessary requirements for the calculation of this head are :-

The length, dia and material of pipe line, type and number of fittings.

The material used in pipe lines will be cast iron. The discharge pipes from the individual tube wells will discharge into a manifold leading to the surface reservoir. The location of reservoir will be such that half of the total ultimate number of tube wells will be located on the left and half on the right. Thus the capacity of the manifold will be 7 x 300 = 2,100 gpm. From nomogram[7] the size of manifold is found as 12" with a velocity of 5.8 fps and head loss of 16 feet per 1000 feet.

Now the discharge per tube well = 300 gpm

From nomogram[7] the size required = 5 inches with a velocity of 4.8 fps and loss of head 32 ft. per 1000 ft.

$\therefore$ Size of suction pipe adopted = 5 inches

Size of discharge pipe adopted = 4 inches

Regarding the type and number of fittings, the following classification is given below :

a) Fittings through manifold:

Type	Size	No. of Units
90° elbow	12 in.	7
45° elbow	12 in.	2
Gate valve	12 in.	1
Check valve	12 in.	1

Note: Connecting tee is considered as 90° elbow.

b) Fittings through each pump:

Type	Size	No. of Units
Foot valve	5 in.	1
Check valve	4 in.	1
Gate valve	4 in.	1
90° elbow	5 in.	2

After the various types and number of fittings have been specified, the next step then becomes to convert these into equivalent pipe lengths and add to the corresponding net lengths of pipes and calculate loss of head for flows through the new sum length.

a) Manifold:

Net length = 250 x 5 + 100 + 125 = 1250 + 225 = 1475 ft.

Equivalent length:

Type of fitting	Total equivalent length[14]
Seven 90° elbows	7 x 32 = 224
Two 45° elbows	2 x 15 = 30
One Gate valve	1 x 7 = 7
	261

$\therefore$ Total length = 261 + 1475 = 1736 feet.

$\therefore$ Loss of head = $\frac{16 \times 1736}{1000}$ = 27.8 feet.

b) Suction and discharge line of each pump.

Net length:

Discharge line = 100 feet

Suction line = 150 feet (one pump house in between two tube wells)

Equivalent length:

Type of fitting	Total equivalent length[15]
One foot valve 5 in.	1 x 76 = 76
Two 90° elbows 5 in.	2 x 13.5 = 27
One check valve 4 in.	1 x 42.3 = 42.3
One gate valve 4 in.	1 x 2.3 = 2.3

$\therefore$ Total length of 5 in. dia pipe = 76 + 27 + 150 = 253 ft.

" " " 4 in. " " = 42.3 + 2.3 + 100

= 144.6

$\therefore$ Loss of head in 5 in. dia pipe = $\frac{32 \times 253}{1000}$ = 8.00 ft.

Loss of head in 4 in. dia pipe = $\frac{90 \times 144.6}{1000}$ = 13.00 ft.

$\therefore$ Total loss of head to be sustained by pump

= 27.8 + 8.00 + 13.00 = 48.80 feet.

<u>Pumping Head</u>:

a) Suction head:

The pumps will be installed 5 feet below ground. The depth on average of the water table is 10 feet below ground level and there are fluctuations of $\pm$ 2 ft. due to Canal opening and closure.

$\therefore$ Maximum suction lift = 7 feet.

b) Discharge head:

The pumps will be required to discharge water into clear water well, the inlet of which will be 2 feet below ground level.

∴ Discharge head = 3.0 feet.

c) Draw down head:

For the maximum discharge, the water surface stabilizes at 12 feet below static water table, as found during experimentation.

∴ Draw down head = 12 feet

∴ Total suction will be below 26 feet.

d) Friction head:

Friction head as found is = 48.8

∴ Total dynamic pumping head required

= 7.0 + 3.0 + 12.0 + 48.8 = 70.8 say 71 feet

<u>Note</u>: No pressure head will be required as the discharge will be at atmospheric pressure.

<u>R.P.M.:</u>

On the gallons per minute basis specific speeds for volute centrifugal pumps range from 500 to 5,000 (16)

∴ Taking specific speed as 500

$$N_s = \frac{N\sqrt{Q}}{h^{3/4}} = \frac{N\sqrt{300}}{71^{3/4}} = 500$$

$$\therefore N = \frac{75^{3/4} \times 500}{\sqrt{300}} = 745 \text{ R.P.M. say } 750 \text{ R.P.M.}$$

<u>Prime mover:</u>

Number of drivers like diesel, steam and gasoline engines and electric motors, are used.

However, because of lower initial and operation cost, occupancy of smaller space, light in weight and easy and efficient perfromance it is recommended to use electric motors as drivers for the proposed pumps. Comparative cost of power is lower in the area.

$$\text{Horse Power of motor} = \frac{GH}{3960 \times E}$$

where, G, discharge in gpm

H, head in feet

E, efficiency of motor taken 60%

$$\therefore \text{H.P.} = \frac{300 \times 71 \times 100}{3960 \times 60} = 9.2 \text{ say } 10 \text{ H.P.}$$

Selection:

Axial flow, volute centrifugal pumps capable of giving a discharge of 300 gpm against a head of 71 feet with the R.P.M. rating of 750 and 5 x 4 suction and discharge pipes coupled with a single phase A.C. motor of 10 H.P. are required to be installed over each tube wells.

Number of standbys will be taken as 50% of total installation. Complete specifications and details of pumps and motors will be supplied by the local manufacturers like BECO and Jaweed pumps and many others, on furnishing them the above requirements details.

Therefore, a total of 6 sets will be required for the 1st phase and 2 sets will be required by the end of the design period ie. year 2007.

5.1.2 Surface water Collection.

No elaborate intake structures will be required for water collection from Mir wah Canal. A sheeted and co-

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vered trench in the Canal bank parallel to the flow, having a cross section sufficiently large to give low depositing velocity, and thus permitting grit to settle down before reaching the pipe, taken from the side of the channel to the storage tank at water works, will be adopted. By opening the flushing conduit at suitable times strong flows created can remove the sediments. Sediments can also be cleared manually by removing the movable cover over the trench. A coarse screen is also provided on the water side to check the entrance of floating materials. Diagramatically it is shown in Figure 5.2.

There are certain general basic considerations in selecting the location etc. of an intake, these are enumerated below:

a) The intake should be located to obtain the best quality of water and free of any contamination.

b) The intake, if it is a pipe intake, should be placed some distance off shore to obtain adequate submergence and avoid entrance of turbid water from surface wash along bank.

c) Intakes must have a stable bottom.

d) Intakes should be designed to draw absolute maximum demand.

e) Velocities in pipes should be sufficient enough to prevent sediment deposits.

From the study of Mir wah Canal, it has been found that the fluctuation in water level are small, there are no serious erosion problems, no high waves and currents,

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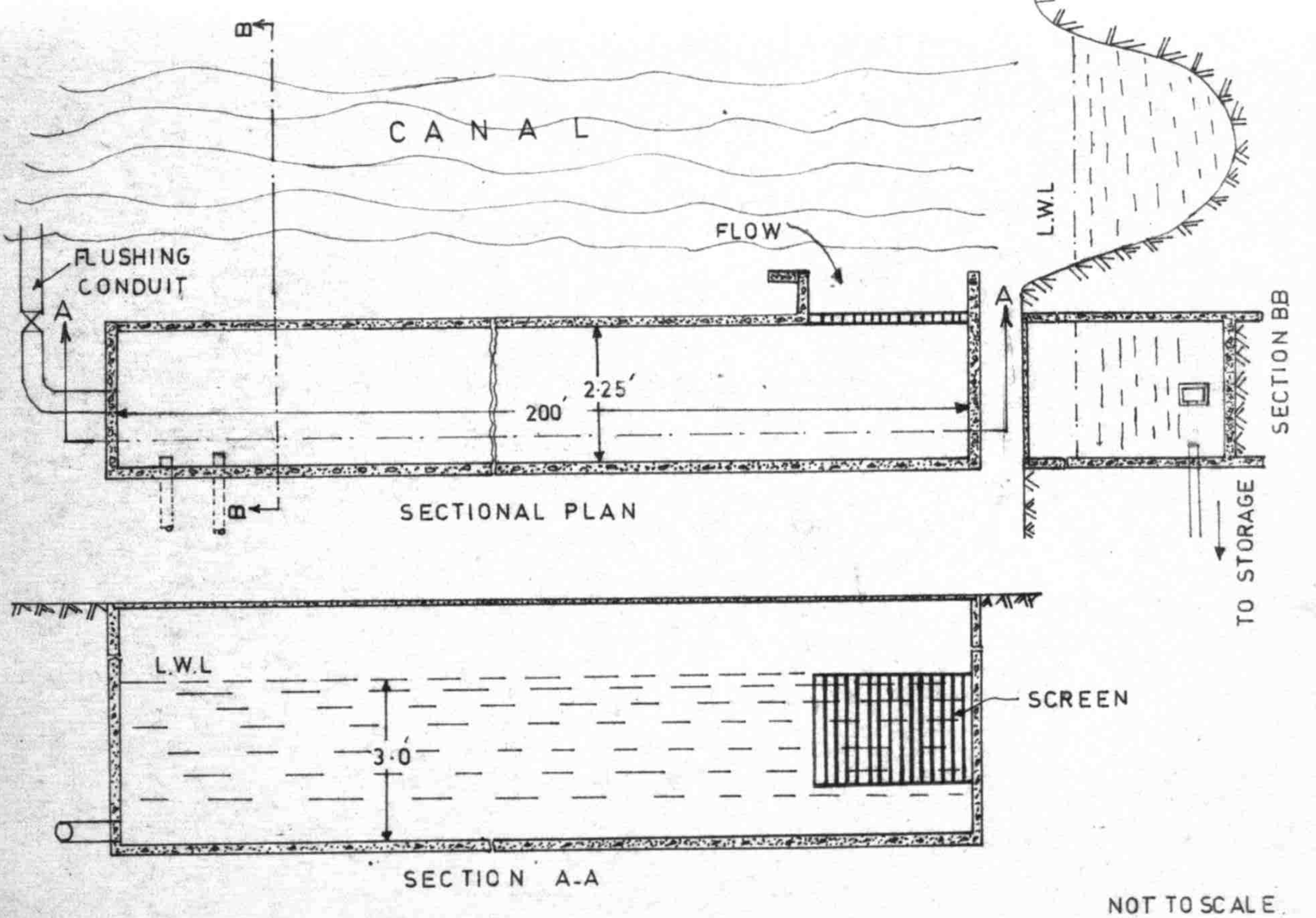

FIG. 5.2 CANAL INTAKE.

amount of debris and floating material is low and there is no possibility of any contamination at the proposed location.

In the proposed intake, the channel will be designed as an ordinary closed rectangular channel. The design criteria adopted will be those used in the design of grit channel. Such design criteria are given as:

a) The velocity through the bar screens should not exceed 30 fpm.[8]

b) The opening between the bars should not be less than 1 to 2 in ches.

c) Velocity in suction pipe of 2 to 3 fps is recommended[7]

d) For conduits upto 12 inches in diamter, standard bell and spigot cast iron pipes with flexible joints are used[8]

e) A horizontal velocity of 0.5 to 1.2 fps is recommended[17]

f) The settling velocity of 0.2 mm particle is about 0.075 fps.[17] Steel recommends 0.1 fps.[7]

g) Volume of grit accumulation @ 4 cft/million gallon of flow.

A) Design of intake

Channel - Quantity of flow = 729,000 gallons/day = 1.12 cfs.

$\therefore$ Flow required through intake for storage and daily requirement = 2 x 1.12 = 2.24 cfs.

$Q = A \times V$ or $A = \frac{Q}{V}$, taking $V = 0.5$ fps.

Area $= \frac{2.24}{0.5} = 5.00$ sq. feet.

Adopting working depth as 2 ft.

width of channel will be $\frac{5.0}{2} = 2.25$

Taking vertical settling velocity = 0.1 fps.

Time required for particles to settle

$$= \frac{2}{0.1} = 20 \text{ ft.}$$

$\therefore$ Length of channel required = 20 feet.

Volume of grit = $\frac{729000 \times 4 \times 2}{10^6} = 5.8$ cft./day.

Taking 7 days clearance

Volume of accumulation = 5.8 x 7 = 40.6 cft.

$\therefore$ Depth of grit = $\frac{40.6}{20 \times 2.25} = 0.9$ feet say 1.0 ft.

$\therefore$ Required depth will be 2.0 + 1.0 = 3.0

The bottom of channel will be 3.0 ft. below L.W.L. However, the channel will be constructed from ground level and will be of such a depth such that its bed is 3.0 ft. below L.W.L.

Screens:

Bar screens of 3/8" thick steel bars placed 1" apart are recommended.

Area of flow per rack = 2/12 x 2.0 = 0.34 sq.ft.

For velocity of 0.5 fps.

Area of flow required = $\frac{2.24}{0.5} = 4.48$ sq.ft.

$\therefore$ No. of racks required = $\frac{4.48}{0.34} = 13.2$ say 13 racks.

Taking 3 width of screen

No. of bars required = $\frac{3.0}{0.094} - 1 = 32 - 1 = 31$ bars.

Top of screen will be at L.W.L. in canal and

.../88.

it will be 1.0 ft. above the bottom of channel.

Suction pipes:

Adopting of two pipes

Discharge per pipe = $\frac{2.24}{2}$ = 1.12 cfs.

From monogram (7) 9 inches pipes giving a velocity of 2.6 ft/sec. to be laid at a slope of 5 feet per 1000 feet are required. The pipes will be supported on masonary pillars at the desired slope, so that gravity flow is obtained.

5.2. Water Purification

Once the required quantity of water has reached the boundaries of the water works, the next step becomes to check for its quality and resort to some sort of purification, if it needs so, before it is supplied to the consumer. Thus by purification is meant the removal of some or all of the impurities, whether suspended, dissolved or colloidal; and the processes are classified into its major classes according to its use as shown below:

a) The treatment carried to improve the usability of the water for particular industrial use.

b) The treatment carried out to make the water suitable for general domestic supply.

In general, the municipalities resort tothe first type of purification process to render it fit for general domestic use and supply the same quality of water to any industry, if it needs so, and let the industry carry out further desired purification process to improve the usability of the water for their particular use, same is true in our case.

Thus the object of purification is to render the water wholesome and pleasant to drink, ie. to remove bacteria causing disease, turbidity, colour, taste and odour.

The treatment usually comprises sedimentation, filtration, and chlorination. In highly turbid waters and where an improvement in the filterability of filter and shorter retention time in sedimentation tank is, of course, an indirect intention, sedimentation is preceded by congulation and floculation.

Aeration is some times used for waters having sufficient content of iron and manganese and also to remove the odours caused by the pressure of hydrogen sulphide.

Softening is not usually adopted in municipal water supplies except when the waters are excessively hard. Softening occupies the most prominent place in industrial purification processes.

In the following sub-sections a brief introduction of the different processes used in water purification will be given and those suitable to our conditions will be chosen.

5.2.1 Methods of Purification.

A) <u>Screens</u>:

.../90.

These are meant to remove the floating material which otherwise may clog the plant or damage the pumps and valves. Thus the purpose of screens is to protect the plant rather than to remove impurities from the water; strainers, however, are quite commonly used to improve the quality of water.

The screens may be fixed, movable or moving. A number of equally spaced, usually 1/4 - 1½ in., steel bars or punched steel plates constitute fixed screen and can be cleaned by means of suitable rake or broom. A movable screens are similar in construction except that these are removed during cleaning as such these are duplicated. Moving screens are commonly bands, drums or discs carrying the straining medium and moved continuously by electric units. During operation these are continuously cleaned by brushes or water jets.

In water supply, usually fixed or movable screens are used at intakes from lakes and rivers to restrict the entrance of floating and inorganic suspended material to the plant.

B) Sedimentation.

A considerable improvement in the quality of raw waters is effected through a certain period of retention in sedimentation tanks.

The oldest used and simplest method of removing suspended impurities from water is by plain sedimentation or subsidence. The water is allowed to stand quiescent or move very slowly through natural or artificial basins until these impurities settle to the bottom and relatively clear water is drawn from the top.

The degree of removal varies in direct proportion to length of retention, size of particles, and the temperature of the water. The higher the temperature the faster the rate of settling.[18]

Sedimentation tanks are usually preceded by "grit chambers" large enough to provide a detention period of few minutes to allow the settling of thelarger particles that settle quickly.

Sedimentation basins may be operated continuously or intermittently. One unit suffices in continuous operation whereas a minimum of two are required for otherwise.

Chemicals are not used in waters meant for plain sedimentation. Such process of settling is given a separate name, "Sedimentation and coagulation", or simply coagulation.

The turbidity removal with usual detention of few hours as achieved by plain sedimentation is said to be 60-70 per cent with bacteria removal in proportion.[18] 80 per cent may be achieved under favourable conditions.[18]

.../92.

Artificial sedimentation basins are generally made of concrete with the bottom sloping towards gutters and drains for washing out the sediment, which is done periodically. Some times mechanical clarifiers, which remove the sediments continuously, are used where conditions permit.

A large storage reservoir, required at some of the water works for storage of water for longer periods of draught serve the dual purpose of storage and plain sedimentation at such places.

Large storage reservoirs with retention of several days have been practised without subsequent filtration.[19]

The sedimentation is generally practised to reduce the subsequent load on filter and thus increase their run and working efficiency. Sedimentation has also been applied before slow sand filtration to reduce the turbidity to less than 5 p.p.m.[19]

C) <u>Coagulation and Flocculation.</u>

The principal draw back to purifying by plain sedimentation is the size of the settling basins required, unless they are needed anyway for storage reservoirs.

An equal degree of clarification can be obtained with a much shorter retention, i.e. through smaller basins, if a coagulating chemical is fed to the water as it enters.[18]

The coagulants usually used are salts of iron and aluminium which react with alkaline salts naturally present or added with the coagulant, to form a gelatinuos precipitate, which settles down with relative rapidity, carrying suspended matter in the water.

Coagulation proceeds in three steps:-

a) As the coagulant dissolves, positive irons from the coagulant become available to neutralize the negative charges on the particles of turbidity, including colloidal clay and colour. During this process, a rapid and intimate mixing is required for greater efficiency. Mixture thus is agitated well.

b) After the first action has taken place it seems that flocs formed are still too small to settle by gravity. Positive charges are still retained by the coagulants perhaps due to adsorption from water. Thus the resulting positive flocs still have the affinity to neutralize negative ions. This is achieved by slow stirring so called conditioning of flocculation. During this small flocs may agglomerate and grow in size until they are in a proper condition for sedimentation.

c) During the third phase, surface adsorption of particles, takes place on the largest surfaces area provided by the floc particles. Entrapment of other, including bacteria, also takes place.

Sedimentation preceded by coagulation and floculation aims to the maximum removal of flocculated or unflocculated suspended matter so as to reduce the load

.../94.

on rapid sand filter and increase the filter run.

D) Filtration.

Through experience it has been found, that the suspended and colloidal matter is partially removed, chemical characteristics of water are changed, and the number of bacteria is materially reduced, if water is passed through a layer of sand. This process technically is called "Filtration", and the unit responsible, as filter.

The two kinds of filters are generally referred to as "slow sand filters" and "rapid sand filters". Filtration media in both cases is sand but they differ in their rate of filtration and thus are claasified as such.

Slow sand filters date back to about 1830, and are commonly regarded as the English type of filter, whereas rapid sand filters is an American introduction around the year 1890.

Chemicals are used with rapid type to assist clarification before filtration. A comparison of operational and constructional features of the two types is delineated in Table 5.3, given below.

TABLE 5.3

GENERAL OPERATION AND CONSTRUCTION FEATURES OF SLOW AND RAPID SAND FILTERS.

FEATURES	SLOW SAND FILTERS	RAPID SAND FILTERS
Rate of filtration	1 to 4 mgad	100 to 125 mgad.

.../95.

Time required for cleaning	3 days in a month	10 minutes in a day
Turbidity for feed water	Not exceeding 40 mg/L (8)	Not exceeding 10 mg/L.(7),(19)
Size of bed	Large, ½ acre	Small,1/100 to 1/10 acre.
Depth of bed	12 in. gravel, 42 in sand.	18 in gravel, 30 in sand.
Grains size distribution	Unstratified	Stratified, with smallest grains at top and coarses at bottom.
Loss of head	0.2 ft. initial to 4 ft. final	1 ft. initial to 9 ft. final.
Method of cleaning	Scraping off surface and washing the sand removed	Back washing or scour by mechanical rakes.
Wash water required	0.2 to 0.6% of water filtered	1 to 5% of water filtered.
Preparatory treatment	Generally confined to aeration, could include floculation and sedimentation.	Floculation and sedimentation are common.
Cost of Construction	Higher	Lower
Cost of operation	Lower	Higher
Penetration of suspended matter	Superficial	Deep
Colour removal	Less efficient	More efficient.

E) <u>Aeration.</u>

Many water supplies do not need to be aerated. Dissolved gases resulting from the decomposition of organic matter in surface supplies producing problems of tastes and odours can be reduced through aeration.

Ground water supplies usually contain carbon dioxide, and sometimes hydrogen sulphide, iron, and manganese. Aeration reduces the carbon dioxide and hydrogen sulphide and in most cases, oxidizes the iron and manganese, causing them to precipitate. Manganese removal usually requires a use of lime to increase pH to 9.4.[20]

Aeration is accomplished by various means. It may be done by allowing the water to run over the steps, by trickling through trays filled with coke, spraying from spray nozzles, or by forced draft aerators, or by a variety of other means.

F) Chlorination.

Water that has been treated efficiently by the various processes of treatment, should be nearly free of harmful bacteria. However, complete removal of pathogenic bacteria is essential and to accomplish this chlorination is perhaps the most effective, cheapest, and the generally used disinfection method.

How chlorine kills the bacteria, is subject of considerable controversy. Some believe that the nascent oxygen produced by the reaction of chlorine with water, as shown below, is responsible for the destruction.

$$H_2O + Cl_2 \rightleftarrows HCl + HOCl$$

$$HOCl \rightarrow HCl + O$$

However, certain facts such as Potassium permanganate, which is a more oxidizing agent than chlorine, is less effective as a germicide, and chloramines which has little if any, oxidizing power, is ver effective as a disinfectant, require further reasoning than oxidation alone.

A plausible explanation is the theory that chlorine units, at least in part, with the cell structure of the organism to form chloro-products that act as toxic poisons to these organisms.[20]

Chlorination can be accomplished by either using free chlorine as gas, or as combined chlorine in compounds like chloramines, hypo chlrites which contain certain percentage of chlorine. However, use of chlorine gas is said to be more economical and effective. Liquid chlorine supplied in steel cylinders and fed through chlorinator almost completely has superseded the chlorine compounds for general use.[20]

Chlorination, if correctly used, undoubtedly provides the surest line of defence against bacteria. It is perhaps the most powerful weapon available against bacteria and can be applied as a final process capable of correcting any re-pollution on the works or deficiences in previous management, in any case it should never be omitted.

5.2.2 Choice of Method.

After a careful and critical review of the local conditions, quality of supplies, economical aspect, various degrees of purification, and purpose served by component units, it is decided and recommended for the following component units to comprise a water treatment plant for Khairpur.

Water testing results of the water samples from tube wells as given in Table 1.5. clearly demonstrate that no special treatment other than chlorination, is required for the source. There are no metals like iron, manganese etc. present in water, and the water does not have high content of chlorides, which are the usual cases with ground water supplies. Nothing is known about the biological quality of water. As such and even otherwise to safe-guard against presence of harmful bacteria particularly pathogenic, disinfection of the supply can not be omitted.

It is therefore recommended to chlorinate the supply in conjunction with the surface water supply in the clear water well where the two supplies are mixed and collected for futher pumping to city reservoir.

Water testing results of the raw water sample for canal as given in Table 4.7, also demonstrates that no special treatment other than sedimentation and filtration, for the removal of suspended and colloidal matter, and of course, chlorination, for disinfection purposes, are the only required treatment processes to be used.

As pointed out earlier that at least 15 days storage, during the canal closure period, is required, as such the storage reservoir provided will serve the dual purpose of storage-cum-sedimentation.

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Coagulation and floculation are not recommended because of the long retention time of the storage reservoir: By this time, it is pre-supposed that nearly all the settle-able and even colloidal solids must have settled, which is the purpose of coagulation. It is also learnt that raw water itself is less turbid.

Water could be directly taken from such storage for distribution after chlorination andthus avoiding even filtration as recommended by some of the authors (pointed out earlier) however, to safe-guard against some balance turbidity and biological growths in storage etc. it has been decided to include filtration.

Rapid sand filters have been adopted for the obvious reasons of better quality effluent, efficiency, size and better adaptability to variations in the influent. Slow sand filters are everywhere being replaced by rapid sand filters because of the greater success in operation, that they have achieved.

5.3. Water Distribution and Pumping

There is an existing complete distribution net work, completed under the new proposed scheme.

In the following some of the points which are usually taken care of, in construction, operation, and design of a distribution system are delineated below. The existing distribution should be checked in the light of such criteria as given below and necessary corrections should be applied wherever feasible. However, actual design of system will not be tackled in this report.

Regarding the city reservoirs which forms a component part of a distribution system, it is to say that there is one service reservoir of 30,000 gallons capacity located at the old water works. An additional reservoir of 100,000 gallons capacity is also being constructed near the city Park. The height of the new reservoir will be 50 feet whereas the height of the old reservoir is 40 ft. These reservoirs will be used as balancing reservoirs and water from the plant will be pumped at average demand rate such that during the low consumption they will act for storing water and vice versa.

A complete lay-out of existing and proposed distribution system is shown in Drawing No. 3.

5.3.1 Distribution System.

Capacity:

The hydrautic capacity of the system should be sufficient to meet the ultimate maximum demand of the community except for major industrial use.

Pressure:

During heavy draught particularly fire, the pressure should not be below 20 psi.[7]

Material:

Number of different materials are used like cast iron, PVC plastic pipe, asbestos cement pipe, steel pipe, R.C.C. pipe and galvanized iron pipe.

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For better results, longer life, high pressures cast iron is better, which of course has been used. For sub-mains and distributories plastic pipes, which are getting into a greater use, should be used. However C.I. pipes and plastic pipes are used. For service pipes G.I. or plastic pipes should be used. G.I. pipes are usually used in the area.

System:

Dead ends are to be avoided and a complete loop system is to be formed. This system has a number of advantages as: it gives uniform pressure and isolation of any part under repair without causing any hinderance to rest of the parts. Dead end system has a major disadvantage of causing taste and odour troubles due to stagnant water at dead ends. Thus in the whole of the system, whether it is a service pipe, distribution pipe or main pipe the loop theory is to be applied. System in the area is however based on closed loop system.

Sizes:

Minimum size of mains should be 6 in. dia.

Minimum size of distribution pipe should be 3 in.dia.

Minimum size of service pipe should be ½ in. dia. and maximum 1½ in. dia.

Minimum size of 6 in. dia. in mains, 3 in. dia. in sub-mains and distributories and ½ in. dia. in service pipes is used in the area.

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Cover:

Minimum cover over the mains should be 3.5 feet whereas over distribution mains 2.5 to 3.0 feet. A minimum of 3 feet cover has been provided.

Valves:

Sluice valves should be provided at the rate of 2 on every tee junction and 3 on every cross junction. Air valves, if needed, should be provided at summits or where there is rapid change in ground slope. Location of sluice valves is shown in Drawing No. 3.

Fire Hydrants:

These should be provided at all crossings of main roads, if possible, and at the rate of one per 1000 sq.ft. Radius minimum. Fire hydrant should be 3 in. dia. Sufficient fire hydrants are provided in the area these are shown in Drawing No. 3.

Velocity:

Maximum velocity of flow should be 3-4 ft/sec.[7]

Life:

The system is supposed to work efficiently upto the end of year 2007.

5.3.2 Design of high lift pumps.

Pumps will be required to lift water from the clear water well to the city distribution system.

The head delivered by the pumps is to be sufficient to give a minimum residual pressure of 20 p.s.i. at all points as required during fire flows, this residual pressure of 20 p.s.i. or 46 feet of water, if maintained, will be sufficient to supply water upto three stories with residual pressure, at the tap of the top building, as 5 p.s.i.

In order to ensure such a residual pressure in a system; it is essential to locate the critical point in the system.

By the critical point is meant, a point which accounts for the maximum head loss due to friction for its alloted flow, in a system. In an area of flat topography and a grid network system it is usually the farthest point in the system.

Regarding the location of a critical point in the city water distribution system of Khairpur, in absence of complete detailed computations of flows and corresponding head losses occuring in individual lines of the system, it is difficult to locate exactly such a point.

However, under the circumstances, it will be reasonable to consider the maximum water level in the elevated reservoir to be the point, to which pumps should be able to supply water, under the conditions when there is practically no flow in the branches such that full flow is taken by the existing longest route to the reservoir.

The designed height of 50 feet for the base of elevated reservoir above ground level is in fact supposed to be arrived at after a due consideration of maintaining the desired pressure at the critical point. As such, our assuming in no way seems to be misleading. Design of pumping units will be as under:

Discharge:

Average daily requirements for the present	= 445,000 g.p.d.
	= 463 g.p.m.
Average daily requirements by the end of 1st phase i.e. by the year 1987	= 2205,000 g.p.d.
	= 1530 g.p.m.
Average daily ultimate requirements i.e. by the year 2007	= 4185,000 g.p.d.
	= 2900 g.p.m.

Pumps will be designed for the average flows.

Pumping head:

Total head required for the pumps comprises of: the suction lift, the static lift, loss of head due to friction, in suction and delivery pipes of pumps, delivery main, and miscellaneous pump losses.

a) Suction lift is taken = 5 feet (Pumps will be installed 5 ft. below ground level)

b) Static lift comprises of the following:

i) Difference of Elevation = 173.0 - 167.0 = 6.0 feet

ii) Height of tank base above ground =50.0 feet

iii) Height of max. water level above base =15.0 feet

∴ Total static lift =71.0 feet

c) Loss of head due to friction comprises of the following:

1. Loss of head in suction and delivery pipes of pump = 5.0 feet

2. Pump and other miscellaneous losses = 5.0 feet

3. Loss of head in delivery main which comprises of the following:

i) 12 inches dia. line

length of line = 1300.0 feet

Equivalent length for gate valve[14] = 1 x 7

= 7.0 feet

Equivalent length for long sweep elbows[14]

= 2 x 15 =30.0 feet

Equivalent length for Tee[14] = 1x60 =60.0 feet

∴ Total length of 12 in.dia. line =

1300.0 + 7.0 + 30.0 + 60.0 = 1397.0 feet

From monogram[7] loss of head for ultimate discharge of 2900 g.p.m. in a 12 in.dia. pipe is

=30 feet

Total loss of head in 12 in.dia.= $\frac{30 \times 1397}{1000}$

= 41.9 feet

ii) 10 in. dia line

Length of line upto the base of the reservoir = 1015.0 feet

Equivalent length for gate valves[14]

= 2 x 6 = 12.0 feet

Equivalent length for tee[14] 1x50 = 50.0 feet

Equivalent length for long sweep elbows[14] = 1 x 13 = 13.0 feet

Equivalent length for standard elbow[14] = 1 x 25 = 25.0 feet

∴ Total length of 10 in.dia.line

= 1015.0 + 12.0 + 50.0 + 13.0 + 25.0

= 1115.0 feet

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From nomogram(7) loss of head for ultimate flow

= 65.0 feet per 1000.0 feet

∴ Total loss of head in 10 in.dia line

$$= \frac{65.0 \times 1115}{1000} = 72.5 \text{ feet}$$

= Total loss due to friction = 5.0 + 5.0 + 41.9 + 72.5 = 124.4 feet

∴ Total head required for the pump

= 5.0 + 71.0 + 124.4 = 200.4 feet say 200.0 feet

The pumping head calculated as above is the maximum head against which a pump will be required to pump during the ultimate required flows in the delivery main. However, during early years of working the head required for pumping through the same lines will be less.

Among the components constituting total pumping head figures for suction lift, static lift, loss of head in pump suction and delivery, and other miscellaneous losses are practically constant, except the loss of head, due to friction in delivery main, which varies in direct proportion to the amount of flow passing through it.

Capacity head curve:

The sum total for the constant heads, which is 86.0 feet, will here be taken as static head, to which different losses of head due to friction in the delivery main for different discharges will be added in the calculation of capacity - head curve required in the selection of the pumps.

For Q = 400 g.p.m.

From nomogram(7) h_f for 12 and 10 in. dia pipes comes to be 0.8 and 2.0 feet per 1,000 feet respectively

$\therefore$ Loss in 12 in. dia. line = $\frac{0.8 \times 1397}{1000}$ = 1.12 feet

Loss of head in 10 in. dia line = $\frac{2 \times 1115}{1000}$ = 2.23 feet

$\therefore$ Total head H = 86.0 + 2.23 + 1.12 = 89.35 feet

For Q = 800 g.p.m.

From nomogram h_f for 12 and 10 in.dia pipes comes to be 2.6 and 7.0 feet respectively.

$\therefore$ Loss in 12 in. dia. line = $\frac{2.6 \times 1397}{1000}$ = 3.6

Loss in 10 in. dia. line = $\frac{7 \times 1115}{1000}$ = 7.8 feet

$\therefore$ H = 86.0 + 3.6 + 7.8 =97.4 feet

For Q = 1500 g.p.m.

From nomogram h_f for 12 and 10 in.dia lines comes to be 9.5 and 21.0 feet respectively

$\therefore$ Loss of head in 12 in.dia. line = $\frac{9.5 \times 1397}{1000}$ = 13.2 feet

Loss of head in 10 in. dia. line =$\frac{21.0 \times 1115}{1000}$ = 23.3 feet

$\therefore$ H = 86.0 + 13.2 + 23.2 = 122.5 feet

For Q = 2000 g.p.m.

From nomogram h_f for 12 and 10 in.dia.lines comes to be 15.0 and 35.0 feet respectively

Loss of head in 12 in. dia line = $\frac{15 \times 1397}{1000}$ = 21.0 feet

Loss of head in 10 in. dia line = $\frac{35.0 \times 1115}{1000}$ = 38.8 feet

$\therefore$ H = 86.0 + 21.0 + 38.8 = 145.8 feet

For Q = 2500 g.p.m.

From nomogram h_f for 12 and 10 in.dia lines comes to be 21.0 and 54.0 feet respectively

$\therefore$ Loss of head in 12 in.dia line = $\frac{21.0 \times 1397}{1000}$ = 29.3 feet

Loss of head in 10 in.dia line = $\frac{54.0 \times 1115}{1000}$ = 60.2 feet

$\therefore$ H = 86.0 + 29.3 + 60.20 = 175.5 feet

For Q = 2900 g.p.m.

H = 200.0 feet

Capacity head curve for pressure line and the performance curves of the pumps selected are shown in Figure 5.3.

Selection of Pumps:

After a careful review of the various characteristic curves of the pumps as given by the different manufacturers it has been decided to adopt "Goulds" centrifugal pumps having following details:

For the present one such pump will be put in operation and one will be working as standbye. Discharge given by one pump will be 1100 g.p.m. against the head of 101 feet and 725 g.p.m. against a head of 200 feet.

As demand increases,a third pump will be put in operation such that two,when put in operation in parallel, give a discharge of 1975 g.p.m. against a head of 145 feet. Third in this case will be working as standbye.

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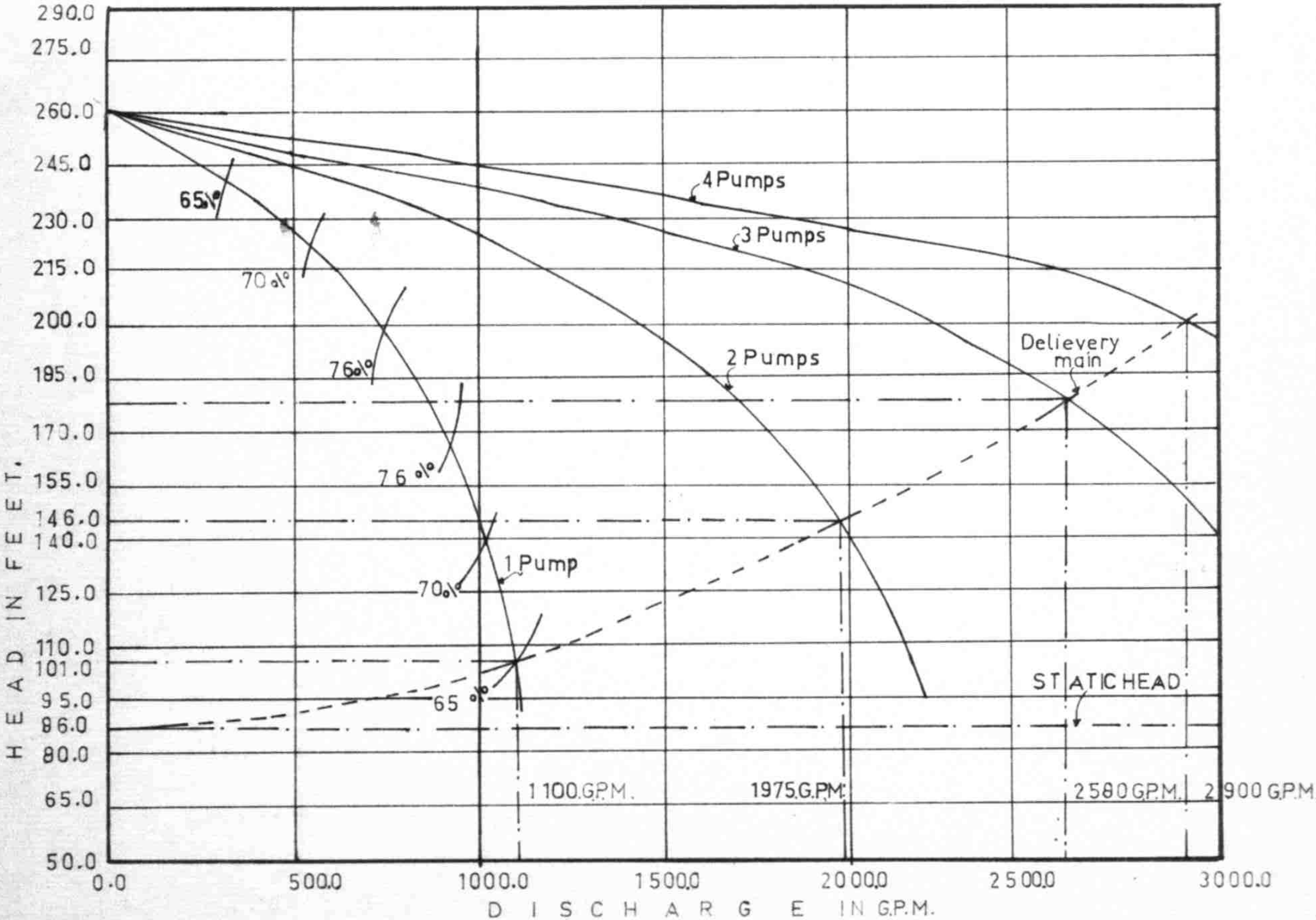

FIG. 5.3 CHARACTERISTIC CURVES OF PUMPS AND CAPACITY HEAD CURVE OF DELIEVERY MAIN

With further increase in demand two more pumps will be installed such that three when put in operation in parallel, give a discharge of 2590 g.p.m. against a head of 180 feet and the two will work as standbyes in this case.

Following the above described sequence, ultimately by the end of the year 2007 four pumps will be required to work in parallel and give a discharge of 2900 g.p.m. against the head of 200 feet. Number of standbyes pumps will, however, be maintainedto two.

Thus in all six pumps will be required.

Higher capacity pumps have been selected than required in the initial years with a view that for few years intermitent pumping will be done in a way that the reservoir nearly remains full during the demand hours.

Selection of pumps as discussed in this chapter with reference to capacity head curves and characteristic curves may be treated as a guide towards the selection of any type, size and capacity of the pumps. The particular pumps referred to here under may not be locally available as such proper selection of the pumps and thus the order with the manufacturer may be placed in accordance with the performance or characteristic curves etc. as supplied by him.

Following are details of the pump selected:[21]

Model : 3405

Size : 4 x $9\frac{1}{2}$

Pattern: 54135

Eye Area: 32.6 sq.in.

Imp. Dwg: 100 - 908

R.P.M.: 2850 CDS 1633

Efficiency: 65

H.P.: 55

Cycle speeds: 50/25

Gould Pumps Inc. Seneca Falls, New York

Bulletin 721.6

TABLE 5.4

CHARACTERISTICS OF THE PUMP

Q (G.P.M.)	H (FEET)	Q (G.P.M.)	H (FEET)
0	260	800	187
200	248	1000	140
400	235	1080	108
600	215	1100	101
725	200		

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CHAPTER 6

DESIGN OF WATER TREATMENT PLANT

As described earlier that storage, sedimentation, rapid sand filters, and chlorination will comprise the water treatment for Khairpur.

Intake, described and designed earlier is also among the plant components.

Water from the canal will flow under gravity, because of level difference, to the storage reservoir. Reservoir will be constructed such that the water level in it is 8 feet above ground level, this will result in direct feeding to the filter which will be located beside the tank and constructed on the ground level or so.

Thus neglecting the loss of head in between the tank outlet and filter inlet, a proper operation of the filter with a depth of 10 feet even can be effected. Water from the filters will be discharged into a clear water pool, which will be covered. This pool will also serve for water collection from tube wells.

Water from this pool will ultimately, through high lift pumps, be pumped into city reservoir and the system, for distribution among the consumers.

.../113.

Chlorination will be done in the clear water well to ensure adequate contact period. Complete lay-out of plant is shown in Drawing No. 4. Before proceeding with the design of various units, a set of design criteria will be described.

6.1. Design Criteria.

The extensive experience of various personnels working in the specific fields of their interest; usually inspires them to put it down in black and white for the benefit and guidance of those, who have yet to experience or have little experience of the practical aspects of the subject.

This is set forth in the form of certain criteria of designs and the designers are supposed to follow these as guide-lines. Designers are at liberty to mold these criteria, if the existing or fore assumed conditions or the past experience warrents them to do so.

In the following such design criteria will be outlined for the various units of the proposed treatment plant and proper figures will be chosen to base the design of the units.

A) <u>Sedimentation basin</u>

No specific design criteria are set for the storage reservoirs as it is purely requirement dependant. However, since in our case the storage reservoir is to serve the dual purpose of sedimentation and storage, as

.../114.

such certain general criteria for the tank, dimensions, inlet, and outlet design etc. are given below. Those feasible will be followed to insure proper operation.

a) For horizontal flow, depths normally range 10-18 ft.[8]

b) For rectangular tanks, usual ratio of length to width of about 2:1 is favoured[7]

c) Velocity of flow in sedimentation tanks should not exceed 1 fpm, tanks with velocities of 0.1 to 0.001 fpm are in existance[8]

d) For sludge collection, in rectangular tanks, a slope of 1% is recommended[4]

e) Large tanks should be partitioned and divided into compartments for ease in clearance of sediments.

f) Inlet baffels should be provided to reduce turbulance

g) overflow rates, for free falling out let weirs, should not exceed 50,000 gal. per lineal ft. per day[7]

h) Submerged weirs or submerged outlet ports in the end wall which discharge into an effluent conduit should be sufficient in number to prevent high velocities and be placed not over 2 ft. below the surface.

i) Basins without sludge-removal apparatus are cleaned by emptying the contents through a drain in the bottom and flushing with a hose supplying water under pressure.

j) To facilitate mud removal a minimum grade of 2% towards the outlet is recommended[7]

B) Rapid Sand Filters.

Rapid sand filters are of two types viz: Gravity and pressure sand filters. For our purpose we will be using rapid gravity sand filters.

Following design criteria is given:

a) Standard filter rate was 2 gal. per sq.ft. per min. but present practice favours 3-4 gal. per sq.ft. per minute.(7)

b) Ratio of filter length to width of about 1.25 to 1.33 is recommended(8)

c) Total depth i/c free board of 1 ft., common filter depth is 10 ft.(4)

d) Depth of sand varies between 24 to 30 inches(7)(8)

e) Effective size of sand usually used is 0.4 and 0.55 mm, with uniformity co-efficient not greater than 1.75 or less than 1.35(8)

f) Size of gravel in use vary from 1½ in. at the bottom to 1/8 in. at top.

g) In a perforated pipe under drain system, the ratio of perforation area to entire area of filter may be 0.2% or 0.3 sq. in. per sq.ft. of filter(8)(7)

h) Spacing of laterals may be as great as 12 inches.(8)(7)

i) The ratio of area of perforation to the area of lateral should not exceed 0.5 for ½ in.dia. perforations and 0.25 for 1/4 in. dia. perforations.

j) For 1/4 in. perforations spacing along laterals may be 3 in. and for ½ in. perforation may be taken as 8 in.[8][7]

k) Ratio of length of lateral to its dia. should not exceed 60[8][7]

l) The area of manifold should be 1.75 to 2.00 times the sum of X-sectional areas of the laterals.

m) Maximum initial loss of head allowed is 4 to 6 in. and final 8 to 10 ft.[8]

n) Spacing of troughs preferably should not exceed 6 feet.

o) Rate of wash water application is 2 cu.ft. per minute per sq. ft dt. filter surface, this is equivalent to a rise of 24 in. per minute.[8]

p) Capacity of wash water storage tank should be sufficient to give a normal wash to two filters for a period of 5 to 6 minutes.

q) Height of wash water trough from sand surface should be equal to rise of wash water, usually 24 to 30 inches[7]

C) <u>Clear Water Well.</u>

a) The tanks are covered to prevent any external contamination.

b) Depth varies to about 20 ft. or so, as per requirement.

c) Time of detention may be taken for few hours or so, as per requirements.

d) For ventilation vents are provided.

e) For rectangular tanks usual length width ratio is adopted.

f) Usually tanks are constructed underground.

g) Tanks are usually constructed of R.C.C., for longer life and prevention of percolation and leakage losses.

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h) For larger size rectangular tanks, compartments, with parts in partion walls, are provided.

D) <u>Chlorination.</u>

Chlorine dosages are not fixed but depend upto the chlorine demand of the water and chlorine residuals to be mained.

a) Actual dosages required can be found by demand tests. Usually combined chlorine residuals of 0.05 to 0.10 or 0.20 PPM are maintained.(7)(8) A dose of 1 PPM with contact time more than 20 minutes is sufficient for above residual.(8) However, "Steel" suggests a dose of 0.25 to 0.5 mg/L.(7)

b) Chlorine contained in standard steel cylinders weighing 100-150 lbs. shall be used.

c) Room for chlorination should be well ventilated and maintained at lower temperatures.

d) The dia. of feeding pipe should be 3/4 in. and of neither hard nor soft rubber.

e) Direct feeders, if used, should be maintained to a pressure of 25 to 30 psi in the delivery pipe.(8)

f) Steel containers should be able to withstand a bursting pressure of 500 psi, corresponding to a temperature of 190°F.

g) Use of chlorine, in clear water well, or in filter effluent line, is preferred. This will ensure adequate contact time. A contact time of 30 minutes is desired where only post - chlorination is being done.(7)

6.2. Design of Units

A) <u>Storage/Sedimentation basin:</u>

Total requirement by the end of year 1987 = 2,205,000 gal/day

Quantity supplied through tube wells = 1,476,000 gal/day.

∴ Balance to be supplied by canal water

= 2,205,000 - 1,476,000 = 729,000 gal/day

∴ For 15 days storage, quantity required will be

= 15 x 729,000 = 10,935,00 gal. = 1,455,000 cu.ft.

<u>Note:</u> In addition to the above net requirements, an amount for evaporation and percolation losses, which are to occur in such a large storage, is to be added.

Percolation losses will be minimized or so to say avoided by constructing concrete tanks. However, for evaporation an approximate allowance of 45,000 cu.ft. is made. A theoritical method of calculations will be described in section 6.2.1.

<u>Tank Dimensions:</u>

Total requirement = 1,455,000 + 45,000 = 1,500,000 cft.

Adopting a depth of 15 feet

Surface area required = $\frac{1,500,000}{15}$ = 100,000 sq.ft.

Adopting rectangular section of tank, with a side length of 400 feet

∴ width of tank = $\frac{100,000}{400}$ = 250 feet.

The tank will be divided into 4 equal compartments to facilitate clearing sediments and also to put into operation the required number of compartments for the present.

Allowing 6 inches for sediment deposit and 6 in. for free board.

Overall depth of tank will be 15.0 + 1.0 = 16.0 ft.

∴ Four comparts of 200 x 125 x 16, with outer side wall thickness of 12 in. and partition wall thickness of 6 in. will be provided.

Inlet:

A vertical baffle, extended upto about 6 in. from bottom and at a distance of 3 feet from side wall, with multiple openings of 6 inches, will be constructed. The inlets will be constructed parallel to width of tank and for first two compartments only. The compartments will be interconnected through ports in the partition walls and will be provided with some device to check the flow or otherwise as and when desired.

Outlet:

Multiple opening of 6 in., at a distance of 10 ft C/C, will be provided. Openings will be 2 feet below water level. The ports will be provided with some flow checking device so as to completely isolate the compartment during cleaning. An outlet channel 4' x 4' will be constructed extending over half the width of a compartment on either side from center. 2-6 in. dia.pipes will be taken from this channel to the filter inlet.

Clearing:

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Clearing will be done manually. During clearance tank will be emptied and connecting ports will be closed to prevent in flow. The tank will be flushed with a water hose and drained through an under drain by pump. Details are shown in Drawing No. 5.

B) <u>Rapid Gravity Filter:</u>

Water required to be filtered= 729000 gal./day

= 506 g.p.m.

<u>Surface dimensions:</u>

Taking filtration rate of 2 gal.per sq.ft. per minute

Area required = $\frac{506}{2}$ = 253 sq.ft.

Adopt two units

$\therefore$ Area of each = $\frac{253}{2}$ = 127 sq.ft.

Taking ratio of length to breadth as 1.27

1.27 x B x B = 127 or B = $\sqrt{100}$ = 10 ft.

$\therefore$ Length = 10 x 1.27 say 12.5 ft.

$\therefore$ Surface dimension of filter = 12.5' x 10'.

<u>Wash Water troughs:</u>

Taking rate of wash water = 2 cft/hour or 15 gal.per sq.ft. per min.

Amount of wash water = 15 x 12.5 x 10 = 1875 gal.per min.

Provide two troughs.

$\therefore$ Capacity of each trough = $\frac{1875}{2}$ = 938. gal.min.

From nomogram,(8) based on the formula $Q = 1.91\ b\ Z^2/3$,

where, Z = 12 inches

Q = discharge of trough in g.p.m.,

b = breadth of trough in inches, assumed 15 inches.

$Z = y_1 + LS$

where,

y_1 = depth of trough at upper end in inches

L = Length of trough (filter) in inches.

S = Slope of trough, assumed 2%

y_2, depth at lower end of trough is given by y_2

$= 2/3\ (y_1 + LS)$.

$\therefore y_1 = Z - LS = 12 - 2/100 \times 12.5 \times 12 = 12-3 = 9$ inches.

$\therefore y_2 = \frac{2}{3}(y_1 + LS) = \frac{2}{3}(9 + 3) = 8$ inches

$\therefore$ Adopt 2 troughs, per filter, 15 in. wide with upper and lower edge depths as 9 and 8 in. respectively, laid at a slope of 2% extending over the entire length of filter, i.e. 12.5 ft. Troughs should be kept 24 in. above sand level.

Under drains:

a) Laterals

Adopting ratio of perforations to filter area as 0.2%

Area of perforations = 0.2 x 125 = 25 sq. in.

Laterals will be laid width wise on either side of manifold, running length-wise in the center of filter.

Adopting 9 in. c/c spacing of laterals with 3 in. clearance from walls

No. of laterals on either side $= \frac{12 \times 12}{9} - 1 = 16-1 = 15$

$\therefore$ For 1/4 in. dia perforations, no of perforations per pipe on either side of manifold will be:

$\frac{25}{2 \times 15 \times .049} = 17$ Nos.

Assuming ratio of 0.25 for perforation and lateral x section

Area of lateral $= \frac{25}{2 \times 0.25 \times 15} = 3.33$ sq.in.

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$\therefore$ Dia of lateral $= \sqrt{\frac{3.38}{0.78}} = 2.3$ in.

Use standard size of pipe as 3 in.

$\therefore$ Provide 3 in. dia. laterals spaced @ 9 in. c/c, with 1/4 in. perforations @ 17 perforations per lateral.

b) Manifold

Taking X-sectional area of manifold as 1.75 times the total for laterals

$\therefore$ X-sectional area of manifold

$= 15 \times 2 \times 0.78 \times 3 \times 3 \times 1.75 = 370$ sq.in.

$\therefore$ Dia. of manifold $= \sqrt{\frac{370}{0.78}} = 21.5$

Adopt standard size of 21 in for manifold.

Depth of Filter

Adopt 15 in. thickness of gravel bed of grading given in Table 6.1, with a layer of 24 in. sand above it. Sand size should be as per criteria (e).

TABLE 6.1[7]

Grading of Gravel Layer

SIZE	DEPTH
$2\frac{1}{2}$ to $1\frac{1}{2}$ in.	5-8 in.
$1\frac{1}{2}$ to 3/4 in.	3-5 in.
3/4 to $\frac{1}{2}$ in.	3-5 in.
$\frac{1}{2}$ to 3/16 in.	2-3 in.
3/16 to 3/32 in.	2-3 in.
	TOTAL DEPTH 15-24 in.

The required depth of filter is sum total of the following :-

a) Height required by strainer system with gravel covering or concrete block support (maximum) = 0.75 ft.

b) Depth of gravel covering = 1.25 ft.

c) Depth of sand covering = 2.00 ft.

d) Depth of water above sand = 4.00 ft.

e) Free board allowance = 1.00 ft.

TOTAL 9.00 ft.

Capacity of Wash water Tank.

With a rate of 15 gal. per sq.ft. of filter area per minute storage for 5 minutes wash for two runs

= 15 x 10 x 12.5 x 10 = 18,750 gal.

Adopt 20,000 gallons capacity tank with a height of 30 ft. above ground.

∴ Provide 2 - 12.5 x 10 ft. filters with side depth of 9 ft., having perforated pipe under drainage system, with laterals 3 in. dia. and having 1/4 in. perforation, and manifold of 21 in. dia.

Details of filter are shown in Drawing No.6

c) Clear Water Well:

Total ultimate requirement = 4,185,000 gal/day.

Adopting 4 hours detention

Capacity required: $\frac{4,185,000 \times 4}{24 \times 7.48}$ = 93,000 cft.

Adopting a depth of 10 ft.

Surface area required = $\frac{93,000}{10}$ = 9,300 sq.ft.

Adopting a side length of 125 ft.

width = $\frac{9300}{125}$ = 74.5 say 75.0 ft.

$\therefore$ Divide the tank into two compartments, divide along the length

$\therefore$ Each compartment will be 75.0 x 62.5 ft.

Provide a free board of 2.0 ft.

$\therefore$ Overall depth of tank = 10.0 + 2.0 = 12.0 ft.

Thickness of wall may be taken as 12.0 in. with partition wall as 6.0 in.

Opening will be provided in the partition wall with control devices to regulate the flow in between the two compartments as and when desired.

$\therefore$ Provide a 12.0 ft. deep an R.C.C. structed underground water tank, with cover and having two compartments with internal dimensions of 75.0 x 62.5 ft.

D) Chlorination:

Actual chlorine demands test will be done, and feeding doses will be varied accordingly. In any case, a minimum of combined residual of 0.1 mg/L will be maintained. However, hereunder, chlorine requirements are calculated on the basis of feeding dose of 1 mg/L.

Ultimate water requirements = 4,185,000 gal/day.

Pounds of chlorine required = $\frac{\text{Gallons of water x 8.34}}{10^6}$ x dosage in mg/L

∴ Ultimate chlorine requirement

$$= \frac{4,185,000 \times 8.34}{10^6} \times 1.0 = 35.0 \text{ lbs./day.}$$

$$\text{Present requirement} = \frac{445000 \times 8.34}{10^6} \times 1.0 = 3.75 \text{ lbs./day.}$$

$$\text{Requirement by the year 1987} = \frac{2,205,000 \times 8.34}{10^6} \times 1.0$$

$$= 18.5 \text{ lbs./day.}$$

Chlorine will be supplied in steel cylinders weighing 100 - 150 lbs.

Cylinders will be placed on balance. The reading of the balance will be recorded before and after switching over the supply to account for the correct dosages given.

All operational instructions, as given by the manufacturer, will be followed strictly.

6.2.1 Evaporation losses from reservoir.

The actual evaporation losses are calculated from the results of U.S. Weather Bureau class A pan, kept for the purpose near the actual reservoir. A multiplying factor of 0.7 is used to reduce the results of pan to equivalent reservoir evaporation.

Some empirical formulas have also been formulated to find out at least approximate water loss if not the true one.

Some of these are as given by "Rohwer" and Dalton's formula. For our purpose we will describe "Dalton's formula".

Dalton formula for evaporation:(22)

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$$E_h = 1.80 \times C \times S \left(\frac{1-d}{b}\right)$$

where,

E_h, is the height in inches per hour due to evaporation loss.

C , is the empirical constant for air, taken 0.55 for quiet air, 0.71 for moderate air, and 0.86 for heavy wind.

S , is the maximum tension of water vapour at the temperature of evaporating water, in. of mercury, given in tables.

b , is the barometer reading, in. of mercury.

d , is the relative humidity.

Taking an average temperature of 80° F.

S , from tables (21)(8) = 0.926, C = 0.55 for quiet air,

d = 50%, and b = 29.5 in. of mercury.

$$\therefore EL = 1.80 \times 0.55 \times 0.926 \times \left(\frac{1 - 0.5}{29.5}\right)$$

$$= 1.80 \times 0.55 \times 0.926 \times 0.5/_{29.5} = .0155 \text{ in/hour.}$$

$\therefore$ Evaporation from tank surface/day

$= .0155/_{12} \times 250 \times 400 \times 24 = 3{,}080$ cft/day.

$\therefore$ Total evaporation loss in 15 days storage

$= 15 \times 3080 = 46{,}200$ cft.

$\therefore$ A provision of 45,000 cft. as provided earlier in the design of storage basin is sufficient.

6.2.2 Reduction in evaporation losses.

Total loss due to surface evaporation seems to be quite sufficient. There are, however, some measures through which the loss can be minimized. These are delineated below :

a) A shady environment around the tank may be developed through the plantation of long shady trees. However, this has a disadvantage of adding leaves etc. to water.

b) A monomolecular layer of fatty alcohols and their chemical relatives, may be provided over the water surface. The most commonly used is Hexadecanol.

c) A film over water surface may be developed by the use of styroper beads which are about 0.3-0.4 mm in diameter and expand to about forty times its original diameter. They being white in colour are said to help in reflecting solar radiations and consequently the water loss. The material is not costly and its use seems to be feasible, as such this is recommended.

The other possible method was to cover the tank but since storage is for sufficient time as such there is a possibility of developing taste and odour problems.

However, the use of styroper beads is recommended for the obvious reasons of its cheapness and ease in application.

CHAPTER 7

DIMENSIONING DATA FOR SEWERAGE PLANNING

Once an adequate supply of portable water has been assured to a community, the second greatest need of a modern municipality is an efficient system for the disposal of the used water and other wastes.

The basic data required in planning for such a system are: the volume of the waste that will find its way into the system and the quality of such wastes produced.

Wastes which a system is required to handle can, in general, be associated to the contribution from the following :

a) Domestic sewage.

b) Commercial sewage

c) Ground water infiltration

d) Industrial waste

The first two types of wastes are more or less similar in character and as such a general name of "Municipal wastes" is usually used to inculde the contribution from residential and commercial sectors, ground water infiltration, and other miscellaneous uses.

Industrial wastes because of their varying quantitative and qualitative characteristics are usually separately catagorized.

In the following, therefore, the two aspects will be dealt with separately for the two wastes contributing quarters and the probable figures will be estimated.

A brief discussion on storm water, which is also a contributing agent, will also be presented.

7.1. Municipal Waste

7.1.1. Volume of waste

Volume of municipal waste as described above comprises of the following:

a) Sewage from Residential and Commercial Areas.

Per capita water consumption forms the basis of the total amount contributed. The amount of waste water or used water reaching a sewer system is generally less than the amount of water supplied to the community. The difference can be attributed due to the various other uses of supplied water viz.: lawn and street sprinkling, gardening, fire-fighting, leakages, manufacturing processes, railroads, and miscellaneous house hold, and commercia l needs. Ordinarily, from 60 to 70 % of the total water supplied becomes waste water.[4] The loss, as it is called, is however largely made up by additions from private water supplies, surface drainage, and other accretions. Sometimes the additions even increase the actual contribution of spent water and the total flow to sewer may range 70 to 130% of the water supplied.[7] However, for practical purposes, it is usually assumed that the average rate of sewage flow, including a moderate allowance for infiltration, equals the average rate of water supplied.(7) In Khairpur, there are sufficient open spaces and the majority of streets are neither brick paved nor metalled, as such a sufficient portion of water will be used in street and lawn sprinkling and thus result in consequent loss. But on the other hand, there are a number of private water supplies, and because of higher water table there are chances for ground water infiltration as such the loss will to some extent be compensated by these additions. As such flow to sewer from the residential and commercial areas will be considered equal to the rate of water supplied, which is, 45g.p.c.d.

b) Ground water infiltration,

Water enters sewers through leaky joints, cracked pipes, or other openings such as man holes etc., this unnecessarily increases the load on, sewers, pumps, and treatment units. It's therefore advisable that some measures should be taken; at least, to minimize the flow if not completly check it. The measures that could be taken are: proper supervision during the laying of the sewers, rejecting cracked pipes, and providing water tightjoints.

No exact figures for the seepage water entering sewers can, however, be estimated. Various authors have specified different rates, stated in different ways. Some have specified it, interms of sewer length, and some in terms of area served by sewer.

Common allowances are:

1) 500 to 5,000 g.p.d. per acre; average 2,000.(4)

2) 15,000 to 50,000 gpd per mile of sewer.(7)

3) 500 to 5,000 gpd per mile of sewer per inch diameter (average 2,500) plus 100gpd per manhole.(4)

4) 5,000 to 100,000gpd per mile of sewer; average 30,000.(4)

No doubt the ground water table in Khairpur is higher and may add sufficient flow to the sewer through ground water seepage. However, it is proposed, to provide water tight joints, and make strict supervision during sewer construction etc. to reduce the infiltration. Moreover some of the flow will also be counter-balanced by the losses as mentioned in section (a) above, as such no extra provision need to be provided for ground water infiltration. The assumptions made in section (a) above thus hold good to cover the possible ground water contribution.

Therefore, volume of municipal waste produced is equal to volume of water consumed,

= 45 x 93,000 = 4,185,000gpd.

<u>7.1.2. strength of waste.</u>

It is usually sufficient to specify this strength of a municipal waste in terms of its 5-day B.O.D.

It has not been possible to investigate the strength of waste produced with respect to B.O.D.

A value of 0.17 pound B.O.D. per capita per day[3] has been used in the design of Stabilization lagoons at Sukkur, a city 14 miles north of Khairpur and having similar climatic conditions and standards of living.

This value can well be adopted for the strength of municipal waste of Khairpur.

Accordingly, the ultimate B.O.D. value will be:

$0.17 \times 93,000 = 15,810$ pounds B.O.D. per day.

7.2. Industrial waste.

Khairpur has since recently started developing in industry. A number of industries are existing in the town. The volume and strength of wastes produced from each unit will be discussed and found hereunder:

7.2.1 Present waste.

Following are the industrial units existing in the area.

a) Slaughter house

The average daily number of animals slaughtered are as follows:

Sheep and goats	130 - 140
Cows	5 - 10

Assuming a daily kill of 150, the amount of waste produced per kill of (Mixed) one animal is 359.[23]

$\therefore$ Volume of waste produced $= 359 \times 150 = 53,850$ gallons per day.

Strength of waste is taken to be 2,240ppm of B.O.D.[23]

∴ The amount of pollution contributed

$$= \frac{53850 \times 2240 \times 8.34}{10^6} = 1{,}000 \text{ pounds of B.O.D. per day.}$$

b) Tannery

At present there is only one tannery called, "Aijaz Tanneries", in the area. All the hides from city slaughter house are received here. The volume and strength of waste is as under:

Volume of waste per sheep skin = 4 gallons[23]

∴ Total volume from 140 skins = 4 x 140 = 560gpd.

Volume of waste per cow skin = 360 gallons[23]

∴ Volume from 10 cow skins =10 x 360 = 3600gpd.

∴ Total volume of waste = 560 + 3600 = 4,160gpd.

Average 5-day B.O.D. is taken as 1,200ppm[23]

$$\therefore \text{ Strength of waste} = \frac{1200 \times 4160 \times 8.34}{10^6} =$$

41.5 pounds of B.O.D. per day.

Note: Vegetable tanning process is used.

(c) R.C.C. Pipe factory

The manufacturing process is rather a dry process. Water is utilized in preparing Cement morters and curing of the finished products. No organic wastes are produced except the domestic waste from those living in the factory. As such the volume and strength of waste produced does not have any significant effect over the sewerage planning of the city and thus do not require any special consideration.

(d) Cotton Factory.

There is one cotton factory in the town. Here, raw cotton is carded, spun, spooled and wrapped. The slashing, drawing, weaving, and final finishing etc. processes are accomplished in the two textile mills in the town.

No water-borne pollution originates in this sequence of operations, since all are mechanical processes.

As such no special account for the waste contribution will be made.

e) Textile mills.

Two textile mills viz:"Khairpur textile mills" and "Ismail Textile Mills" are existing in the town. The various processes used include slashing i.e. Sizing the wrap thread received from town and other nearby cotton factories, desizing, kiering or boiling, bleaching, scouring, mercerizing, dyeing and printing.

Total daily product from the two, is said to be 4 tons.

Volume and B.O.D. contribution of waste will be as under.

Table 7.1 given below gives a typical waste analysis for the different processes in volved in cotton textile.

TABLE 7.1 [24]

VOLUMES AND STRENGTHS OF COTTON TEXTILE WASTES.

PROCESS	GALLONS PER 1000 POUNDS	B.O.D. P.P.M.
Sizing	60	820
Desizing	1,100	1,750

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Kiering	1,700	1,240
Bleaching	1,200	300
Scouring	3,400	72
Mercerizing	30,000	55
Dyeing (Vat)	19,000	140
Total	56,460	

Note: To the above figures, figures for printing and subsequent washing are to be added.

N.L. Nemerow gives the figures for B.O.D. in pounds per 1000 pounds of cloth resulting from printing and subsequent washing as given below.(25)

Process	B.O.D.	% of total
colour - shop wastes	12	7
Wash after printing, with soap	17 - 30	17 - 30
Wash after printing, with detergent	7	7
Sub total (printing)	-	15 - 35

A general figure of 70,000 gallons for volume of waste and 200 - 600 ppm for average B.O.D. of waste per 1,000 pounds of product are also given by him.(25)

W.A. Hardenbergh suggests that for 1,000 gallons of composite wastes from cotton mills, a B.O.D. contribution of sewage from 20 persons may be taken.(24)

From the above discussion it follows that a figure of 70,000 gallons for volume and 400 ppm for B.O.D. per 1,000 lbs of produce, will be reasonable to adopt.

$\therefore$ Volume of waste = 4 x 70,000 x 2 = 560,000gpd.

$$\text{Strength of wast} = \frac{560{,}000 \times 400 \times 8.34}{10^6} = 1{,}860 \text{ lbs/day.}$$

f) Silk factories

Number of small silk factories are existing in the town. These are located in a coloney called "Banarsi Silk Coloney" near Khairpur Textile Mills. The total product from all is about 500 lbs of Silk per day. The volume and strength of waste are: 850 gallons and 900ppm B.O.D. per 1000 lbs of silk respectively.[26]

$\therefore$ Total volume of waste $= \frac{850 \times 500}{1000} = 425$ gpd.

Strength of waste $= \frac{425 \times 900 \times 8.34}{10^6} = 3.2$ lbs per day.

Total volume and strength of various wastes.

Volume of waste per day

$= 53,850 + 4.160 + 560,000 + 425 = 618,435$ gpd

5-day B.O.D. of waste per day

$= 1,000 + 41.5 + 1,860 + 3.2 = 2904.7$ lbs of B.O.D. per day.

It is important to make a mention of the following:

No special treatment will be required to handle the additional B.O.D. loads from the industrial units, as the stabilization lagoons, which will be proposed to be used as a treatment method for sewage, are highly susceptible to the shock loads and varying characteristics of the industrial wastes.

However, the two industries viz: Textile and Tannery will be required to discharge their waste after they have at least provided the following pre-treatment. This will result in. benefits as outlined below:

Testile waste:

Screens with fine mesh to intercept fibers and floating v

materials will be provided.

Willem Rudolf(26) suggests: "Bleaching wastes usually may best be handled by storage and gradual discharge in to the combined wastes from other finishing and dyeing operations".

Therefore, the industry will be required to construct tanks where the two wastes will be combined and discharged therefrom at a regulated rate into the sewer.

These steps will, therefore, result in reducing B.O.D., and suspended solids load to some extent neutralize the two wastes and thus adjust pH, and will allow a regular discharge in the trunk sewer, thus reducing the size of pipe and possibility of back flow.

Tannery waste

Fine screens with 3/32 to 1/4 in. perforations will be provided to intercept hairs, fleshings, and other floating materials.(23)

Alkaline wastes from beam house and acidic wastes from tan yard will be mixed in anequalizing tank and discharged at regulated rate in to sewer.

This will result in the benefits as outlined above under textile waste.

7.2.2. Future wastes.

It is very difficult to predict exactly the type and number of industrial establishments without any fixed future plan at hand. There is'nt any fixed plan set by the authorities for the future as yet. However at present, as already described, the major fields of development are industry and agriculture.

It will, therefore, be reasonable to expect an increase of about 50% in the production of cotton for the obvious reasons of land reclamation and thus there will be a subsequent increase of textile mill production equivalent to one more number.

Regarding tanneries and slaughter house, these vary in direct proportion to the increase in population and standards of living. Thus an increase of about 150% in slaughter house and 150% in tannery seem to be a fair approximation.

The silk industry is also expected to increase, but since the volume and B.O.D. contribution of the waste are not much as such these will not be considered in calculating the loads.

Soft drink bottling plant may ~~be~~ also be installed but this too is insignificant for our purposes, as the volume of waste is 1500 gallons with a B.O.D. of 150ppm per 100 cases bottled,(24) which is not too much.

Rayon and Synthetic fiber production may find their way in the area too, however wastes are not of so much importance, a volume of 2000 - 5000 gallons with B.O.D. of 20 - 500ppm per 100 lbs of rayon,(26) for example, is produced.

However, it will be reasonable to consider about 60% of total present textile waste, 150% of slaughter house waste, and 150% of tannery, as an additional load, with respect to volume and B.O.D., reaching the treatment plant by the end of year 2007. These three, will be considered in load calculations. The following considerations are taken, while assuming the above mentioned percentages.

a) That, all the industries will be required to give at least pre-treatment to their waste, to the extend as specified by municipal authorities.

b) That, these percentages, which are taken for raw wastes without pre-treatment, will in fact be able to cater for higher loads. Thus the additional loads if contributed by some industrial unit will safely be taken care of.

c) That, if it is felt sometime that the loads have increased the designed loads, tests on influent will be made and additional lagoons, which are easy to increase, will be constructed to take care of such an increase.

Following the above discussion volumes and strengths of waste reaching the treatment plant will be as follows:

a) Textile wastes

Volume of waste = 560,000 + 0.6 x 560000

= 560,000 + 336,000 = 896,000gpd

5-day B.O.D. strength of waste = 1860 + o.6 x 1860

= 1860 + 1116.0 = 2976.0 pounds per day.

b) Slaughter house wastes

volume of waste = 53,850 + 1.5 x 5385.0 = 53,850 + 80,750

= 134,600gpd

5-day B.O.D. strength of waste = 1000 + 1.5 x 1000

= 1500 pounds per day.

c) Tannery wastes

Volume of waste = 4160 + 1.5 x 4160 = 4160 + 6240

= 10,400 gpd

5-day B.O.D. strength of waste = 41.5 + 1.5 x 41.5

= 104.0 pounds per day.

∴ Total ultimate volume of industrial waste

= 896,000 + 134600 + 10400 = 1,041,000gpd

Total B.O.D. of industrial waste

2976.0 + 1500 + 104.0 = 4580 pounds per day.

7.3. Storm Water

Data concerning the relation between rainfall, intensity, duration, and frequency could not be made available for Khairpur.

Moreover, from the data given in section 1.7.2, it is deduced that amount of preeipitation is too small and unreliable to provide for any special provision in the system.

However, in the following, a brief discussion on the subject will be presented to serve as guiding outlines for the designer some time in the future; if such necessity arises.

7.3.1 Storm Run-off.

The determination of storm run-off or required capacity of the sewer are among the first steps in designing the system.

Number of methods, such as: "Rational method" or empirical formula, are available for the purpose. Rational method is thought to find a better place in its application. Mathematically it is described as: (27)

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$$Q = CIA$$

Where, Q, is run off in C.F.S.

C, is run off co-efficient

I, is intensity of rain fall in C.F.S. per acre (inches per hour is the approximate equivalent)

A, is drainage area in acres.

Note: Value of intensity, I, used in above equation should be taken as the greatest intensity of rainfall lasting for time of concentration, not the intensity for shorter period.

Time of concentration can be defined: as the time it takes to establish run-off and flow from the most distant point in the district to reach the nearest in let and thence flow through the sewers to the point in the drainage system for which the required capacity is to be estimated.

The value of C must be estimated from a study of the soil, slope and the character of the surface, and a consideration of probable future development. Table 7.2 gives value of run off co-efficient or relative imperviousness as it is called for different surfaces.

TABLE 7.2[27]

RANGES OF RUN-OFF CO-EFFICIENTS FOR DIFFERENT CLASSES OF SURFACES

Class of Surface	Range of Co-efficient.
1. Water-tight roof surfaces	0.70 - 0.95
2. Asphalt, pavements in good order	0.85 - 0.90

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3. Stone, brick and wooden block pavements with tightly cemented joints.	0.75 - 0.85
4. Same, with open or uncemented joints	0.50 - 0.70
5. Inferior block pavements with open joints	0.40 - 0.50
6. Macadamized roadways	0.25 - 0.60
7. Gravel roadways and walks	0.15 - 0.30
8. Unpaved surfaces, railroad yards and vaccant lots	o.10 - 0.30
9. Parks, gardens, lawns and meadows, depending on surface slope and subsoil character	0.05 - 0.25

For practical purposes value of the run-off co-efficient may be assumed from the ranges given below.(27) It should be noted that conditions for which the value is selected should describe for those assumed to exist at the end of the design period.

1. For the most densely built-up portion of the district	0.70 - 0.90
2. For the adjoining well built-up portions	0.50 - 0.70
3. For the residential portions with detached houses	0.25 - 0.50
4. For the suburban portions with few buildings	0.10 - 0.25

Thus by selecting a proper value of, rainfall intensity I, from rainfall duration - intensity curves, as discribed above, and run off co-efficient, amount of run off from a known measured area can be made.

CHAPTER 8

PROPOSED SEWERAGE SYSTEM

Collection of sewage is usually accomplished by underground conduits called sewers.

The general process of removal or collection and treatment of sewage is designated as sewerage and entire system of conduits and appartenances is known as a sewerage system.

A system may be specifically intended for removal of storm sewage, or sanitary sewage, which includes domestic and trade wastes, or for the both at one time. Choice of either of the systems is governed by economical, local and many more considerations.

In the following, these systems will briefly be described and final choice for the one including the pipe material etc. will be presented.

A typical design of one trunk line of the system will also be presented.

8.1. Choice of the Sewerage System

In practice, two sewage systems are used viz: separate, and combined system.

In the combined system **one** sewer takes all the domestic sewage, trade waste, surface water, and storm water. Whereas, in the separate system domestic including trade wastes and surface drainage including storm water are carried by separate pipes.

A third system (so called partially separate system) is a compromise between these two: there is only one set of drains from the buildings, storm water and domestic sewage being discharged into the sanitary sewer; storm water from roads, pavements and, from house fronts is kept separate and discharged into a storm water drain. This system is no more in use as it is difficult to adopt in practice.

Adoption of either of the two systems is governed by a set of considerations; among them the main being economy and volume of rain water.

Economic necessity, either real or fancied, has in general urged the designers to adopt a separate system, even when the volume of rain water had required to adopt a combined system. Small towns, which are not in a position to finance adequate combined systems, usually provide separate system for sanitary sewage collection and let the storm water over flow and be taken care of by street gutters and natural water courses. This has been practised for years and these occasional over flows do not cause nuisance. However, even a brief storm water over flow mingled with sanitary sewage is objectionable for the obvious reasons of health and asethetic problems.

Moreover, it is often easier to secure funds for the less costly separate systems than for the more costly combined systems.

These considerations call for a prompt decision in favour of a separate system without even weighing the comparative merits and demerits of the two. However, the following additional considerations limit the adoption of a separate system:

a) Troubles are likely to arise in a combined sewer laid on a flat gradient from deposition of organic matter and subsequent clogging.
b) The variations in strength and flow that result from the combined system cause difficulty at the treatment plant.
c) Where rain fall is scanty, a combined sewer will run under loaded for most of the time and thus will be under constant danger of clogging and choking.
d) Deep excavations are required for combined sewers where topography of thearea is flat.
e) Where pumping of sewage is involved, pumping costs are unnecessarily increased in a combined system.
f) Treatment costs in a combined system are also increased.
g) Constant velocities of flow are difficult to maintain.

A study of Khairpur leads to the adoption of a separate system for the following reasons.

a) Topography of area is flat and more over water table in the area is high, as such deep excavations can not be afforded.

b) Rainfall in the area is little and unreliable. Therefore occasional showers can be taken care of by existing open main drains which will be repaired for the purpose and rainwater from these drains will be used in irrigation or discharged directly into the canal.

c) A heavy financial burden can not be sustained by the municipal authorities particularly at present when both water supply and sewerage schemes are to be executed.

d) Steeper slopes can not be provided to maintain self-cleaning velocities for dry weather flows, because of restriction to deep excavations.

Thus a separate system to carry sanitary sewage i.e. domestic and trade wastes will be provided for Khairpur sewerage system.

8.2. Pipe Material

A variety of materials is in use for sewer pipes, each having its own merits and demerits. However, selection of one or the other is governed by the factors such as economy, working efficiency, adaptability to local conditions, and availability.

There are certain principal requirements for the sewerepipes in light of which a selection, with above factors in mind, should be made. The requisites are enumerated as below :

a) Sufficient strength to stand the stresses developed.

b) Durability for longer life.

c) Imperviousness.

d) Smoothness of internal surface to avoid excessive frictional resistance.

e) Uniformity of size and shape.

f) Joints of such type that they can be made tight.

g) Resistance to acids, alkalies, corrosive gases.

h) Hardness to resist erosion.

i) Ease of handling and installation.

It is practically impossible to find any pipe which could meet all of the above mentioned requisites. However, an effort should be made to select the one which could meet the majority of these requisites, easily available, best suited to local conditions, and is comparatively economical.

Below are described few of the pipe materials with emphasis on their advantages, disadvantages, and their applicability to local conditions.

a) Vitrified clay pipes:

This pipe has number of advantages as: Resistant to corrosion due to decomposition products of sewage like

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acids, corrosive gases and alkalies, smoothness of internal surface, imperviousness, sufficient crushing strength which can be varied with wall thickness as specified by A.S.T.M., the socket and spigot pipes should sufficiently be made water tight, are reasonably cheaper subject to availability locally or in nearby districts.

Its disadvantage is the liability to damage in transit and handling.

These pipes are neither manufactured locally nor anywhere in nearby districts as such its disadvantages cannot be avoided in any case and also it will cost much. Therefore, these pipes are not recommended for use in this project.

b) Corrugated iron pipes:

These pipes are extensively used for storm sewers. It has advantages of: high resistance to corrosion, support heavy loads without damage because of elasticity, easy to lay and join, jointing needs only tightening of clamps. It has a disadvantage that corrugations reduces the velocity of flow and consequently the capacity as compared to a smooth iron pipe of same diameter.

These pipes can not be recommended for use in this project as, it is not locally available, require larger sizes, it has not been extensively used for sanitary sewers, and will be costly.

c) Cast iron pipes:

These pipes have advantages: high strength, water tightness, variety of sizes, imperviousness, longer life, and better adoption in pressure sewers.

THe disadvantages they have are of: high cost, heavy weight and thus larger sizes, are difficult to handle, are subject to corrosion by highly septic sewage.

Since these pipes, if used, will have to be imported from foreign countries as such will be very costly, however, these may be considered for pressure line and outfalls if it is desired.

d) Steel pipes:

These have been used under unusual conditions, as for instance where light weight may be of importance. They are rarely used in sewers, metal is subject to rapid corrosion and the obstructions caused by rivet heads and the joints are objectionable.

These pipes, therefore, for obvious reasons, is not recommended.

e) Transite pipes:

These pipes are made of asbestos and cement and could be used for pressure pipes. These pipes have the advantages of: lightness in weight, lesser number of joints as normally they are available in lengths of 13 feet, easeness and quickness in the formation of joints, which are sleeve-type joints, and reduce ground water infiltration. These pipes, in general, may be used where cast iron is suitable.

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These pipes are neither locally manufactured nor available in the country and as such will have to be imported, however, their use in pressure lines is recommended. Either of the pipes viz: cast iron or transite pipe, can be used for pressure lines wherever required, subject to ready availability.

f) Bituminous fiber pipes (Pitch fiber pipes):

These pipes have many advantages such as: durability, strength, resistance to corrosion, water tightness, and ease of construction.

Among the disadvantages: limitation of size from 2 to 8 in., and subject to deterioration if exposed to direct sun light for a long time.

These pipes therefore, cannot be considered for this project for obvious reasons and that they are not locally available.

g) Cement concrete pipes:

Concrete pipe may be plain or reinforced. Plain concrete pipe are usually smaller in diameter and are usually used for storm sewers. However, for our purpose in the following we mean reinforced concrete pipe (R.C.C.) for diameters above 12 in. and plain cement concrete pipes for diameters below 12 in.

They have a number of advantages: cheap, variety of sizes, strength, variety in types of joints like socket and spigot, and mortise and tenon joints, it can be made of sufficient strength and tightness, extra strength can be obtained by bedding and cradling.

It has few disadvantages like: less resistant to corrosion and thus early deterioration particularly in hot climates, less resistant to erosion, and is brittle. It also involves greater number of joints and thus there is a possibility of seepage if joints are not properly water tight.

However, for the purpose of project this pipe will be used in sewerage system for the following reasons.

a) This pipe is locally available, transit losses and charges are reduced as the factory is located right in the town on the west of Mir wah canal.

b) The cost of pipe is fairly cheap.

c) Because of flater topography of the area and restrictions to deep excavations, flater slopes, those required to maintain self cleansing velocity will have to be adopted, this will indirectly control the velocity in lower limits and thus there will be no erosion problems.

d) To reduce ground water infiltration proper type joints can be made, thus joints can be made water tight.

e) Corrosion problems can be greatly reduced by proper ventilation of line.

f) Because it is locally manufactured as such the manu-

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facturer can be asked to change the quality of constituents, like use sulphate resistant cemnet, to improve the quality of material. Moreover, the manufacturing processes can be strictly supervised, if so desired.

g) A bitumen bath can be given to the pipes, this will improve the pipe durability and resistance to crown corrosion. The process though may be expensive but still on the whole costs will be much lower as compared to those imported otherwise.

8.3. Design Criteria

An efficient and successful operation of a sewerage system calls for certain basic considerations during its planning, design and construction. A system, which entails quick conveyance of sewage, does not cause deposition problems and subsequent choking of pipe and septicity of sewage, is laid at suitable slopes, does not maintain either too excessive or too low velocities, and is of sufficient capacity etc., should be the aim of every designer. These and many other details from the design criteria for the sewerage system and are important to consider before actual design is precedded.

8.3.1 Minimum and peak flow of sanitary sewage.

There are constant hourly, daily, and seasonal variations in flow as well as characteristics of the sewage. However, its variation is in direct proportion to the water consumption.

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Since no such records are available so that the exact relationship between average, minimum and maximum flows could be deduced from, hence for design purposes it is sufficient and reasonable to follow certain standards set by the authorities working in this field. These are given below:(28)

Peak flow in laterals and sub-mains = 4 x Average daily flow

Peak flow in mains and trunk sewers = 2.5 x Average daily flow

Minimum flow in all sewers = 0.5 x Average daily flow

8.3.2 Velocities of flow.

a) Minimum velocity.

The velocity of flow in a sewer should be sufficient to prevent the deposition. Such a velocity is in the neighbourhood of 1 f.p.s., which should be available under all the conditions of dry-weather flow.(29) The minimum velocity when full should, therefore, be about 2 f.p.s. Under this condition, the velocity of 1 f.p.s. occurs when the sewer is less than 17% full.(29)

Therefore, a minimum permissible velocity of 2 f.p.s. will be adopted in this project.

b) Maximum velocity.

The limit for maximum permissible velocity of flow in sewer is fixed with an intention to avoid excessive erosion of the invert, which is caused by the grit ladden water moving at a high speed.

The maximum permissible velocity of flow is fixed at about 10 f.p.s.,(29) and this will be adopted in this project.

8.3.3 Sewer slopes.

a) Minimum slope.

Where topography of area is flat, a minimum slope which produces minimum velocity of 2 f.p.s. when running full is to be adopted. At other places it is desirable to follow the ground slopes and to avoid deep excavations. Sewers should be laid at slopes parallel to ground slopes and at minimum required depths. This will be followed in this project.

b) Maximum slope.

The maximum allowable slope should be one which gives a maximum velocity of 10 f.p.s. when running full. The same will be followed in this project.

8.3.4 Sewer Shape.

Varieties of sewer are used in a sewerage system, each having merits and demerits of their own. However, most commonly used is a circular section and will be adopted in this project for number of reasons as under:

a) Circular section possess good hydrautic properties, however, egg shaped are claimed to possess better but they have the disadvantage of being unstable in place and requiring some type of base to hold it in place.
b) It gives maximum of X-sectional area for the amount of material in the wall.
c) Convenience in manufacture.

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d) Locally available.

e) More stable in place than other sections

8.3.5 Sewer Size.

Limit for minimum size to be used in a sewerage system is to avoid, clogging of sewer and the consequent nuisance, and troubles in cleaning.

No limit for maximum size has been set, as it is capacity requirement dependant.

Babbitt has given a minimum size of 8 inches to be used where as Escritt has given a minimum size of 6 inches to be used.[19]

However, a minimum of 8 inches size will be used in this project.

8.3.6 Sewer depth.

Sewers are laid sufficiently below ground surface in order to protect them against breakage or traffic shock, to permit them to drain the lowest fixture on the premises served by them, and to avoid freezing problems wherever these occur.

Building sewers are usually laid at a slope of 1/4 in. per ft.[4]

An earth cover of 2 ft. will cushion most shocks.[4]

Since at Khairpur, inside streets are mostly narrow and there is less vehiculars traffic specially of heavy load and fast moving vehicle as such it is desirable that all street sewers should be laid at a minimum depth of 2 ft., and all mains and trunks should be laid at a minimum depth of 3 ft. below ground surface.

8.3.7 Miscellaneous criteria.

a) At any junction where sewers of different sizes meet, it is good practice to locate the sewers in the manhole so that the difference between the elevations of their inverts is not less than one half of the difference between the vertical diameters of the sewers for small sewers, and not less than three-quarters of the difference between the diameters for large sewers.[24] However, an average distance of 1 inch between the inverts will be adopted.

b) Manholes will be provided at every change of direction, grade, elevation, size of pipe, and at street junction. Babbitt recommends a range of 300 to 500 ft. with 200 - 300 ft. preferable.[29] However, a spacing of 300 ft. will be kept between the two manholes on straight reaches.

8.4. Sewer Embedment.

Where the bottom of thetrench for a pipe is in firm earth, a suitable bed for the pipe may be formed by shaping the natural earth. Similar procedure will be followed in laying sewers at Khairpur. Where the soil is too soft to furnish a satisfactory bedding for the pipe, it is then necessary to excavate to a depth somewhat below the proposed grade of the sewer and to provide a concrete cradle or a suitable foundation of some other materials.

Variety of types of embedment of sewers are used in practice, these are shown in Figure 8.1 as suggested by the W.S. Dickey Clay Manufacturing Company.(7)

Type 3 onwards are practiced in soft soils. Type 5 is recommended. This type will be adopted at places where sewer is quite close to water table and soil is very soft. For moderately soft soil, type 3 may be used to reduce costs. Type 5 is comparatively cheap and has sufficient load factor.

8.5. Man holes.

Variety of appartenances are required in a sewerage system to ensure proper operation. Among these, manholes are the most important, these provide an access for cleaning, inspection, and sometimes for ventilation when not completely closed.

These will be provided at, an intersection of two or more sewers, every change in grade, change in size, change in direction, and mid-way between intersections in long blocksand 300 ft. apart on straight reaches.

Size of manhole should be sufficiently large to permit, easy entrance of a man, and the use of standard 4-foot cleaning rod. A minimum of 4-feet bottom diameter manholes will be provided.

Manholes are made either bottle shaped, or with the lower parts vertical, or nearly so, and the upper most

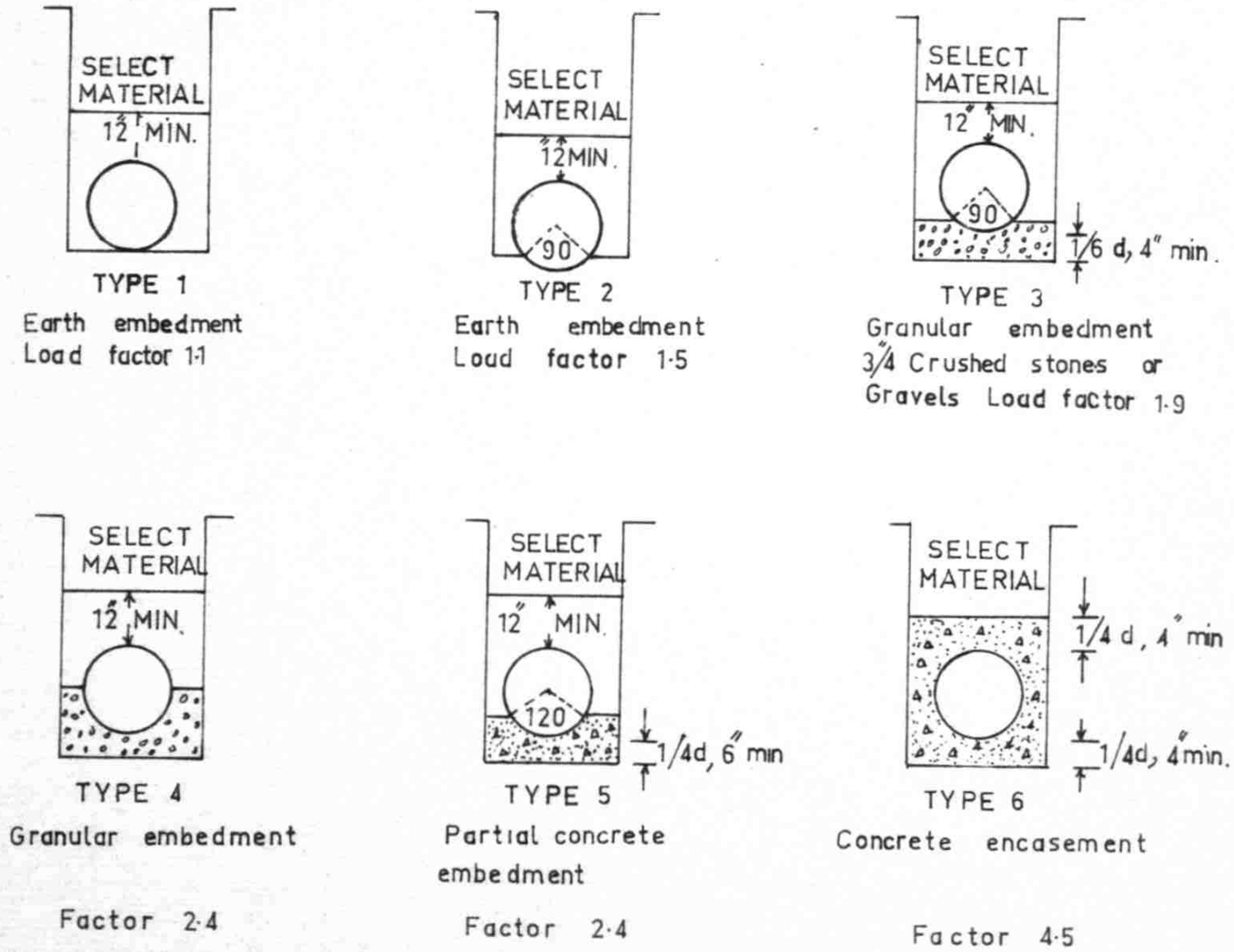

FIG 8·1 SEWER EMBEDMENT

few feet domed or stepped in. Usually they are cricular in plan. The inside diameter at the top is commonly 22 or 24 inches.

Thus manholes will be 24 in. at top and 24 in. dia cast iron covers will be provided. These will be closed so as not to permitstorm water, sands, debris, and other street washes. Covers will be roughened to avoid slipping ventilation, which is essential, particularly in our case where summer temperatures are appreciably high, will be provided through house vents, traps will not be provided in house sewers.

Among materials in construction of manholes, usually concrete is preferred, which if used, should be about 8 inches in thickness. However, since clay bricks are locally available and are cheap as such best quality available of 9"x $4\frac{1}{2}$"x $2^1/4$" dimensions bricks should be used. Thickness of walls should be kept as 9 inches. Walls should be plastered both sides to ensure water tightness.

The bottom of manholes, however, should be made of 8 inches concrete slabs sloping towards the outgoing sewer making a continuation of the sewer pipe. Where branch sewers meet the mains, proper curvatures will be maintained in the bottom slabs. Inverts of branch sewers should always be set above the inverts of mains or sub-mains to avoid sewage from backing up in the smaller sewers.

Cast iron steps, coated with anti-corrosion enamel or tar which, otherwise would be corroded very

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quickly in the presence of sewage corrosive gases and moisture, will be provided vertically 1 foot apart and will be set horizontally. This will facilitate entrance to, and exist from, manholes.

8.6. Design of Sewers

8.6.1 Design of storm water sewers.

Aspointed out earlier for the present provision for the removal of storm water neither will be kept in sanitary sewers to work as combined sewers nor a separate storm sewers will be provided.

The storm water will be collected in the present main drains which will be repaired for efficient operation, the present pumping sets will be used to pump the water in canal or to agricultural lands as is being done for sewage water at present. Storm water from minor overflows or otherwise, which does not find its way to the main drains will not cause nuisance in any way and will seep down in thesoil.

However, with the development in the standards of living of thecommunity and improvement in the financial status of the municipality, a system can be designed on the similar lines as will be outlined for sanitary sewers

Division of the city into catchment areas as specified for sanitary sewer holds good for the storm sewers. However, a self cleansing velocity of 3 fps is to be maintained in storm sewers.

Procedure for, arriving at the amount of run off resulting from the maximum considered storm, has also been outlined in section 7.3.1.

8.6.2 Design of Sanitary Sewers.

The city has been divided into twenty two catchment areas, designated A through V. The catchment areas with the layout of the main sewers, trunk, sub-trunk line, and treatment plant are shown in Drawing No. 7. Main sewers A through H, and M through P, serve Zone A area which consists of the densily populated area, so called, maintown commercial center, and majority of public buildings and offices. These mains except main sewer H serve the northern portion of the town located on the east of Mir wah canal. Sewage from these mains is collected through trunk into a wet well for low lift pumps located near the road bridge. This wet well also receives sewage from Zone B. The sewage from the two zones is lifted to a level such that it could flow thereafter to the treatment plant under gravity. The same trunk line is continued thereafter carrying the sewage from Zone C on its way and ends in a sump well at the treatment plant which is located in the southern part of the town, on the western side of Mir wah canal and the eastern side of railway line.

Main sewers I through L serve Zone B, area located on the southern part of the town and on the east of Mir wah canal. At present there are less buildings but there is a major tendency of building in this zone as described earlier. Sewage from these mains and main

H, is collected through a sub trunk in the wet well of the lifting station as described in the above paragraph.

An intermediate lifting station will also be provided on this sub-trunk line.

Lifting of sewage in both cases is done to avoid laying of sewers to depths exceeding 11-12 feet from ground levels. At these depths sub-soil water occurs as such sewers, if laid below this level, will have to be laid in water and sewers will always have to remain in water, which is not desirable.

Sewage from Zone C so called industrial area zone will be collected through mains Q through V. These mains will be connected to the trunk line coming from the eastern part of town (lift station) and thus the sewage is taken to the treatment plant.

Each of the main sewers A through V serve individually the catchment areas A through V.

The area, the future population, and the amount of waste water of each catchment area or in other words of each main sewer is given in Table 8.1.

The design of sewers is presented in Tables 8.4 and 8.3. Of the main sewer only Main sewer A, which is the starting point of the trunk sewer and main sewer L, which is the starting point of the sub-trunk sewer have been designed. These were necessary to fix the initial invert levels of the trunk and sub-trunk lines connecting these and other intermediate main sewers, The sub-trunk line and the trunk line have also been designed completely.

Profile of trunk line from treatment plant sump well to the lift station is shown in Drawing No.8.

TABLE 8.1

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

CATCH MENT AREA	AREA ACRES	MAIN SEWER SERVING	PRESENT POP.DEN-SITY PER-SONS/ACRE	FUTURE POP.DEN-SITY PER-SONS/ACRE	FUTURE POPULA-TION PERSONS	EXPECTED WASTE WATER GALLONS/ DAY.
A	40	A	46	72	2,883	129,735
B	74.0	B	46	72	5,335	240,075
C	38.0	C	46	72	2,630	118,350
D	39.0	D	46	72	2,812	126,540
E	43.0	E	46	72	3,105	139,725
F	81.0	F	46	72	5,845	263,025
G	64.5	G	46	72	4,650	209,250
H	54.0	H	46	72	4,055	182,475
I	134.0	I	29	75	10,070	453,150
J	70.0	J	29	75	5,252	236,340
K	87.0	K	29	75	6,528	293,760
L	102.0	L	29	75	7,655	344,475
M	30.0	M	46	72	2,160	97,200
N	28.0	N	46	72	2,020	90,900
O	39.0	O	46	72	2,810	126,450

P	51.5	P	46	72	3,710	166,950
Q	73.0	Q	15	45	3,290	282,650(148050 + 134600)
R	82.0	R	15	45	3,695	166,275
S	81.0	S	15	45	3,645	164,025
T	63.0	T	15	45	2,840	486,200(127800 + 358400)
U	88.0	U	15	45	3,960	715,800(178,200 + 537600)
V	90.0	V	15	45	4,050	192,650(182250 + 10400

The determination of velocities, slopes, and sizes, in the design of sewers is based on a nomogram based on Manning Formula for circular pipes flowing full with n = 0.013.[7]

The selction of the particular formula and the value of "N" is made in light of thefollowing discussion.

An extensive comparison of values of n, computed by the Kutter and Manning formulas from the results of experiments, was made by King,[30] from which he concludes that: "The agreement between Manning's N and Kutter's N is extremely close, and the two formulas (using the same value of N in each case) give results agreeing well within the limits of uncertainity which must exist in the selection of N, under all ordinary conditions."

Divergence is, however, marked in case of the small rather than of the large sewers.

It has also been found that for ordinary works the Kutter and the Hazen - Williams formulas agree closely enough to permit the use of either.[26]

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Thus with the proper selection of N in Manning and Kutter and C in Hazen - Williams either of the three could be used.

Regarding the value of N, following table is given by Robert E. Horton, as reproduced in King's "Hand book of Hydraulics."[30]

For sewers 24 in. or less in diameter	0.015
For sewers over 24 in. in dia. of bestwork	0.012
For sewers over 24 in. in dia. under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard burned brick and laid with great care, with close joints	0.014
For brick sewers under ordinary conditions	0.015
For brick sewers, rough work	0.017 to 0.020

Met calf & Eddy suggest the value N = 0.015, in view of the possibility of rough pipe and poor pipe-laying, as well as the presence of branches and manholes, all of which will increase resistance.[27] He further points out that usual practice is to adopt N = 0.013.

Steel, states that:"For clay or concrete pipe, N is sometimes considered as 0.015, with good construction methods, careful aligning of the pipe, and smooth joints, "N" may be taken as 0.013 and this is the standard practice.[7] For large concrete pipes with careful construction, "N" may be even less.

In view of the above discussion, Manning's formula with value of N = 0.013 is taken in the design of the sewer system.

The sizes of sewers selected are standard sizes as specified by the American society of Testing materials (A.S.T.M.), thickness of pipes are also taken as specified by the same authority, these are given in Table 8.2.

A drop of 0.04 ft. is given in manholes where change in direction occurs.[7]

The design of sewers is presented in Tables 8.3 and 8.4. The minimum depth of flow has been checked to be not less than 2.0 inches and the velocity not less than 1.4 ft/sec., by using curves at minimum flows not partial flow.[7]

TABLE 8.2[14]

DIMENSIONS OF CEMENT CONCRETE & R.C.C. SEWER PIPES

Int.Dia (inches)	Plain C.C.Pipe (A.S.T.M.Specif.C-14-41)	R.C.C. Pipe (A.S.T.M.Spec. C-75-41)
	Shell Thickness(in.)	Shell Thickness (in.)
8	1	-
10	1 1/8	-
12	1 1/4	1 3/4
15	1 1/2	2
18	1 3/4	2 1/4
21	2	2 3/8
24	2 1/4	3 3/8
27	-	3 1/2
30	-	4

TABLE 8.3

FLOW IN SEWERS

LINE	Manholes From	Manholes To	Increment of Area (acres)	Population Density	Population Increment	Population Total	Sewage Flow (Gal / Day)	Sewage Flow (Cfs)
Main	A-MH 5	A-MH 4	11.5	72	830	830	37,350	.058
Line	4	3	10.5	72	757	1587	71,415	0.110
A	3	2	8.0	72	576	2163	97,415	0.150
	2	1	6.0	72	432	2595	116,775	0.180
	1	T-MH 56	4.0	72	288	2883	129,735	0.200
Main	L-MH 8	L-MH 7	40.0	75	3000	3000	135,000	0.208
Line	7	6	15.0	75	1125	4125	185,625	0.286
L	6	5	12.0	75	900	5025	226,125	0.348
	5	4	10.5	75	789	5814	261,630	0.402
	4	3	9.5	75	714	6528	283,760	0.437
	3	2	8.0	75	600	7128	320,760	0.494
	2	1	5.5	75	414	7542	339,390	0.520
	1	STMH	1.5	75	113	7655	344,475	0.530

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LINE	Manholes From	Manholes To	Increment of Area (acres)	Population Density	Population Increment	Population Total	Sewage Flow (Gal / Day)	Sewage Flow (Cfs)
Sub-	ST-MH 18	ST-MH 17	102.0	75	7,655	7.655	344,475	0.53
Trunk	17	11	87.0	75	6,528	14,183	638,235	0.98
Line	11	6	70.0	75	5,252	19,435	874,576	1.35
	6	3	134.0	75	10,070	29,505	1327,725	2.04
	3	1	54.0	75	4,055	33,560	1510,200	2.33
	1	Lift station - Wet Well		-	-	-	1510,200	2.33
Trunk	T-MH 56	T-MH 51	40.0	72	2,883	2,883	129,735	0.20
Line	51	50	74.0	72	5,335	8,218	369,810	0.57
	50	47	38.0	72	2,630	10,848	488,160	0.75
	47	45	39.0	72	2,812	13,660	614,705	
	-	-	51.5	72	3,710	17,370	781,650	1.21
	45	43	43.0	72	3,105	20,475	921,375	1.42
	43	39	39.0	72	2,810	23,285	1047,825	1.61

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LINE	Manholes From	Manholes To	Increment of Area (acres)	Population Density	Population Increment	Population Total	Sewage Flow (Gal / Day)	Sewage Flow (Cfs)
	T-MH 39	34	28.0	72	2,020	25,305	1,138,725)	
			81.0	72	5,845	31,150	1,401,750)	2.16
	34	33	64.5	72	4,650	35,800	1,611,000	2.43
	33	Lift Station	30.0	72	2,160	37,960	1,708,200	2.62
	Lift station	32	447.0	75	33,560	71,520	3,218,400	4.95
	T-MH 32	T-MH 18	-	-	-	-	3,218,400	4.95
	18	14	73.0	45	3,290	74,810	3,501,050	-
			82.0	45	3,695	78,505	3,667,325	5.65
	14	11	81.0	45	3,645	82,150	3,831,350	5.9
	11	7	63.0	45	2,840	84,990	4,317,550	6.68
	7	4	88.0	45	3,960	88,950	5,033,350	7.75
	4	T-MH 1	90.0	45	4,050	93,300	5,226,000	8.03

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TABLE 8.4

DESIGN OF SANITARY SEWERS

LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation Upper Manhole	Ground Elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity Flowing full (cfs)	Invert elevation Upper Manhole	Invert elevation Lower Manhole
Main	A-MH 5	A-MH 4	300	.058	0.145	184.23	183.56	8	.0032	2.00	0.70	180.50	179.54
Line	4	3	300	0.110	0.276	183.56	183.00	8	.0032	2.00	0.70	179.54	178.58
A	3	2	300	0.150	0.375	183.00	181.19	8	.0032	2.00	0.70	178.58	177.62
	2	1	300	0.180	0.450	181.19	180.22	8	.0032	2.00	0.70	177.62	176.66
	1	T-MH 56	300	0.200	0.500	180.22	179.32	8	.0032	2.00	0.70	176.66	175.70
Main	L-MH 8	L-MH 7	250	0.208	0.520	173.00	173.00	8	.0032	2.00	0.70	169.27	168.57
Line	7	6	250	0.286	0.715	173.00	173.00	8	.0035	2.10	0.725	168.57	167.70
L	6	5	250	0.348	0.870	173.00	173.00	10	.0025	2.00	1.10	167.62	167.00
	5	4	200	0.402	1.005	173.00	173.00	10	.0025	2.00	1.10	167.00	166.50
	4	3	200	0.437	1.090	173.00	173.00	10	.0025	2.00	1.10	166.50	166.00
	3	2	200	0.494	1.230	173.00	173.00	12	.0020	2.10	1.60	165.92	165.52
	2	1	200	0.520	1.300	173.00	173.00	12	.0020	2.10	1.60	165.52	165.12
	1	ST-MH18	200	0.53	1.325	173.00	173.00	12	.0020	2.10	1.60	165.12	164.72

.../171.

TABLE 8.4

DESIGN OF SANITARY SEWERS

LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation Upper Manhole	Ground Elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity Flowing full (cfs)	Invert elevation Upper Manhole	Invert elevation Lower Manhole
Main	A-MH 5	A-MH 4	300	.058	0.145	184.23	183.56	8	.0032	2.00	0.70	180.50	179.54
Line	4	3	300	0.110	0.276	183.56	183.00	8	.0032	2.00	0.70	179.54	178.58
A	3	2	300	0.150	0.375	183.00	181.19	8	.0032	2.00	0.70	178.58	177.62
	2	1	300	0.180	0.450	181.19	180.22	8	.0032	2.00	0.70	177.62	176.66
	1	T-MH 56	300	0.200	0.500	180.22	179.32	8	.0032	2.00	0.70	176.66	175.70
Main	L-MH 8	L-MH 7	250	0.208	0.520	173.00	173.00	8	.0032	2.00	0.70	169.27	168.57
Line	7	6	250	0.286	0.715	173.00	173.00	8	.0035	2.10	0.725	168.57	167.70
L	6	5	250	0.348	0.870	173.00	173.00	10	.0025	2.00	1.10	167.62	167.00
	5	4	200	0.402	1.005	173.00	173.00	10	.0025	2.00	1.10	167.00	166.50
	4	3	200	0.437	1.090	173.00	173.00	10	.0025	2.00	1.10	166.50	166.00
	3	2	200	0.494	1.230	173.00	173.00	12	.0020	2.10	1.60	165.92	165.52
	2	1	200	0.520	1.300	173.00	173.00	12	.0020	2.10	1.60	165.52	165.12
	1	ST-MH18	200	0.53	1.325	173.00	173.00	12	.0020	2.10	1.60	165.12	164.72

.../171.

LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q max. (cfs)	Ground Elevation Upper Manhole	Ground Elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity Flowing full (cfs)	Invert elevation Upper Manhole	Invert elevation Lower Manhole
Sub-	ST-MH 18	ST-MH 17	300	0.53	1.325	173.00	174.35	12	.0020	2.1	1.6	164.72	164.12
Trunk	17	16	300	0.98	2.450	174.35	174.30	15	.0015	2.05	2.5	164.04	163.59
Line	16	15	300	0.98	2.450	174.30	174.10	15	.0015	2.05	2.5	163.59	163.14
	15	Lift St:W.W.	100	0.98	2.450	174.30	174.30	15	.0015	2.05	2.5	163.14	163.00
	L.Station W.W.	14	150	0.98	2.450	174.10	174.30	15	-	-	-	-	170.00
	14	13	300	0.98	2.450	174.30	174.00	15	.0015	2.05	2.5	170.00	169.55
	13	12	300	0.98	2.450	174.00	174.00	15	.0015	2.05	2.5	169.55	169.10
	12	11	300	0.98	2.450	174.00	173.00	15	.0015	2.05	2.5	169.10	168.65
	11	10	300	1.35	3.375	173.00	173.00	18	.0011	2.00	3.5	168.57	168.24
	10	9	300	1.35	3.375	173.00	173.00	18	.0011	2.00	3.5	168.24	167.91
	9	8	300	1.35	3.375	173.00	173.00	18	.0011	2.00	3.5	167.91	167.58
	8	7	300	1.35	3.375	173.00	173.00	18	.0011	2.00	3.5	167.58	167.25
	7	6	300	1.35	3.375	173.00	173.00	18	.0011	2.00	3.5	167.25	166.92
	6	5	300	2.04	5.100	173.00	173.00	18	.0024	2.80	5.1	166.84	166.12

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LINE	Manholes From	To	Length (feet)	Q ave. (cfs)	Q Max. (cfs)	Ground Elevation Upper Manhole	Ground Elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity flowing full (cfs)	Invert Elevation Upper Manhole	Invert Elevation Lower Manhole
	5	4	300	2.04	5.100	173.00	173.00	18	.0024	2.80	5.1	166.12	165.40
	4	3	300	2.04	5.100	173.00	173.00	18	.0024	2.80	5.1	165.40	164.68
	3	2	300	2.33	5.825	173.00	173.00	18	.0032	3.40	5.9	164.68	163.72
	2	1	300	2.33	5.825	173.00	173.00	18	.0032	3.40	5.9	163.72	162.76
	1	Lift St: W.W.	125	2.33	5.825	173.00	173.00	18	.0032	3.40	5.9	162.76	162.36
Trunk T-MH	56	T-MH 55	300	0.20	0.500	179.32	178.87	8	.0032	2.00	0.70	175.62	174.68
Line	55	54	300	0.20	0.500	178.87	178.35	8	.0032	2.00	0.70	174.68	173.72
	54	53	300	0.20	0.500	178.35	177.83	8	.0032	2.00	0.70	173.72	172.76
	53	52	300	0.20	0.500	177.83	177.47	8	.0032	2.00	0.70	172.76	171.80
	52	51	175	0.20	0.500	177.47	177.29	8	.0032	2.00	0.70	171.80	171.24
	51	50	300	0.57	1.425	177.29	176.92	10	.0045	2.80	1.50	171.16	169.81
	50	49	225	0.75	1.875	176.92	176.28	12	.0030	2.50	1.95	169.73	169.01

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LINE	Manholes From	To	Length (feet)	Q ave. (cfs)	Q Max. (cfs)	Ground elevation Upper Manhole	Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity flowing full (cfs)	Invert Elevation Upper Manhole	Lower Manhole
	49	48	225	0.75	1.875	176.28	175.64	12	.0030	2.5	1.95	169.01	168.38
	48	47	200	1.21	3.025	175.64	175.00	15	.0022	2.55	3.1	168.28	167.82
	47	46	300	1.21	3.025	175.00	174.40	15	.0022	2.55	3.1	167.82	167.16
	46	45	300	1.21	3.025	174.40	173.80	15.	.0022	2.55	3.1	167.16	166.49
	45	44	225	1.42	3.550	173.80	173.35	18	.0012	2.1	3.6	166.39	166.11
	44	43	225	1.42	3.550	173.35	173.00	18	.0012	2.1	3.6	166.11	165.84
	43	42	225	1.61	4.025	173.00	173.05	18	.006	2.35	4.1	163.80	165.47
	42	41	250	1.61	4.025	173.05	173.00	18	.0015	2.35	4.1	165.43	165.06
	41	40	150	1.61	4.025	173.00	173.00	18	.0015	2.35	4.1	165.06	164.83
	40	39	300	1.61	4.025	173.00	173.00	18	.0015	2.35	4.1	164.83	164.36
	39	38	300	2.16	5.400	173.00	173.00	21	.0012	2.3	5.5	164.26	163.90
	38	37	300	2.16	5.400	173.00	173.00	21	.0012	2.3	5.5	163.90	163.54
	37	36	250	2.16	5.400	173.00	173.00	21	.0012	2.3	5.5	163.54	163.24
	36	35	250	2.16	5.400	173.00	173.00	21	.0012	2.3	5.5	163.24	162.94
	35	34	200	2.16	5.400	173.00	173.00	21	.0012	2.3	5.5	162.94	162.70

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LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q Max. (cfs)	Ground elevation Upper Manhole	Ground elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity flowing full (cfs)	Invert Elevation Upper Manhole	Invert Elevation Lower Manhole
	34	33	200	2.49	6.225	173.00	173.00	21	.0015	2.65	6.25	162.70	162.40
	33	LIFT STATION W.W.	125	2.62	6.550	173.00	173.00	21	.0018	2.82	6.75	162.40	162.17
	LIFT STATION W.W.	32	250	4.95	12.375	173.00	173.00	24	-	-	-	-	168.00
	32	31	225	4.95	12.375	173.00	173.00	24	.003	3.9	12.5	168.00	167.33
	31	30	225	4.95	12.375	173.00	173.00	24	.003	3.9	12.5	167.33	166.66
	30	29	300	4.95	12.375	173.00	172.79	24	.003	3.9	12.5	166.66	165.76
	29	28	300	4.95	12.375	172.79	172.15	24	.003	3.9	12.5	165.76	164.80
	28	27	300	4.95	12.375	172.15	171.41	24	.003	3.9	12.5	164.80	163.90
	27	26	300	4.95	12.375	171.41	170.92	27	.0016	3.1	12.5	163.86	163.38
	26	25	300	4.95	12.375	170.92	170.36	27	.0016	3.1	12.5	163.38	162.99
	25	24	300	4.95	12.375	170.36	169.80	27	.0016	3.1	12.5	162.99	162.42
	24	23	300	4.95	12.375	169.80	169.22	27	.0016	3.1	12.5	162.42	161.84
	23	22	300	4.95	12.375	169.22	168.46	27	.0016	3.1	12.5	161.84	161.36

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LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q Max. (cfs)	Ground elevation Upper Manhole	Ground elevation Lower Manhole	Dia. (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity flowing full (cfs)	Invert Elevation Upper Manhole	Invert Elevation Lower Manhole
	22	21	300	4.95	12.375	168.46	167.26	27	.0016	3.1	12.5	161.36	180.88
	21	20	300	4.95	12.375	167.26	166.52	27	.0016	3.1	12.5	160.88	160.40
	20	19	200	4.95	12.375	166.52	166.08	27	.0016	3.1	12.5	160.40	160.08
	19	18	200	4.95	12.375	166.08	165.60	27	.0016	3.1	12.5	160.08	159.76
	18	17	300	5.65	14.125	165.60	165.81	27	.0022	3.6	14.5	159.72	159.10
	17	16	300	5.65	14.125	165.81	165.18	27	.0022	3.6	14.5	159.14	158.44
	16	15	300	5.65	14.125	165.18	164.76	27	.0022	3.6	14.5	158.44	157.78
	15	14	300	5.65	14.125	164.76	164.22	27	.0022	3.6	14.5	157.78	157.12
	14	13	200	5.90	14.75	164.22	163.87	27	.0024	3.75	15.0	157.12	156.64
	13	12	200	5.90	14.75	163.87	163.52	27	.0024	3.75	15.0	156.64	156.16
	12	11	300	5.90	14.75	163.52	162.97	27	.0024	3.75	15.0	156.16	155.44
	11	10	300	6.68	16.70	162.97	162.76	30	.0017	3.45	17.0	155.36	154.93
	10	9	300	6.68	16.70	162.76	162.55	30	.0017	3.45	17.0	154.93	154.42

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LINE	Manholes From	Manholes To	Length (feet)	Q ave. (cfs)	Q.Max. (cfs)	Ground Elevation Upper Manhole	Ground Elevation Lower Manhole	Dia (inch)	Grade of Sewer	Velocity flowing full (ft/sec)	Capacity flowing full (cfs)	Invert Elevation Upper Manhole	Invert Elevation Lower Manhole
	9	8	300	6.68	16.70	162.55	162.30	30	.0017	3.45	17.0	154.42	153.91
	8	7	300	6.68	16.70	162.30	162.08	30	.0017	3.45	17.0	153.91	153.49
	7	6	300	7.75	19.375	162.08	161.86	30	.0022	3.9	19.5	153.40	152.74
	6	5	300	7.75	19.375	161.86	161.65	30	.0022	3.9	19.5	152.74	152.08
	5	4	200	7.75	19.375	161.65	161.49	30	.0022	3.9	19.5	152.08	151.64
	4	3	300	8.03	20.075	161.49	161.28	30	.0025	4.15	20.5	151.64	150.89
	3	2	300	8.03	20.075	161.28	161.07	30	.0025	4.15	20.5	150.89	150.14
	2	1	300	8.03	20.075	161.07	160.44	30	.0025	4.15	20.5	150.14	149.39
	1	Treatment Plant sump well	100	8.03	20.075	160.44	160.00	30	.0025	4.15	20.5	149.39	149.14

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8.7. Design of Lift Stations

Centrifugal pumps of the non-clog type are proposed to be installed for lifting the sewage to a level where from it can run under gravity to the desired point of collection or treatment plant.

The pumps will have their suction from a wet well or sump which will be constructed by the side of a dry well, where pumps will be installed. Electrical prime movers will be directly coupled to these pumps.

Two such stations will be required in the sewerage system. One such station is located in between the manholes numbering 12 and 13 on the sub-trunk line.

Another station will be located between manholes numbering 32 and 33.

These lift stations have been provided to avoid very deep excavations and letting the sewer run under water. Since the water table happens to be between the range of 10 - 12 feet below ground level, efforts have been made not to lay beyond 10-11 feet below ground level.

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In order to avoid lengthy processes of screening and grit chamber preceding wet well, as is the usual practice in pumping installations; it has been decided to provide a compact type pumping units which are able to lift the raw sewage without causing clogging or other damages to the pumps. Sump screens are usually built in the units itself which convert the large size solids, to the sizes which do not clog the pump and are taken through the pump along with the flow.

Of the many, pumping units as supplied by "Smith & Lovelles"[31] or approved equal are selected for the purpose. These pumps are supplied in standard sizes for constant speed, multi-speed and variable speed, having capacities from 100 G.P.M. to 4500 G.P.M. Higher capacity pumps can be obtained on order.

The manufacturer has claimed the following additional advantages for the units such as :

i) They are designed to make the maintenance, man's job, easier, faster, safer.

ii) Operating and maintenance instructions are easy to understand.

iii) Most reliable control system ever built.

iv) Ample "elbow" room is provided around the pumps.

v) Pumps are non-clog and vertical close coupled.

vi) Unit is precision manufactured for heavy duty sewage service specially designed for ease of maintenance.

vii) Units are dependable and require trouble free service.

viii) Units are completely factory built designed to be installed directly in sewage lift stations.

a) Sub-Trunk line lift station.

Ultimate average sewage flow = 0.98 cfs = 440.0 g.p.m.

Ultimate maximum sewage flow = 2.450cfs =1100.0 g.p.m.

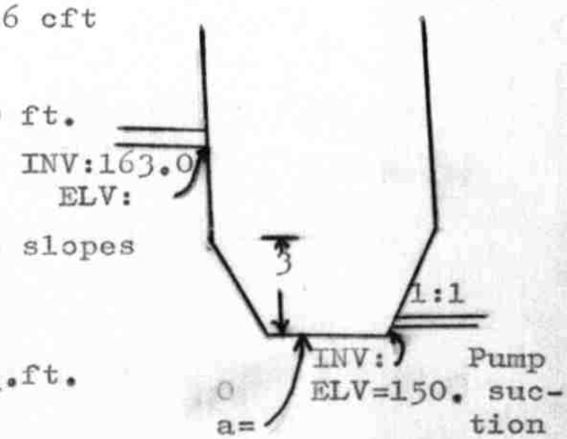
Present average flow = 440 x 0.44 = 194.0 g.p.m.

Present maximum flow = 1100 x 0.44 = 484.0 g.p.m.

<u>W. Well.</u>

Taking 10-minute capacity for wet well

Volume required = $\frac{10 \times 440}{7.48}$ = 586 cft



Taking length of w.well as 25.0 ft.

& working depth as 3.0 feet

Width "a" at the base with side slopes of 1:1 will be:

x-section area = $\frac{586}{25}$ = 23.44 sq.ft.

$\therefore$ 23.44 = $\frac{a + a + 6}{2} \times 3 = (a + 3) \times 3$

or a + 3 = 7.81

or a = 7.81 - 3 = 4.81 say 5.0 feet

∴ Width at W.level = 5 + 6 = 11.0 feet.

Working depth of W.well = 3.0 ft. with 1.0 free board below sewer invert.

Pumping Head.

Inv: level at discharge line = 170.0

Inv: level at suction line = 159.0

∴ Static lift = 170.0 - 159.0= 11.0 feet.

Taking pumping and other losses as 5.0 feet

Total required lift = 11.0 + 5.0 = 16.0 feet.

Selection.

Thus for the present three pump having capacities of 200, 300 and 500 g.p.m. respectively will be installed. Of these three one having capacity of 200 g.p.m. will be working for average low flows and a pump of 300 g.p.m. capacity when put in parallel operation with this pump will be able to meet ~~or~~ peak demands. 500 g.p.m. capacity will be working as standbye.

With an increase in demand any two ofthe three pumps will be put in parallel operation leaving third as standbye.

Thus by the end of the designed period i.e. by the year 2007, two pumps of 200 and 300 g.p.m. capacity will be replaced by two 600.0 g.p.m. capacity pumps such that one of these will be working as standbye and one with the existing 500 g.p.m. pump will be able to care for peak ultimate demands.

Thus in all the following units will be used:

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200 g.p.m. capacity unit	1 No.
300 g.p.m. capacity unit	1 No.
500 g.p.m. capacity unit	1 No.
600 g.p.m. capacity unit	2 Nos.

Complete details of units will be supplied by the manufacturer.

b) Trunk line lift station.

Ultimate average flow = 4.95 cfs = 2230 g.p.m.

Ultimate maximum flow = 12.375 cfs=5575 g.p.m.

Present average flow =2230 x 0.44= 980 g.p.m.

Present maximum flow =5575 x 0.44=2450 g.p.m.

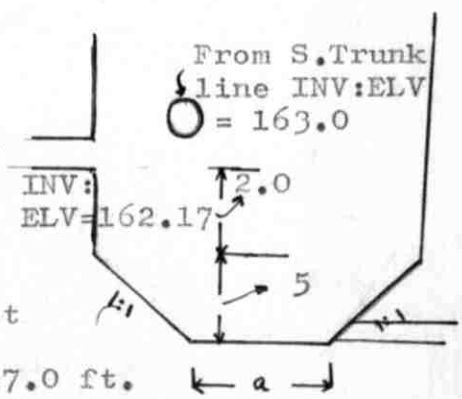
W. Well.

Taking 5 minutes capacity for wet well

Volume required = $\frac{5 \times 2230}{7.48}$ = 1490 cft.

Taking length of w.well as 25.0 ft. and working depth as 5 feet, width "a" at the base with side slopes of 1:1 will be:

X-section area = $\frac{1490}{25}$ = 59.5 sq.ft.



59.5 = $\frac{a + a + 10}{2}$ x 5 = (a + 5) x 5

$\therefore$ a + 5 = 11.9

or a = 11.9 - 5.0 = 6.9 say 7.0 feet

$\therefore$ Width at water level = 7 + 10 = 17.0 ft.

Working depth of wet well will be 5.0 feet

With a free board of 2.0 ft. below the lowest invert of inlets, which is 162.17 for trunk line from Zone A.

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Pumping head.

Invert level at discharge line = 168.0

Invert at suction line = 155.17

$\therefore$ Static lift = 168.0 - 155.17 = 12.83 say 13.0 ft.

Taking pumping and other loss as 5 ft.

Total head required = 13.0 + 5.0 = 18.0 feet.

Selection.

For the present three pumps having capacities of 1000, 1500 and 2500 g.p.m. respectively will be installed. Of these three one having capacity of 1000 g.p.m. will be working for average flow and a pump of 1500 g.p.m. capacity when put in parallel operation with this pump, will be able to meet peak demands. 2500 g.p.m. capacity pump will be working as standbye.

With an increase in demand any two of the three pumps will be put in parallel operation leaving third as standbye.

Thus by the end of the design period i.e. by the year 2007 two pumps of 1000 and 1500 g.p.m. capacity, which have bared their useful life by this time, will be replaced by two 3000 g.p.m. capacity pumps such that one of these will be working as standbye and one with the existing 2500 g.p.m. pump will be able to care for peak ultimate flows.

Thus in all the following units will be used:

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1000 g.p.m. capacity unit	1 No.
1500 g.p.m. capacity unit	1 No.
2500 g.p.m. capacity unit	1 No.
3000 g.p.m. capacity unit	2 Nos.

Complete details of the units will be supplied by the manufacturer.

c) Operation of the units.

Automatic pump control,system will be adopted. Working of the system is outlined as under, numbers refer to those given in Fig. 8.2.

A compressor, 1, supplies a stream of air through the air-bubbler pipe, 2, to the wet well. As sewage rises, back pressure in the line actuates "low level" mercury pressure switch, energizing a magnetic starter and starting one of the pumps, 3. Sewage flows from the wet well, through the suction pipe, 4, a gate valve, 5, into the pump, 6, out through check valve, 7, discharge gate valve, 8, and discharge force main,9.

If the sewage continues to rise above the starting level of the first pump, a "high level" mercury pressure switch actuates the starter of the second pump and both pumps operate until the wet well level drops to a pre-determined shut-off point.

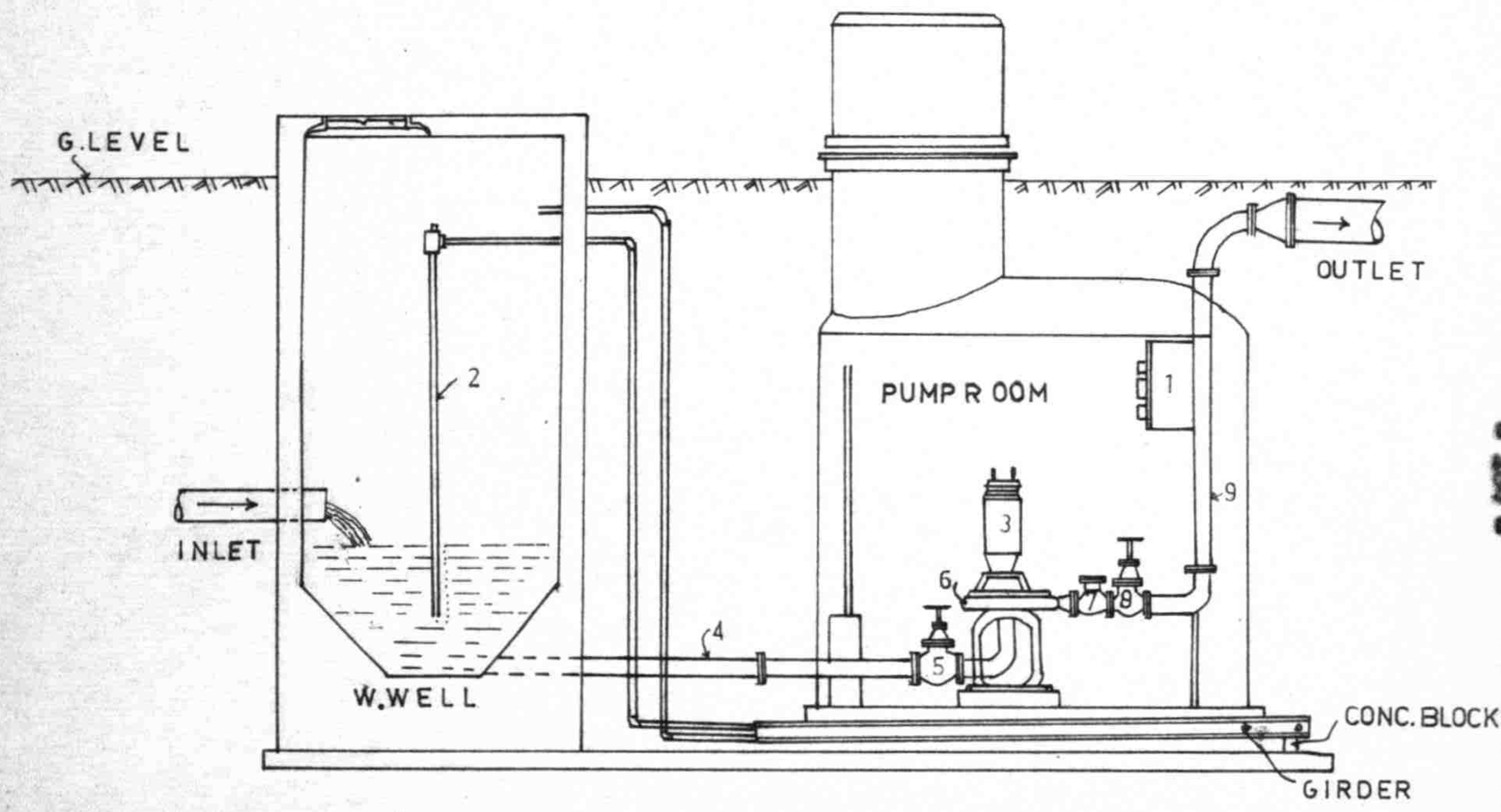

FIG. 8·2 A SECTION THROUGH LIFT STATION.

CHAPTER 9

SEWAGE TREATMENT

The main considerations of the public health and welfare makes it not only important, but a must, to resort to some satisfactory means and methods of town sewage disposal. Sewage in addition to pathogenic bacteria contains highly putrescible matter like: fecas, bit of garbage, decaying fruits, and useless and discarded material. All these materials decompose quickly specially in warm weather with the production of abnoxious smells.

It is, therefore, desirable to adopt certain means and methods, which are economical and suitable for local conditions, to dispose off the sewage and thus avoid in sanitary conditions and health hazards in the area.

In the following, therefore, a brief review of the various methods usually adopted will be outlined and the one, which could be used for our purpose, will be selected.

9.1. Methods of Disposal

Basically there are two methods of disposal of sewage: I - Disposal of sewage without treatment.

II - Disposal of sewage with treatment.

Each of these methods can further be sub-divided

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as follows:

I - Disposal of sewage without treatment.

a. Dispdsal into water (dilution)

b. Disposal on land.

II - Disposal of sewage with treatment.

a. Imhoff tank.

b. Oxidation ditch.

c. Activated sludge process.

d. Trickling filter.

e. Sewage stabilization ponds or lagoons.

In the following paragraphs, an attempt will be made to describe each of these methods of disposal, their merits and demerits in general, and their possibility of adoption in our case.

9.1.1 Disposal of Sewage without treatment.

A) Disposal into water.

The sewage is discharged into large bodies of water where transformation of organic matter into stable organic substances takes place by natural agents under aerobic and anaerobic conditions without causing nuisance. The dilution factor, the velocity and the direction of wind are important factors which should be taken into account, otherwise water pollution, formation of heavy sludge deposits on banks, and unsightly floating matter, foaming and frothing caused by grease and soaps, and may more nuisance problems are likely to occur.

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This method is most economical when local conditions permit. Normally such type of disposal is recommended only when a dilution factor of 500:1, normally prescribed for rivers, is obtainable. Other nuisance problems can be reduced by screening before disposal, and disposal sufficiently off-shore.

In our case, since the nearest body of water is a canal such dilution etc. is not possible to achieve, moreover the canal is not a perinal and remains dry for few days during closure period. Hence the discharge during such closure period will cause a number of problems as described above.

B) Disposal on Land.

This method consists in discharge of sewage on the ground surface, part of it evaporates while the remainder percolates down the subsoil to the ground water level.

To control bad odours, percolation through subsurface drainage, which consists of sewage being discharged through perforated pipes buried underground can be resorted to.

Sub-surface water pollution chances are less as the sewage through its process of percolation gets filtered and the unstable organic matter gets stabilized by soil bacteria and oxygen and pathogenic bacteria die out due to longer process.

This method can best be adopted where there is a cheap available land, lower rainfall and lower water table.

In our case, the first two conditions, are best met with, but the third one is lacking. Since water table is high as such early saturation of whole surface with sewage will take place, percolations and stabilization of organic matter will not effectively be achieved and sewage stagantion over ground will occur thus causing in-sanitary conditions. Hence this method can not be adopted.

From the above it is, therefore deduced that disposal of sewage without treatment can only be adopted when a certain set of natural or environment conditions are fulfilled. Once any one of the requirements happens to lack, in-sanitary conditions are created in terms of anaesthetic and unstabilized organic matter causing bad smells and other nuisance problems.

It is, therefore, a must to stabilize the putrescible organic matter, either through some artificial means, so called treatment processes, or through some modifications in natural processes to enhance the action which otherwise is long time taking, before it is finally disposed.

Thus to resort to some sort of treatment is a must and same will be the condition in our case.

9.1.2 Disposal of Sewage with~~out~~ treatment.

There are a number of different treatment processes as listed above and each has its merits and demerits and its best adaptability in certain natural and environmental conditions. However, the selection of the one, should be based on certain sound factors, which are described hereunder together with the various methods of treatments.

9.2. Factors affecting choice of a Treatment Method

Following are the various factors:

a) The method of final disposal.

b) The efficiency and characteristics of the treatment process. The criterion must be the quality of the effluent produced.

c) Available financial resources.

d) Constructional, operational, and maintenance costs.

e) Quality of the sewage to be treated.

f) Required skill in operation, and availability of personnel of required caliber.

g) Available facilities for, maintenance, repair, and procurement of required equipment in the local market.

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

i) Possibility of future expansion of the treatment plant.

j) Natural and environmental conditions.

k) Quantity and quality of sludge to be handled.

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9.3. Methods of Sewage Treatment

In the following, we will describe in brief the various treatment methods usually adopted and evaluated the use of one which will suit our conditions best.

a) Imhoff Tank.

This method which was used before the development of the sedimentation and the sludge-digestion process has been superseded by modern and efficient processes except in small installations for 1000 persons or less because the new methods permit easier control of operation and give better results. It consists of two compartments in which sedimentation and anaerosic decomposition of sludge is accomplished. It is not really a cheaper process of treatment as claimed because for complete purification, filter beds and secondary sedimentation units will have to be added particularly in our case, thus adding to the cost. Reduction in B.O.D. is not very high.

Foaming, unsightly conditions and odours greatly hinder the operation and maintenance.

It is also found that this method requires constant and continuous supervision for successful operation. Also the cost and results of separate digestion tanks are more attractive.[29]

Thus it is observed that this method does not provide special attraction for adoption in the town of Khairpur.

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b) Oxidation ditch.

It is a modified form of the activated sludge process and may be classified in the extended aeration group. This type of plant produces B.O.D. reduction of 90% or better and lower initial cost.(32) Under normal condtions, cost of this type of plant is 20 - 33% below that normally expected for other treatment plants, capable of producing an equivalent quality effluent. Area required for similar population by stabilization lagoons is 97% more than oxidation ditch.(33)

It has a number of advantages such as: simple operation, simple construction, low maintenance, and less area requirements. It has its main disadvantages of constant use of power for rotor.

However, this method has not been used for communities exceeding 12,000 persons and its feasibility for higher populations has not been justified yet.

It is therefore, felt that its adoption as a treatment process for the town of Khairpur willnot be justified.

c) Activated sludge Process.

This method has a number of advantages such as:

i) Producing a clear, spark-ling, and non-putrescible effluent.

ii) Freedom from odours during operation.

iii) Less area requirements.

iv) High fertilizing value of the sludge.

v) B.O.D. and suspended solids removals upto 85-95%, and bacteria removal upto 90-98%.(33)

Although the method seems to be ideal for adoption yet the following disadvantages restrict its use. The disadvantages are :-

i) Lack of adaptability to variation in strength and composion of sewage.

ii) Requires highly skilled personnel for operation.

iii) Higher operational costs.

iv) Production of greater quantities of sludge and difficulty in dewatering and disposing.

v) The need for continuous and skilled attendance.

d) Filtration.

This process consists of three stages viz: Primary treatment, filtration and final sedimentation. Among the advantages which it possesses over other methods of treatment are :-

i) Relatively low operational costs.

ii) Its ability to function under extreme weather conditions.

iii) Resistance to shock loads.

iv) Production of sludge in final settling is low.

v) Good performance with a minimum of skilled technical supervision.

vi) Little problems due to foaming.

Inspite of such advantages there a a number of disadvantages and the most important one is the head loss through it, which varies from 5 to 11 feet in addition to depth of filter which is usually betwwen 6 to 10 feet.[29] Other disadvantages which it includes over the activated sludge process are :-

a) Odour and fly breeding.

b) Relatively larger area required.

c) High initial cost.

Thus the method seems to be feasible for adoption for the moment till some other method which is comparatively economical and less skilled personnel requiring is found.

Off-hand these two factors viz: economy and skilled labour are the two decisive factors for adoption of any method of treatment at Khairpur.

e) Sewage Stabilization Lagoons.

Oxidation ponds or lagoons are the most simple and cheap treatment processes and are claimed to treat successfully either settled or raw sewage with effluents at least equal to those from sewage treatment plants that provide complete treatment, but these require large areas of land which is not a problem in our case.

It has the number of advantages as described below:

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i) Operation of ponds does not require either constant supervision or skilled personnel to look after.

ii) No mechanical equipments are involved, like rotor in oxidation ditch, except few pumps are required for lifting the sewage if the ground elevations at treatment plants dictate so.

iii) The treated effluent can either be discharged into a body of water, or else utilized for irrigation purposes as the same has high fertilizer value.

iv) There is no sludge problem and desludging of ponds' bottoms is not required. Figures show that for a 10 acre pond, it will take 135 years to build up a one foot layer of sludge in the bottom of the pond.[34] In our case, as regards land which is the main feature of this system, it is to say that there is plenty of barren land in the southern portion of Khairpur where at present the nearby textile mills and other factories discharge their untreated wastes into low lying nearby areas in a haphazard way which can; and is responsible for a number of problems like: stagnation of waste, mosquito breeding, pollution of ground waters, and raising the water table. However, no health hazards fortunately have been caused as the area is remote from inhabitant locality. Hence by providing this system of treatment, we can on one hand control the haphazard condition and on the other hand irrigate the hundreds of acres of barren land and thus bring to the local munipipal committee authorities a very handsome revenue.

It is also to be noted here that West Pakistan Public Health Engg: Deptt: has also recommended such system of treatment as mentioned in the rough cost estimate for sewerage scheme for Khairpur.

It is, therefore, decided that we will adopt oxidation lagoons as the only system of treatment of sewage.

Lower rainfall, higher temperatures, plenty of sunshine, and clayey deposits, in the area as described in Chapter I, are further factors which support the adoption of the stabilization lagoons as the only method of treatment most feasible for the area.

9.4. Recipient Conditions

Mir wah Canal could be the only receiving body of water. It runs in the middle of the town, the discharge data at head works for Khairpur East feeder are given in the Table 4.6. The actual discharge for the canal is much less as this feeder feeds two other canals. Mir wah canal is a non-perennial canal and remains closed during its closure period for maximum of 15 days in a month, in certain months. The difference between the maximum water elevation in the canal and the general ground elevation at the proposed site for treatment works is more than 20 feet. Therefore, the effluent discharge into canal will require pumping and thus subsequent additions in operational costs.

Thus direct utilization of effluent for agricultural purposes of the nearby barren lands seems to be more feasible.

9.5. Location of Treatment Plant

The main considerations which govern the selection of treatment plant location include topography, soil conditions, proximity to the receiving body of water, direction of wind, ground water table conditions, remoteness from the inhabited areas and danger from flooding. Location of treatment plant in our case is fixed having the following considerations in mind:

a) The topography of area dictate such location.

b) It is remote from the inhabited areas.

c) Plenty of cheap barren land is available in the area.

d) The direction of nearby canal flow is in this direction.

e) Effluent can best be utilized for irrigation of the barren lands.

f) It is in the wind direction which is away from inhabitation.

g) Sub-soil exploration show clayey soil, which is best suited for a stabilization lagoon.

9.6. Available Financial Resources

The Municipal Committee Khairpur will wholly be responsible for all the operational expenses of the scheme, however, on third of the constructional costs will be paid by the West Pakistan Government and execution of the works will be done by the W. Pak: P.H. Engg: Deptt: The income of municipal committee is very much limited, the city is yet in its stages of development, hence cannot be able to pay for any heavy operational expenses. In our case, the proposed system is expected to require less operational costs and offer more returns through the revenue of lands irrigated by the effluent.

9.7. Extent of Treatment

Sewage treatment works are designed to reduce the strength of sewage to a value which may be expected to ensure avoidance of nuisance under the conditions in which sewage is discharged. The extent of treatment, however, depends upon the required quality of the effluent which is governed by the following factors:

a) Regulatory agencies.

In developed countries like United States and the United Kingdom, etc. there are limits setforth by the authorities for the quality of effluent which could be discharged. There such limits govern the design of treatment plants. As for example, in the United Kingdom,

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for instance, the Royal Commission on sewage Disposal has laid down as a general standard for satisfactory sewage "effluents as not to exceed 20 P.P.M. and 30 P.P.M. in its 5-day B.O.D. and suspended solids content respectively."(35) In Pakistan however, there are no such limits set forth by the authorities so far, hence in design, one has to consider that effluent produced is at least of such a quality that it does not create any nuisance and health hazard problems.

b) Receiving body of water.

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

The possible receiving body as stated above remains dry for a major portion of the year and since effluent utilization for agricultural purposes seems to be more beneficial as such its discharge into a receiving body of water will not be considered.

c) Effluent utilization.

The effluent from the treatment plant may be used for irrigation, recharge of groun-water reservoirs, industrial purposes, and even for domestic use, in water shortages. In our case there is a major possibility of utilization of effluent for irrigation of nearby lands.

Thus it is apparent that higher B.O.D. reduction should in any case be achieved to safe-guard one self from every short coming. Hence strength will be reduced upto

20-30 PPM which can easily be achieved by the proposed treatment process.

9.8. Conclusion

The comparison of the relative merits and demerits of the different processes of treatment of sewage and their suitability for the area clearly dictates that the most economical, and suitable method of sewage treatment will be by "Sewage Stabilization Lagoons". This will require minimum finance for construction, operation and maintenance, and will give effluent of low B.O.D. suitable of direct discharge into nearby lands for agricultural purpose.

Further aspects of the method will be covered in detail in subsequent chapters.

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CHAPTER 10

HISTORICAL DEVELOPMENT, & DESIGN THEORIES OF SEWAGE STABILIZATION LAGOON

10.1. Introduction

Before going into the history of sewage lagoons, it might be well to briefly define what is meant by the term Lagoon as used in this discussion. Various names have been given to an open pond system which receives sewage, such as: sewage stabilization Lagoons, waste stabilization ponds, oxidation ponds and lagoons and a variety of others. For the consistency sake in this report, the nomenclature adopted will be "Sewage stabilization Lagoons", defining it as: " A man engineered pond of controlled shape, area, and depth, designed and constructed for the purpose of treating and stabilizing sewage to a pre-determined standard of purity in the most efficient and economical manner.[34]

It is further assumed that oxidation ponds are considered to be those ponds that are designed for the purpose of receiving, at least, primarily settled sewage. This may be from an ordinary sewage treatment plant, imhoff tank, septic tank, and the like. Sewage lagoons, however, receive raw sewage from a municipality including industrial wastes.[36]

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10.2. Historical Development

Sewage lagoons are reported to have been known in Asia for centuries and yet the earliest data available in their performance dates back to only half a century ago.(34)

During the mid-twenties, cities in California, Texas, North Dakota and probably other states, used lagoons as a means of treating municipal sewage, however, in each case it seemed to be more by accident than design.(36)

In U.S., history seems to go to the year 1924, when the first pond accidently originated with an emergency discharge of sewage to a basin dug in old creek bed at Santa Roaa in California.

From 1925-1950 which was called as a "transition period" can really be counted as a period during which more knowledge was acquired as to their principles, operation, and the advantages that resulted from their use at different places.

By the year 1955, there was little, if any, controverysy with regards to the use of sewage lagoons, except in a few isolated cases. By that time, there were nearly 100 lagoons in the Missouri Basin States.(36)

A final report published in 1957 indicated the success of sewage lagoons even though some of these studied were not built in accordance with accepted design.(36)

During this same period, the U.S.P.H.S. further babked their interest in lagoon research by constructing a group of ponds at Lebanon and Ohio. The results of these experimental works had considerable influence on their adoption in the lower Missouri Basin.(35)

Nowadays, an ever increasing popularity in their use is reflected by the countless number of installation so far built recently and planned for construction in various parts of the world.

10.3. Fields of Application

Since the beginning, stabilization lagoons have been used to perform a number of purposes viz: as an emergency holding ponds, an equalization basins for industrial wastes, temporary method of treatment, and polishing of conventional primary and secondary effluent. It is only recently that their use as a final and complete treatment process came into practice.

It was realized that in addition to providing decided advantages of high degree of treatment, low initial capital investment, and low operational and maintenance costs, ponds possess flexible features by which they are able to cope with the treatment of ~~to~~ various types of sewage and industrial wastes, and their adaptability of being resited with land reclamation for housing or industrial site development in areas that are subject to rapid population growth.(34)

10.4. Mechanisms of Stabilization

Before detailing the various design theories to provide the basis on which actual design of treatment unit could be based, it will not be out of place to briefly describe the mechanism involved in the treatment process and thus enable the designer and reader to be provided with the basic ideas and understanding of the mechanism and also ofthe various factors which effect his design.

Sewage treatment through stabilization Lagoon is the result of various natural mechanisms which involve certain physical, chemical and mainly biological agencies. Fundamentally it is similar to any other biological treatment process, however, here all the reactions physical, chemical and biological take place in one single unit.

The stabilization mechanism involves an oxidation reduction process. The degree of reduction and stabilization of the organic compounds is proportional to the metabolic conversion process of the organisms. Two main processes are usually associated with metabolism, a process of synthesis and a process of respiration.

The synthetic process involves the intake, digestion, and assimilation of food into tissue and waste products, while the process of respiration involves the oxidation of carbohydrates to change the potential energy into kinetic energy with waste products resulting from this process as well. It can be expressed in general as:[34]

Food + organisms + $O_2 \longrightarrow$ more organisms + H_2O + CO_2 + nitrogenous wastes + Energy

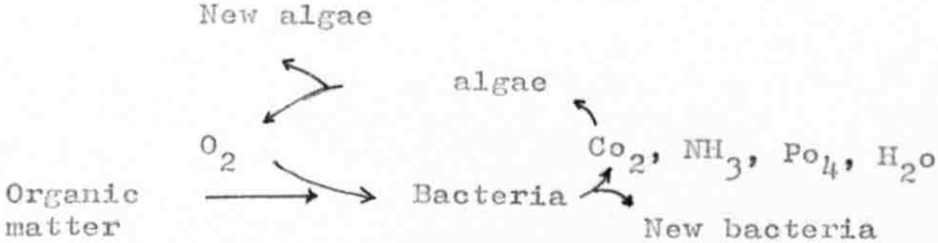
Thus the ingredients of the process are oxygen, food and micro organisms.

The necessary micro organisms are found in sewage to start the cycle however, further growth is limited by environement conditions, bio-chemical characteristics of the micro organisms and chemical characteristics of the sewage.

In as far as food supply is concerned, it has been found that organic and mineral components of sewage are sufficient to meet the requirements.

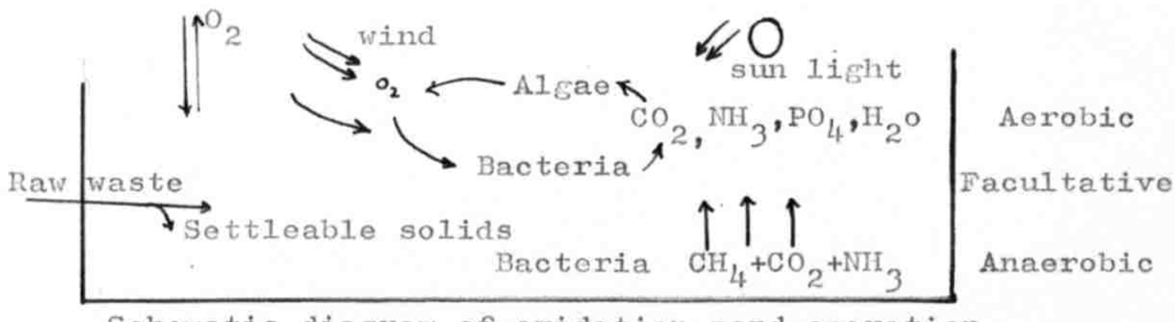
Oxygen is supplied in the form of dissolved oxygen and combined oxygen. The dissolved oxygen is supplied by atmosphere through surface aeration and mainly through algal photosynthetic oxygenation process.

The main micro organisms responsible for stabilization of organic matter are bacteria and algae. who live under symbiosis, although certain of the flagellated protozoa do give an assist. Symbiosis explains a relationship in between two different micro-organisms, here algae and bacteria, who do not compete with each other for food, but their activities are often inter dependent. A schematic diagram is shown as:(37)



Schematic diagram of bacteria-algae symbiosis

The presence of the type of the bacteria predominating in the lagoon determines the type of biological process taking place, that is to say aerobic, anaerobic or the combination of two so called facultative. It is, however, believed that the process can not be defined specifically by one type because surface layers which are rich in algael growth and aerated through surface aeration by wind, etc. give rise to aerobic bacteria hence the process in that zone is aerobic, whereas in the bottom layers where, settleable solids settle and also since the depletion of dissolved oxygen is there, such conditions give rise to anaerobic bacterial growth hence the process in that zone is anaerobic, however, in the central zone there is a third species of bacteria called facultative which thrive in either of the two conditions. Since the conditions in the central zone are a combination ofthe two, hence the process in this zone is facultative. Most of the stabilization lagoon designs are based on this assumption of conditions. A schematic diagram of oxidation or stabilization pond operation is given below:(37)



Schematic diagram of oxidation pond operation

Algae produce oxygen though photosynthesis in pressure of sun light which is utilized by bacteria which stabilizes the organic matter producing the end products as CO_2 and H_2o, the CO_2 produced is used by algae, hence the cycle goes on. Wind provides another source of oxygen by aerating the surface layers. The third factor of temperatures play an important role too, because microbial growth is enhanced at higher temperatures and vice versa. Thus sun light, wind, and temperature are the three main natural environmental factors which are uncontrolable. It is only when such uncontrolable factors act favourably that one is able to achieve the best environmental conditions and ultimately the best efficiency from the system.

10.5. Classification of Lagoons

In section 10.4 three major biological processes, depending upon the biological species predominating in a pond, were described. Based upon this and from the treatment stand-point three major classes of ponds could be defined as : anaerobic, facultative, and aerobic.[38]

Anaerobic ponds are defined as those in which the major fraction of the applied B.O.D. is decomposed through methane-fermentation. These are designed to establish conditions which encourage methane-fermentation.

The ponds are as much as 10 feet deep with smaller surface area and loaded in excess of 400 lbs per acre per day.[39] Effluents usually have B.O.D. in excess of 200 P.P.M. and hence require further treatment.[39] These ponds may be carefully located with respect to populated areas, as they may be extremely odourous if active sulphide reduction by photosynthetic bacteria does not occur.

Facultative ponds are described as: ponds where B.O.D. removal occurs as a result of both aerobic and anaerobic processes. Aerobic reactions occur in the top layers where as anaerobic reactions occur in the bottom sludge layers. Loadings are usually confined to 50 lbs per acre and effluents rarely have B.O.D.'s in excess of 30 P.P.M.[39]

In aerobic ponds organic matter is decomposed solely through the mechanism of aerobic oxidation.

The ponds are designed with large surface area to volume ratio. Stabilization is achieved through photosynthetic oxygenation. Large quantities of algae are grown and may be removed through separation as a valuable by product. Loadings usually employed are 100 to 200 lbs. per acre per day.[38] In either case B.O.D. levels of 20 to 30 P.P.M. are obtained.[38] If algae are removed by flocculation, an effluent B.O.D. of less than 10 PPM and amount of algae produced as 20 to 30 tons per acre per year, may be obtained.[39]

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For our purpose, we will be using aerobic ponds for the obvious reasons of high quality effluent, free from odours and other nuisances.

10.6. Basic design theories

Lot of work has been done lately by authors like Oswald et al, Hermann & Gloyna, and Marais, G.V.R., on the development of fundamental theories and criteria upon which pond design can be based.

Before discussing theories, the various standard design parameters will be described.

<u>10.6.1 Loading parameters.</u>

The 5-day, 20°C, B.O.D. of sewage is universally accepted as a unit of measuring sewage strength, usually expressed as milligram per litre, parts per million or pounds of B.O.D. Loading parametres are mostly based on the B.O.D. per capita contribution. The most common and most widely used is a surface loading parameter which is expressed in terms of 5-day, 20°C, B.O.D. per acre per day. Hermann and Gloyna recently proposed the use of a volumetric loading expressed as pounds of B.O.D. per day per acre foot.[40]

Areas with serious ice forming problem loadings as 20-40 lbs per acre per day, have been specified and allowed where as areas with ice free climatic conditions loadings from 100-250 lbs per acre per day have been specified and allowed.

10.6.2 Effluent standard parameters.

Quality of effluent or so called the degree of purification is dictated by the ultimate use of effluent. Effluent can be used for either of the following purposes.

a) Domestic use.

b) Irrigation purposes.

c) Liquid medium for fish breeding.

d) Recharging underground reservoirs.

e) Direct discharge into a body of water.

f) Direct discharge on land for disposal by seepage and evaporation.

The above classification of use describes also the degree of purity required in descending order.

The British Royal Commission standards specify a 5-day, 20°C, B.O.D. level not greater than 20 P.P.M. and a suspended solids level not greater than 30 P.P.M. as a standard for effluents that are to be discharged into a body of water.[34]

Different specified effluent standards can, however, be achieved with the use of stabilization lagoon. Series operation is beneficial where a high level of B.O.D. or coliform removal is important.[41]

10.6.3 Design theories.

Three basic theories will be discussed in the following pages, merits and demerits of each will also be discussed and the one most feasible for application will be evaluated.

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I- Oswald, Gotaas, Golueke, and Kellen.[42]

They based their theory on the fact that stabilization is mainly accomplished as a result of the photosynthetic oxygenation of organic wastes and is proportional to the sun light conversion efficiencies attained by algae. Various equations were formulated and following assumptions were made.

i) Algae is dispersed throughout the sewage medium.

ii) Proper nutrition for algael growth are contained in the sewage medium.

iii) Environmental factors of light and temperature are adequate.

iv) Algae will develop in such a medium provided sufficient time is allowed.

Based on the above assumption, a formula relating the light energy utilization by algae and the production of oxygen is given as :

$$W_o = 0.25\ F\ S \qquad 1 - a$$

where, W_o is the quantity of oxygen in lbs per acre per day.
F is the light conversion efficiency of the algae expressed in percentage and is a variable dependant upon environmental conditions.
S is the quantity of solar energy in calories per sq. cm. per day. The figure of 3.68 is taken as the energy required to produce 1 mg of O_2 to solve for W_o, an average value of 4 is taken to be a good approximation, however for S, tables are available which give values

of S with respect to latitude and time of year.[43]

A conclusion therefore is drawn that the oxygen produced is function of visible light energy and photosynthetic efficiency attained by the organisms.

A second equation relating depth-retention time factors to the total oxygen produced and termed as "loading factor" is described as:

$$\frac{d'}{D} = \frac{1000\ FS}{W_o \times 3.68} \qquad 1 - b$$

where, d' is the depth of pond in inches.

W_o is the oxygen produced in D days expressed in mg. per litre. Since, F, varies with depth and retention time, as such, an assumed value of F may lead to big error is estimating depth-retention period ratio hence two other methods for depth determination were put forth.

a) The method is a Beer-Lambert Law derived expression:

$$d' = \frac{\mathrm{Ln}\ I}{C_c} \qquad 1 - c$$

where, d' is the depth to which the visible light penetrates.

I is the incident light intensity.

C_c is the concentration of algae

is the absorption co-efficient of the algael cell taken 1.5×10^3 usually.

b) This method is applicable when depth, retention time and B.O.D. are known

$$W_{B.O.D.} = 0.22\ L_t\ \frac{d}{D} \qquad 1 - d$$

where, $W_{B.O.D.}$ is the theoretical amount of oxygen in lbs per acre per day required to stabilize the applied B.O.D. L_t in mg. per litre.

d = is the depth in inches

D is the retention time in days.

An "oxygenation factor" $F/_{Fc}$ is introduced where F_c is the theoretical efficiency required to oxidize the total B.O.D. load and F is the actual efficiency. F can be found by the following relationship

$$F = \frac{H}{S'} \qquad 1 - e$$

where, S' is the quantity of sun light expressed in calories per litre per day and obtained from:

$$S' = \frac{1000S}{d'} \qquad 1 - f$$

H is the calories of energy produced in the algael cell obtained from:

$$H = y_c\ h \qquad 1 - g$$

where, y_c is the yield of cell material expressed in mg. per litre per day, obtained from:

$$y_c = \frac{Cc}{D} \qquad 1 - h$$

h is the heat of combustion in calories per mg and usually taken as 6. Recommended retention periods vary between 3-5 and 5 days. Experimental results show that an average oxygenation factor of 1.6 keeps aerobic condition in a pond and a loading factor between 2 and 6 seems to give satisfactory results.

II- <u>Hermann and Gloyna.</u>[40]

They, in developing their work, considered that the factors of temperature, light, loading, size, number and shape of ponds together with the hydraulic appertenances as the controlling factors in the operation of the biological processes involved in the stabilization mechanism.

Their attempt was based on developing these factors into design criteria thus controlling the physical conditions and providing suitable environmental surroundings.

A relationship was formulated based the fact that the rates of most chemical reactions increase greatly as the temperature is raised. A frequently used very approximate rule, enunciated by Van't Hoff, is that the rate doubles for each rise in temperature of $10^{o}C$.[(40),(4)]

$$\frac{t}{t_o} = e^{c(T_o - T)} = \Theta\ (T_o - T) \qquad 2 - a$$

where, t is reaction time required at any temperature T

t_o is the original reaction time at an original temperature T_o.

e is the base of natural logarithm

C is the energy temp: constant.

An ideal curve was drawn using the equation and passing curve through $\frac{t}{t_o}$ = 1 and T = $35^{o}C$, the constant e^{c}, was taken as $e^{0.0693} = \Theta = 1.072$, valid only for temperature range of 3 to $35^{o}C$, being the effective temperatures for the algael-bacterial activity.

It was pointed out that in hot climates with exceptionally good operating controls curves could be drawn with lesser values of, C, the least value however should be 0.0555.

A second attempt based on experimental data using optical density measurements in accordance with the Beer-Lambert law was formulated as:

$$I = I_o \times 10^{-Kcd} \qquad 2 - b$$

or

$$d = \frac{\text{Log } I_{o}/I}{KC} \qquad 2 - c$$

where, d is depth in centimetres

K is the absorption co-efficient

C is the concentration of absorbing material in grams per litre.

From experimental results for practical solution, value of KC has been found as 0.056 cm^{-1} and I is replaced by I_c which is the light intensity of the compensation point at which photosynthesis is balanced by respiration and the value of which will be dependent on the algael species present.

A value of $I_c \leq 24$ ft.-c was given for chorella pyrenoidosa.

From these calculations rough results were obtained because of numerous assumptions involved and particularly because of conditions embodying the definite culture of algael species, a quite unusual occurence that could be expected in practice.

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A third step taken was in formulating B.O.D. loading based on governing influence of temperature rather than light. A volumetric B.O.D. loading parameter was introduced expressed in lbs. of B.O.D. per day per acre ft.

$$V = N_q \frac{Y}{200} t_o e^{c(T_o - T)} \qquad 2 - d$$

where, V is volume of pond required

N is number of people served

q is daily sewage flow per capita

Y is the influent B.O.D.

$t_o e^{c(T_o - T)}$ is the retention time from equation 2 - a

200 is the average B.O.D. of U.S. Sewage.

From the previously developed physical criteria a rational formula can be evaluated now by taking t_o = 3.5 day based on experimental value, T_o as 35^oc and C as .0693, as follows:

$$V = 5.37 \times 10^{-8}\ N\ q\ y \times 1.072^{(35-T)} \qquad 2 - e$$

where, V is pond volume in acre ft.

q is the per capita daily sewage flow in gallons.

y is B.O.D. in mg per litre.

T is operating temperature ^oc taken as average pond temperature during the coldest month at the geographic location of pond. An operating depth ranging between 2 to 3.5 ft. is recommended.

III- <u>Marais, G.V.R.</u>[44]

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By him a differential equation governing the B.O.D. and faecal bacteria concentrations in a pond is derived, based on the monomolecular law, of decrease in pollution concentration, and in terms of the volume of the pond, the influent and effluent flows, the B.O.D. and faecal bacterial concentrations and the monomolecular constant, K. Thus his theory, which was mainly inspired by the natural purification processes in rivers, considers only two parametres of pollution namely the B.O.D. and faecal bacterial pollution, and the inspiration has been given a practical shape of applicability in a series of pond system.

The theory is primarily concerned with the determination of the equilibrium concentrations of the B.O.D. and faecal bacteria in the pond under the daily cyclic pollution inflow and out-flow.

The theory does not concern itself with the biological agencies responsible for the degradation action but only in the results they produce, which gives rise to the value of K.

Following basic assumptions were made:

i) The reduction in concentration, S, takes place, according to the monomolecular law, i.e. the rate of change of concentration S, at any time, t, is proportional to the concentration at that time, expressed as:

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$$\frac{ds}{dt} = - KS \text{ or } ds = - KSdt \qquad 3 - a$$

where, K, the monomolecular constant expressed in Log 5-day units.

ii) Instantaneous and complete mixing of the influent and pond contents is achieved which makes the effluent and pond contents qualitatively similar.

iii) The effects of evaporation and percolation on the concentration and the effluent flow are negligible.

The differential equation derived equates the change in concentration, dS, to the sum of:

i) The increase in concentration due to the influent flow which is equal to:

$(Q_1/V)\ S_o\ dt.$

ii) The decrease in concentration due to the effluent flow which is equal to:

$- (Q_{2/V})\ S\ dt.$

iii) The decrease in concentration due to the stabilization agencies which is equal to: - KSdt.

$\therefore\ dS = (Q_{1/V})\ S_o dt - (Q_{2/V}) Sdt - KSdt$

$$\text{or } \frac{ds}{dt} + (K + Q_{2/V})S = (Q_{1/V})S_o \qquad 3 - b$$

where, Q_1 is influent flow at any time, t, in gallons or cu.ft. per day

Q_2 is the effluent flow at any time, t, in gallons or cu.ft. per day.

V is the volume of pond in gallons or cu.ft.

S_o is the concentration of pollution in the influent at any time, t.

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S is the concentration of pollution in the pond at any time, t.

t is the time in days and measured from any arbitrary instant

Based on a theorem which has been proved analytically, it has been found that values of terms used in the above derived differential equation could be taken as <u>mean</u> values hence the values used hence forth will be mean unless stated otherwise.

Further development is introduced as: "For a steady state to obtain in the pond, the mean daily values of Q_1, Q_2, and S_o must remain constant with respect to time. Thus equation 3 - b becomes:

$$S = \frac{S_o\left(\frac{Q_1}{V}\right)}{\left(K + \frac{Q_2}{V}\right)} = \frac{S_o}{\left(KR_1 + \frac{R_1}{R_2}\right)} \qquad 3 - c$$

where, R_1 is the influent retention time & is equal to $\left(\frac{V}{Q_1}\right)$

R_2 is the effluent retention time and is equal to $\left(\frac{V}{Q_2}\right)$

when retention time is short and R_1 is equal to R_2, equation becomes:

$$S = \left(\frac{S_o}{KR_1 + 1}\right) \qquad 3 - d$$

Conversely if the effluent flow is largely reduced by evaporation then R_2 becomes very large and hence, $\frac{R_1}{R_2}$ is negligible and equation becomes:

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$$S = \frac{S_o}{KR_1} = \frac{S_o}{KR} \qquad 3 - e$$

When long retention times are allowed it does not matter which equation is used, because KR_1 becomes very big as compared to unity. For further development in the theory, equation 3 - d was used to a series of ponds where the concentration of the effluent in one pond becomes the concentration of the influent in the following pond.

With the assumptions that K remains constant and R is the same for all the ponds, an equation is developed giving S in the n'th pond as:

$$S_n = \frac{S_o}{(KR+1)^n} \qquad 3 - f$$

Experimental verification showed that the theory applies for a series of ponds in the case where faecal bacterial concentrations are to be determined, while in the case of B.O.D. concentrations, the theory proved to apply only to the first pond and not to the secondary ones. This is attributed to the variation in the value of K. However, an equation by which B.O.D. concentrations can be determined for a series of ponds is given as:

$$P_m = \frac{P_{m-1}}{KR_m\left(\frac{P_m}{P_{m-1}}\right)^n + 1} = \frac{P_0}{\prod_1^m KR_m\left(\frac{P_m}{P_o}\right)^n + 1} \qquad 3 - g$$

where, P_m is the B.O.D. concentration at any pond m.

K is the initial degradation constant in log P-days units.

n is the efficient of non-uniformity.

But since no values of K and n have been devised, this relationship can not be used in design.

However Fig. 10.1. shows a chart in which a solution to the equation 3-g is given, and by which it is possible to determine the percentage pollution from a series of ponds for any values of K and n.(45)

Following equations can be used for determining pollution concentration in both cases:

a) For the concentration of faecal bacteria,

$$N = \frac{N_o}{(KR + 1)} \qquad 3 - h$$

where, N_o and N represent the number of bacteria per ml. in the influent and in the pond respectively.

A percentage reduction can be expressed as:

$$\left(\frac{N}{N_o}\right)\% = \frac{100}{(KR + 1)} \qquad 3 - i$$

A value of 2 for K can be used in above equations.

For ponds in series equation 3-i can be expressed as:

$$\left(\frac{N}{N_o}\right)\% = \frac{100}{(KR + 1)^n} \qquad 3 - J$$

b) For the B.O.D. concentrations:

$$P = \frac{P_o}{(KR + 1)} \qquad 3 - k$$

where, P_o and P represent the B.O.D. concentrations in the influent and in the pond respectively.

A percentage reduction in concentration is expressed as:

$$\left(\frac{P}{P_o}\right)\% = \frac{100}{(KR + 1)} \qquad 3 - L$$

A value of 0.17 for K can be used in the above equations. Experimental verification has shown that a large number of ponds in series, each with a

SOLUTION TO EQUATION

$$\frac{P_m}{P_o}\,\% = \frac{P_1}{P_o}\cdot\frac{P_2}{P_1}\cdots\frac{P_m}{P_{m-1}}\,\% = \frac{100}{\prod_{1}^{m}\left[K R_m\left(\frac{P_m}{P_o}\right)^n + 1\right]}$$

TANKS IN SERIES	K = 8	n = 0.8		
TANK (m)	1	2	3	
R_m	15	6	6	DAYS
P_m/P_{m-1}	6.9	30	45	%
P_m/P_o	6.9	2.07	0.93	%

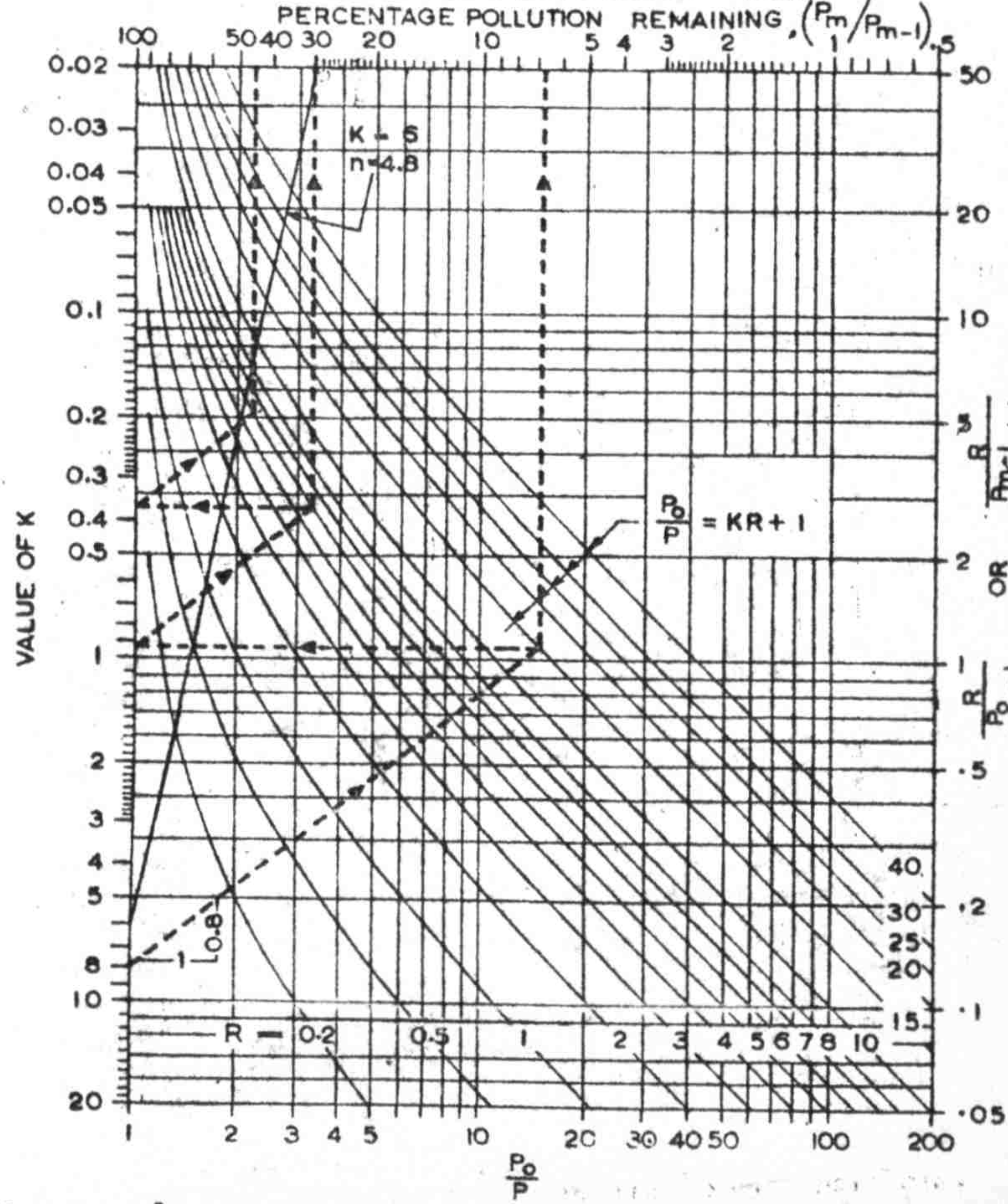

FIG 10.1 CHART FOR DETERMINING THE PERCENTAGE POLLUTION FROM A SERIES OF TANKS OR PONDS FOR ANY VALUE OF K & n

very short retention time, would be the most efficient way of stabilizing the influent pollution. Also the ponds should never be let to turn anaerobic to avoid odours etc.

At this stage, a criterion, based on the re-oxygenation rate which is a result of the photosynthetic activity of the algae and which in turn is dependent on radiation and depth of pond, was developed by which aerobic conditions can be maintained. The criterion for aerobic conditions can, therefore, be established as follows: Mean oxygen demand rate $\leq$ Mean re-oxygenation rate of the mean minimum radiation on a region is approximately constant, the mean oxygen production will be a function of depth alone.

$$\therefore P \leq f(d)$$

$$\text{or} \quad \frac{P_o}{(0.17R + 1)} \leq f(d) \qquad 3 - m$$

Verification of above criterion and various plots of B.O.D. concentrations in ponds against the total depth of ponds signify that a grouping of concentrations leading to aerobic and anaerobic conditions do exist. A rationalized equation for this separating line or in other words, which represent the critical value at which the concentration should be, to main aerobic conditions, was formulated from such experimental analysis and can be represented as:

$$P = \frac{P_o}{(KR + 1)} = \frac{1000}{(0.6d + 8)} \qquad 3 - n$$

where, d is total depth in feet and K is taken 0.17.

However, a lower value for the concentration is recommended. A figure of 75% of the limiting value has proved to be satisfactory.

∴ Therefore equation can be written as:

$$P = \frac{P_o}{(0.17R + 1)} \qquad \frac{750}{(0.6d + 8)} \qquad 3 - 0$$

Following may be taken as limitations of equation:

a) It is applicable to depths between 2 to 10 feet.

b) An area depth ratio of not less than 1000 is desired.

c) Minimum retention time for 1st pond should be 7 day and shorter for the following ponds.

Recommended design depths are to range between 3 to 5 feet.

10.7. Evaluation and Application

Untill very recently, no rational formula was presented for the design of lagoons. Design criteria were mainly based on actual results of existing pilot plants, and an intelligent guess on the part of the designer. Commendable work which has been lately done by authors like Oswald et al, Herman & Gloyna, and Marais, G.V.R. on the development of fundamental theories and criteria upon which pond design can be based, was described in last few pages.

In evaluating the various theories presented, a review of the basic principles upon which the theories were based and developed indicate that:

a) Oswald et al use the availability of sun light energy and the photosynthetic efficiency of algae in producing the required oxygen for stabilization, as the basic factors in their design theory.

b) Hermann and Gloyna assume that degradation follow the form e^{-ct} to which they have introduced the effects of temperature on the reaction time. The value of c'was found experimentally.

c) Marais, G.V.R. developed his theory by assuming that the degradation follows the monomolecular law, expressed in differential form as, $dS = - KS\,dt$, where K was determined experimentally.

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

However, various conclusions can be derived from the previous discussions that the most rational and practical approach was taken by Marais G.V.R.

In the case of Oswald et al, it might be true that photosynthesis plays an important role in the stabilization mechanism, yet it is well established that many other factors as well contribute considerably to the process. Thus to base the whole theory on these two factors which are variable and unpredictable in nature, seem to result an unefficient and unreliable method of design.

In the case of Hermann & Gloyna, the approach seems to be more rational in that the mechanism is considered as a reaction that follows a certain law, with the factor of temperature as decisive and responsible for the required reaction time.

However, there is one draw back of the theory which is, that the temperature taken in design so called as operating temperature is such that, it is the average of pond temperature during the coldest month at the geographic location of pond. Such temperature which is not the representative of the mean cyclic temperature and is dominant only for a very short period during which the possibility of the pond turning septic is remote, results into an over design.

Marais' approach seems to be the more practical in that he developed the actual ultimate results that have been achieved by various applications into design criteria. His method is a combination of theoretical and practical considerations. Assumptions are supported through experimental verifications. This by itself gives a more realistic approach as to the reaction of the various installations to the existing conditions. To take a different approach would mean the evaluation of the different factors affecting the biological agencies responsible, and which if completely considered, would render the development of any design criteria practically impossible.

Thus the theory set forth by Marais will be considered here in the actual design of lagoons, required for the treatment of sewage.

CHAPTER 11

DESIGN OF SEWAGE TREATMENT WORKS

Earlier in Chapter 10 various theories and design criteria set forth by Oswald et al, Hermann & Gloyna, and Marais G.V.R., were described and it was decided, after a critical review of all these, to adopt the theory and design criteria as given by Marais G.V.R., in actual design of sewage stabilization lagoons.

Thus, we will be using the criteria and formulas as given by Marais, which seems to apply best to our conditions.

In the following, therefore, complete a design of the units will be presented:

11.1. Design Procedure

The recommended procedure to be adopted in applying the Marais theory to practical uses in lagoon design is delineated below:(44)

a) An estimate of, the flow, the B.O.D. concentrations, and the faecal bacterial concentration, in the influent to the primary lagoon, should be made.

b) Select the depth of the pond in feet. Recommended range is between 3 to 5 feet.

c) Determine the maximum total B.O.D. concentration in the first lagoon consistent with aerobic conditions for the assumed depth from the formula:

$$P = \frac{750}{(0.6d + 8)}$$

where, P is the total B.O.D. concentration in mg/L.

d is the depth in feet.

d) Determine the retention time in the first lagoon from the formula:

$$R = \left(\frac{P_o}{P} - 1\right)\frac{1}{K} = \left(\frac{P_o}{P} - 1\right)\frac{1}{0.17}$$

where, R is the retention time in days

P_o is the influent B.O.D. concentration

P is the B.O.D. concentration in 1st lagoon consistent with aerobic conditions as found in (c)

K is the constant taken as 0.17 as described earlier.

e) Determine the surface area required for the assumed depth and required retention time from the formula

$$A = \frac{Q\,R}{7.48d}$$

where, A is the area of lagoon in sq.ft.

Q is the daily inflow in gallons.

R is the retention time in days.

d is the depth in feet.

f) Check that the area-depth ratio does not fall below 1000. If it does, increase the surface area or reduce the depth.

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g) Adjust the surface area of the lagoon to allow for the slopes of the banks, for the required retention time.

h) Select the depth of the second lagoon and determine the surface area, keeping a minimum of 7 days retention time.

i) Select the depth of the third lagoon and determine the surface area, keeping again a minimum of 7 days retention time and assuming no water losses in the first and second lagoons.

<u>Note</u>: 7 days limit for retention time applies to the cases where the retention time for 1st pond, as determined for the given flows, happens to be 7 days or so, and that secondary lagoons will be required for high quality effluents.

11.2. Design of Stabilization Lagoons

From the previous chapters following data is reproduced:

Ultimate domestic sewage flow = 4,185,000 gal.per day
Ultimate industrial waste flow= 1,041,000 gal. per day

Total flow = 5,226,000 g.p.d.

Ultimate B.O.D. contribution from domestic sewage
= 15,810 pounds/day.

Ultimate B.O.D. contribution from industrial waste
= 2904.7 pounds/day.

Total B.O.D. = 18714.7 pounds/day

$\therefore$ B.O.D. concentration $= \frac{18714.7 \times 10^6}{5226000 \times 8.34} = 430$ mg/L

Adopt a depth of 4 feet.

B.O.D. concentration in 1st lagoon is given by:

$$P = \frac{750}{(0.6d + 8)} = \frac{750}{.6 \times 4 + 8} = 72.0 \text{ mg/L}$$

Retention time is given by:

$$R = \left(\frac{P_o}{P} - 1\right) \frac{1}{0.17} = \left(\frac{430}{72} - 1\right) \frac{1}{.17} = \frac{4.95}{0.17} = 29 \text{ days}$$

Area required for lagoons:

$$A = \frac{Q\,R}{7.48d} = \frac{5226000 \times 29}{7.48 \times 4} = 5.0 \times 10^6 \text{ sq.ft.} = 115.0 \text{ acres.}$$

From the experiments conducted by Marais at Kafue in Northern Rhodesia, he deduced that"a series of ponds of retention time equivalent to that of a single pond will deliver a superior effluent to that from a single pond."(44)

Ponds are also required to be constructed in parallel for flexibility in operation.

Thus whole area as required above will be splitted into a system of series and parallel pond units.

Maximum desirable limit for the size of each cell as given by "Parker" is 10 acres.(46) Therefore, adopt 12 cells arranged in two parallel rows of six cells each. The size of 1st five cells in each row will be 10 acres each and the last two cells as 7.5 acres each.

The sum total of all the cells will give the required area of 115 acres.

The above requirements are of course ultimate, however, for the initial periods only two cells in each row comprising of 10 acres each will be sufficient to construct. This will give an area requirement of 40 acres. However, total land required will have to be acquired by the municipal committee.

The present flow will thus have a retention time for this area as:

$$R = \frac{43560 \times 40 \times 7.48 \times 4}{1063435} = 49 \text{ days.}$$

This retention time is much more than the required and will be beneficial towards the production of better quality effluents. Design charts have been prepared, by Marais,[43] which give the minimum required retention time for a given B.O.D. load and selected pond depth. This is shown in Fig. 11.1.

a) Dimensions of the cells.

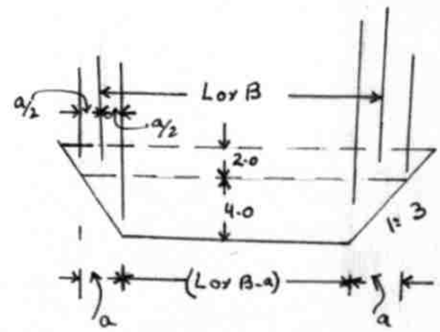
i) 10-acre cell:

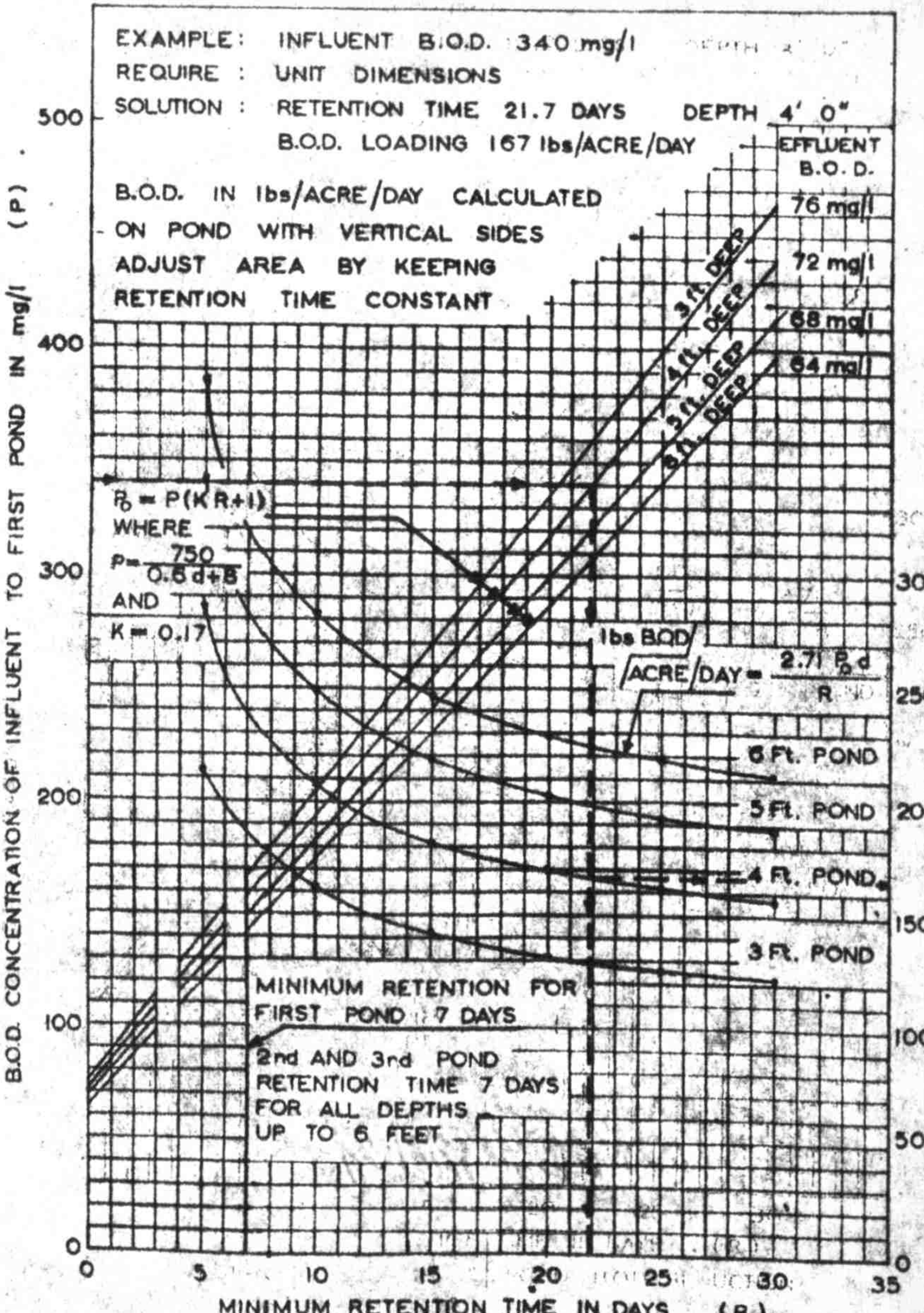
Area of cell = 10 x 43560 = 435600 sq.ft.

Taking length of cell as 900.0 feet

Width of channel will be $\frac{435600}{900}$ = 484.0 feet.

These are the dimensions for vertical sides. However, inner side slopes are to be provided. Usually slopes of 3 or 4 horizontal to 1 vertical are provided.[40]





DESIGN CHART FOR 1ST. POND B.O.D. REDUCTION

Adopting a side slope of 1:3

Horizontal length at the bottom of cell will be:

900 - 12 = 888.0 feet

Horizontal breadth at the bottom =

484.0 - 12 = 472.0 feet

∴ Horizontal length at water surface

= 888 + 24 = 912.0

Horizontal breadth at the water surface

= 472 + 24 = 496.0

$$\text{Area of cell} = \frac{888 + 912}{2} \times \frac{472 + 496}{2}$$

$$= \frac{1800}{2} \times \frac{968}{2} = 900 \times 484 = 435{,}600 \text{ sq.ft.}$$

These dimensions of the cell therefore, give, for the same depth, same retention time in the cell as desired.

A free board of 2 feet will also be provided and the banks will be continued with same slope till the top.

Horizontal length at top of the cell

= 888 + 36 = 924.0 feet.

Horizontal breadth at top of the cell

= 472 + 36 = 508.0 feet.

Area/depth ratio of the lagoon

$= \frac{435600}{4} = 108{,}900$ which is more than required minimum of 1000.

ii) 7.5 Acre Lagoon:

Area of lagoon = 7.5 x 43560 = 327,000 sq.ft.

Taking length of cell = 900.0 feet.

Width of cell will be = $\frac{327000}{9}$ = 364.0 feet.

With same side slopes and free board

∴ Horizontal length at the bottom = 900-12 = 888.0 ft.

Horizontal breadth at the bottom= 364-12 = 352.0 ft.

Horizontal length at the water surface

= 888 + 24 = 912.0 feet

Horizontal breadth at the water surface

= 352 + 24 = 376.0 feet

Horizontal length at the top = 888.0 + 36 = 924.0 ft.

Horizontal breadth at the top= 352 + 36 = 388.0 ft.

Area depth ratio = $\frac{327000}{4}$ = 81750

which is greater than 1000

b) Bottom of the Cell.

The Lagoon bottom should be made as level as possible with a maximum variation of 6 inches. The bottom should be well compacted and relatively water tight to avoid excessive losses through seepage.

Removal of porous top soil and the compaction of sub-soil will improve the water holding characteristics of the bottom.

The laggon area should be cleared of excessive vegetation and debris.

Conventional earth moving equipments should be used for the above purposes.

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c) Embankments.

Embankments should be seeded along the outer slope, the top, and along the inner slope to the normal water line. This minimizes erosion, facilitates weed control, and permits the maintenance necessary for good appearance.

Banks should be wide enough to permit access of maintenance machinery and inspecting vehicles.

A width of 10 feet, therefore, will be adopted. For percolation controls, banks should be properly compacted, and if possible, seepage controlling materials should be used.

d) Fencing.

The lagoon area should be enclosed with a suitable fence to preclude entrance of live stock and to discourage trespassing. Fences consisting of 3 or 4 stands of tightly stretched barbed wire will be used for the purposes.

e) Distribution Box.

Sewage from the sump well, which gets its flow from the city trunk line, will be pumped to a distribution box, which will be located amidst the distance between the two parallel rows of the lagoons and at a distance of about 5 feet from the starting lagoons. The height of liquid maintained in the distribution box will be such that sewage flows under gravity to the lagoons.

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f) Influent lines.

The influent line to the primary units should discharge far enough from the bank to ensure minimum interference with normal circulation. For small lagoons the discharge should be at the approximate center of the lagoon. For medium-sized installations, it has been found not to be so advantageous to locate inlets at more than 200 feet from the nearest bank.(41) A distance of 400 feet is found desirable for lagoons of 40 acres or more in area.(41)

For our purpose, we will be using 24" in.dia. C.I. pipe, which will discharge horizontally under gravity head as supplied by the distribution box. The line will extend for a distance of 200 feet from the nearest bank. Concrete aprons will be provided at discharge points to avoid erosion.

Inlets to the secondary units will be provided through inter-connecting pipes which will be also of 24 in. dia. C.I. pipe, about three of these pipes will be provided to afford proper mixing of influent and pond contents. Valves or other arrangement to regulate flow between structures will be provided.

The in-let will be constructed near the dike.

Flexible control depth in each unit is desirable. Inter-connecting piping for multiple installations is to be such as to facilitate maximum liquid retention in the system.

g) <u>Outlet structures:</u>

Outlet pipes are usually located 1.5 feet from the bottom. They are designed to permit lowering of water level at a rate of one foot per week while the facility is receiving its normal loads.(41)

A provision for the complete draining of lagoon is to be kept. Sewage from the outlet will be taken to a channel designed to feed, through tributories, to the nearby lands.

h) <u>Overflow structures.</u>

These are designed to permit operation of the lagoon at selected depths. Such flexibility can be achieved through valved piping or other adjustable overflow devices.

Levels of withdrawal are given varying between 6" and 12" below the water surface.(34) Weir manhole structure seems to be a satisfactory device for the purpose.

Complete details of lagoon constructional features are shown in Drawing No. 9.

11.3. Design of pumping units

Pumps will be required to lift sewage from the sump well to the distribution box.

<u>Discharge:</u>

Ultimate average inflow = 5,226,000 g.p.d. = 3600 g.p.d.

Present average inflow = 1,063,435 g.p.d. = 740 g.p.m.

Sump well:

Taking 5-minutes capacity of well.

Volume required = $\frac{3600 \times 5}{7.48}$ = 2400.0 cft.

Taking 5 feet as working depth, measuring 1 foot below pipe invert.

Area required = $\frac{2400}{5}$ = 480 sq.ft.

$\therefore$ diameter d = $\sqrt{\frac{480}{.78}}$ = 24.5 feet.

$\therefore$ Bottom elevation of sump = 143.0 ft.

Pumping Head:

i) Level required for gravity flow in inlet pipe will be as under:

Size adopted = 24 in. dia.

Discharge per pipe = $\frac{3600}{2} \times 2.5$ = 4500 g.p.m.

Length of line = 250 feet.

Equivalent length[14] for gate valve = 15 feet.

Equivalent length[14] for 90° elbow = 30 feet.

Equivalent length for exist[14] = 68 feet

Equivalent length for entrance[14] = 35 feet.

$\therefore$ Total length = 250 + 15.0 + 30.0 + 68 + 35.0 = 398 ft.

From nomogram[7] loss of head for 24 in. dia. pipe

= 2.0 feet per 1000 feet.

$\therefore$ Loss of head = $\frac{2 \times 398}{1000}$ = 0.8 feet.

Ground elevation at lagoon site = 159.0

Keeping lagoon base 1 foot below ground level

Water elevation in lagoon = 162.0 feet.

∴ Water elevation required in distribution box

= 162.0 + 0.8 = 162.80

If pumps are installed 5 feet below ground level, with a foot valve 2.0 feet above bottom.

ii) Suction lift = 10 feet.

iii) Delivery head = 162.80 - 155.0 = 7.80 feet

iv) Frictional losses in pump etc. = 5.0 feet.

v) Frictional losses in delivery line:

Length of line = 200.0 feet.

Equivalent length(14) for exist = 68.0 feet.

Equivalent length(14) for 90^{o} elbow = 30. feet.

Equivalent length(14) for Tee = 30 feet.

∴ Total length = 200.0 + 68.0 + 30.0 + 30.0 = 328.0 feet.

From nomogram(7) loss of head is equal to 8.0 feet per 1000 feet.

∴ head loss = $\frac{8 \times 328}{1000}$ = 2.62 feet.

∴ Total dynamic head = 10.0 + 7.80 + 5.0 + 2.62

= 25.42 say 26.0 feet.

Max. present inflow = 740 x 2.5 = 18500 g.p.m.

Thus for the present three pumps one with a capacity of 750 g.p.m. may be installed, such that during low flows, one pump will suffice and during peak flows, two pumps when put in operation in parallel will be able to meet the demands.

The third pump in either case will work as a standbye with the increase in demand a pump of higher capacity say 3000 g.p.m. may be installed such that all the three pumps when working inparallel will be able to meet the requirements and the new one will work as a standbye. Hereafter, further replacement should be made of low capacity pumps with higher i.e. 3000 g.p.m. pumps to meet the ultimate demands.

The head required for the pumps as calculated above is that required for ultimate flows, however, the actual heads required for initial flows may be found from the capacity head curve plotted for the delivery line. Thus the actual selection for pumps is made on the basis of such curves and performance curves of the pumps selected and as supplied by the local manufacturer.

A procedure was outlined in Chapter 5, as a guide towards such selection, the same may be referred for the purpose.

Ultimate peak flow = 3600 x 2.5 = 9,000 g.p.m.

Thus three pumps of 3000 g.p.m. and fourth as a standbye may be selected.

In all three pumps capable of giving a discharge of 750 g.p.m. and four pumps capable of giving a discharge of 3000 g.p.m. will be required by the end of year 2007.

H.P. for lower capacity pumps, assuming 60% efficiency

$$= \frac{750 \times 26 \times 100}{3960 \times 60} = 8.25 \text{ say 10 H.P. pumps.}$$

H.P. for higher capacity pumps

$$= \frac{3000 \times 26 \times 100}{3960 \times 60} = 33 \text{ say } 35.0 \text{ H.P.}$$

Centrifugal pumps, non-clog type, factory built, as supplied by the "Smith & Loveless"(31), or approved equal company will be installed for the obvious reasons, as described in Chapter 8, Screens and grit chambers will not be provided, screens are built in the pump unit itself and thus will be no pmp clogging and other problems. Complete details and instructions as given by the manufacturers are to be strictly followed.

CHAPTER 12

COST ESTIMATE

An approximate cost estimate of the projects is essential. This will help the authorities to allocate and procure the necessary funds required, and also to invite for tenders.

The estimate of quantities and the cost of the different items of the two projects is therefore, given below:

In the following estimates, the portions of the scheme, e.g. water distribution system and elevated reservoirs, which are already completed or are in the process of completion, are excluded.

12.1. First Phase

12.1.1 Tube wells installation.

Item No.	Description	Estimated quantities	Unit Rate Pak:Rupee	Total cost in Pak:Ruppee.
1.	Boring for tube wells upto 100 feet depth i/c supplying & installing 12 in.dia. 40 feet strainer & 55 feet blank casing, with drawing of boring casing pipe, sh-			

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	rounding around the strainer and cement grouting around the blank casing, and development of tube well complete as specified and directed by Engineer in charge	4 Nos.	each	10,000/-	40,000/-
2.	Supplying and installing centrifugal pumps, with necessary suction and delivery pipes and specials, i/c cost of electric motors of 10 H.P.	6 Nos.	each	7,500/-	45,000/-
3.	Construction of brick masonary pump house of required dimensions as per specifications & instructions of engineer in Charge etc. complete	2 Nos.	each	4,500/-	9,000/-
4.	Providing & laying of 12 in. dia C.I. manifold i/c the cost of fittings etc. complete	1475	Rft	30/-	44,250/-
5.	Miscellaneous	-	Lump sum	12,000/-	12,000/-
					150,250/-

Say Rs.150,000/-

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12.1.2 Water Treatment Plant.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Ruppee.
1.	Constructing of inlet structure of required dimensions i/c supplying and laying of 2-6 in.dia. C.I.pipes as inlets from inlet channel to the storage reservoir etc. complete	1 No.	each	6,000/-	6000/-
2.	Construction of 400'x 250'x 15' R.C.C. structured sedimentation storage reservoir, i/c construction of partition wells for compartments, flow controlling devices etc. complete	1 No.	each	75,000/-	75000/-
3.	Construction of rapid gravity filter 12.5'x 10.0' surface area i/c cost of manifold, laterals, filter sand, all accessories required for back washing.	2 Nos.	each	45,000/-	90,000/-

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Ruppee	Total cost in Pak:Rupees
4.	Construction of over-head reservoir of R.C.C. structure 30 feet high & 20,000 gallon capacity	1 No.	each	45,000/-	45,000/-
5.	Construction of clear water underground well of R.C.C. structure, having dimensions of 75'x 62.5'x 12' with partition wall etc. complete	1 No.	each	50,000/-	50,000/-
6.	Providing and installing chlorinating plant with all necessary equipment i/c the construction of room etc. complete	1 No.	each	15,000/-	15,000/-
7.	Construction of Laboratory with all necessary equipments	1 No.	lump sum	10,000/-	10,000/-
8.	Construction of store	1 No.	lump sum	10,000/-	10,000/-
9.	Cost of land (as acquired by M.C?)	12 acres	per acre	1,500/-	18,000/-

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
10.	Wire fencing all round the water works i/c the construction of gates etc. complete	-	Lump sum	7,000/-	7,000/-
11.	Development of water works site i/c construction of roads etc.	-	Lump sum	15,000/-	15,000/-
12.	Miscellaneous requirements & general services	-	Lump sum	25,000/-	25,000/-
			Total	Rs.	366,000

12.1.3 Sewerage Network.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.(a)	Providing & laying 8 in.dia. concrete pipe sewer at invert depth upto 6 feet	30,000	per foot	15/-	450,000/-
(b)	Providing & laying 8 in.dia. concrete pipe sewer at invert depth above 6 feet	3,000	per foot	18/-	54,000/-

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee
2.	Providing & laying 10 in.dia. concrete pipe sewer at invert depth above 6 feet.	300	per foot	20/-	6,000/-
3.	Providing & laying 12 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	1,400	per foot	25/-	35,000/-
4.(a)	Providing & laying 15 in.dia. R.C.C. pipe sewer at invert depth upto 6 feet	900	per foot	28/-	25,200/-
(b)	Providing & laying 15 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	1,350	per foot	30/-	40,500/-
5.(a)	Providing & laying 18 in.dia.R.C.C. pipe sewer at invert depth upto 6 feet	900	per foot	32/-	2,880/-
(b)	Providing & laying 18 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	3650	per foot	35/-	127,750/-

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
6.	Providing & laying 21 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	1650	per foot	38/-	62,700/-
7.(a)	Providing & laying 24 in.dia. R.C.C. pipe sewer at invert depth upto 6 feet	1000	per foot	40/-	40,000/-
(b)	Providing & laying 24 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	600	per foot	42/-	25,200/-
8.	Providing & laying 27 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	3400	per foot	45/-	153,000/-
9.	Providing & laying 30 in.dia. R.C.C. pipe sewer at invert depth above 6 feet	3000	per foot	48/-	144,000/-
10(a)	Constructing manholes over 8 in.dia. pipe sewers upto 6 feet depth	1200	each	300/-	360,000/-
(b)	Constructing manholes over 8 in.dia. pipe sewers above 6 feet depth	100	each	400/-	40,000/-

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
11.	Constructing manholes over 10 in.dia. pipe sewer above 6 feet dep.	1	Each	450/-	450/-
12.	Constructing manholes over 12 in.dia. pipe sewers above 6 feet depth	5	Each	500/-	2,500/-
13(a)	Constructing manholes over 15 in.dia. pipe sewers upto 6 feet depth	3	Each	550/-	1,650/-
(b)	Constructing manholes over 15 in.dia. pipe sewers above 6 feet depth	5	Each	650/-	3,250/-
14(a)	Constructing manholes over 18 in.dia. pipe sewers upto 6 feet depth	3	Each	700/-	2,100/-
(b)	Constructing manholes over 18 in.dia. pipe sewers above 6 feet depth	14	Each	750/-	10,500/-
15.	Constructing manholes over 21 in.dia. pipe sewers at depths above 6 feet	7	Each	800/-	5,600/-

16(a)	Constructing manholes over 24 in.dia. pipe sewers at depths upto 6 feet	4	Each	850/-	3,400/-
(b)	Constructing manholes over 24 in.dia. pipe sewers at depths above 6 feet	2	each	900/-	1,800/-
17.	Constructing manholes over 27 in.dia. pipe sewers at depths above 6 feet	16	each	1000/-	16,000/-
18.	Constructing manholes over 30 in.dia. pipe sewers at depths above 6 feet	11	each	1050/-	11,550/-
19.	Miscellaneous & general services	-	lump sum	60000/-	60,000/-
			Total		1,685,030/-
			Say	Rs.	1,685,000/-

12.1.4 Sewage Treatment Plant.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.	Construction of sewage sump well as specified	1 No.	each	5,000/-	5,000/-

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
2.	Construction of dry well for pumps, control room as specified	1 No.	Each	6000/-	6000/-
3.	Providing & installing sewage lift pumps from sump well to distribution box, complete factory built type, i/c cost of electric motors etc.	3	Each	8000/-	24000/-
4.	Construction of distribution box as specified	1 No.	each	750/-	750/-
5.	Constructions of lagoon i/c embankment, removal of top loose soil, compacting the bottom etc. complete	4	each	5000/-	20000/-
6.	Providing & laying 24 in.dia. C.I. inlet pipe with necessary fittings	500	foot	60/-	30000/-
7.	Providing & laying inter-connecting pipes 24 in.dia. C.I., with necessary control devices	500	foot	60/-	30000/-

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8.	Construction of outlet manholes as specified	4 Nos.	each	1000/-	4000/-
9.	Construction of outlet dhannel	lump sum		5000/-	5000/-
10.	Cost of land	120	Acre	500/-	60000/-
11.	Fencing around treatment plant i/c construction of gates etc.	-	lump sum	20000/-	20000/-
12.	Development of land i/c construction of internal roads etc.	-	lump sum	15000/-	15000/-
13.	Miscellaneous & general services	-	lump sum	5000/-	5000/-
		Total			224750/-
				Say Rs.	225000/-

12.2. Second Phase

12.2.1 Tube well installation.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.(a)	Construction of wet well, dry well and control room for sub-trunk line lifting station as specified	1	Each	10,000/-	10,000/-
(b)	Construction of wet well, dry well and control room for Trunk				

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.	Boring for tube wells upto 100 feet depth i/c supplying & installing 12 in.dia. 40 feet strainer & 55 feet blank casing, with drawing of boring casing pipe, shrouding around the strainer and cement grouting around the blank casing, and development of tube well complete as specified & directed by engineer in charge	10	Nos.each	10,000/-	100,000/-
2.	Supplying and installing centrifugal pumps, with necessary suction & delivery pipes & fittings, i/c the cost of 10 H.P. electric motors	15	Nos.each	7,500/-	112,500/-
3.	Construction of brick masonary pump houses of required dimensions as per specifications & in-				

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak;Rupee.
	structions of the engineer in charge	5 Nos.	each	4,500/-	22,500/-
4.	Providing & laying of 12 in.dia. C.I. manifold i/c the cost of fittings etc. complete	1475	Rft	30/-	44,250/-
5.	Miscellaneous	-	lump sum	10,000/-	10,000/-
			Total		289,250/-

12.2.2 Water Treatment Plant.

1.	Providing & installing chlorinating plant with all necessary equipment i/c the construction of necessary room etc. complete	1 No.	each	15,000/-	15,000/-
			Total		15,000/-

12.2.3 Sewage Treatment Plant.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.	Providing & installing sewage lift pumps from sump well to distribution				

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Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
	box i/c the cost of electric motor of capacity 3000 g.p.m.	4 Nos.	each	12,500/-	50,000/-
2.	Construction of lagoon i/c embankments, removal of top loose soil, compacting the bottom etc. complete.				
	a) 10 acre lagoons	6 Nos.	each	5,000/-	30,000/-
	b) 7.5 acre lagoons	2 Nos.	each	4,000/-	8,000/-
3.	Providing & laying inter-connecting pipes 24 in.dia. C.I., with necessary control devices etc.	1500	foot	60/-	90,000/-
4.	Construction of outlet manholes as specified.	4 Nos.	each	1000/-	4,000/-
5.	Miscelleaneous	-	Lump sum	-	6,000/-
				Total	188,000/-

12.2.4 Lift Stations.

Item No.	Description	Estimated quantities	Unit	Rate Pak:Rupee	Total cost in Pak:Rupee.
1.	Providing & Installing sewage lift pumps complete factory built type coupled with electric motors as specified of capacity i/c the cost of pipes, fittings etc. complete				
	a) 600 g.p.m.	2 Nos.	each	8,500/-	17,000/-
	b) 3000 g.p.m.	2 Nos.	each	12,500/-	25,000/-
2.	Miscellaneous	-	Lump sum	2,000/-	6,000/-
				Total	48,000/-

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12.3. Abstract of Costs

a) Water Supply Scheme

Tube well installation 1st stage	Pak.Rs.	150,000/-
Water treatment plant 1st stage	Pak.Rs.	366,000/-
Tube well installation 2nd stage	Pak.Rs.	289,250/-
Water treatment plant 2nd stage	Pak.Rs.	15,000/-
High service reservoir (as taken from feasibility report(1))	Pak.Rs.	200,000/-
Distribution system (as taken from feasibility report(1))	Pak.Rs.	300,000/-
	Total	Rs.1,320,250/-

∴ Cost per capita = Rs. 14.25 = approx. $ 3.5 per capita.

b) Sewerage Scheme

Sewerage net work 1st phase	Pak.Rs.	1,685,000/-
Sewage treatment plant 1st Phase	Pak.Rs.	225,000/-
Sewage treatment plant 2nd Phase	Pak.Rs.	188,000/-
Lift stations 1st phase	Pak.Rs.	80,000/-
Lift stations 2nd Phase	Pak.Rs.	48,000/-
Total	Pak.Rs.	2,226,000/-

Per capita cost = Rs.23.95 = approx. $ 5.5 per capita.

SUMMARY

The purpose of this report is to carry out a design of a water supply and sewerage system for Khairpur Mir's W. Pakistan.

This city has an existing water supply system, which being inadequate, had to be supplemented. The existing system only serves one zone out of three, which have to resort to individual shallow insanitary hand pumps and tube wells. The progressive standards of living of the community in the area and subsequent increase in water demand has, therefore, called for an improvement in the existing system.

To this effect the Government of W. Pakistan has already approved such a scheme. Of the various component items of the scheme the distribution system is completed and an additional reservoir of 100,000 gallons capacity is bein constructed. Source selection is the problem yet to be solved.

This project is designed for the coming 40 years ending in the year 2007. The present and future population is estimated to be 44,500 and 93,000 respectively. The design is based on the ultimate future population. The complete scheme is scheduled to be completed in two phases of 20 years each.

In this study, the problem of source selection

has been fully reviewed and finally it has been decided, for the 1st phase of the scheme, to let the two available sources viz. surface water from Mirwah Canal and ground water collected through tube wells, share the total requirements. An approximate sharing of about 40% by the Canal water and the rest by the tube well water is recommended.

This sharing etc. has been suggested because of the reasons that good and sweet quality of tube well supply is dependent on the seepage water from the canal, on the bank of which these tube wells are proposed to be installed. This canal has a maximum closure period of 15 days in certain months. As such to base completely on tube wells, there is every possibility of contamination of the supply with distant brackish water specially during the maximum canal period. Also to base completely on Canal source, would require very large storage etc. which could neither be afforded nor is desired.

For the second phase, it has been recommended to base the entire supply on tube wells. This has been done with a view that complete observations of ground water behaviour particularly during canal closure period will be taken and also an improvement in the ground water quality is also expected due to a plan by Government under "Water logging and Salinity Control" scheme, which is scheduled to be completed in coming 20 years. The final decision, however, depends on the favourable and supporting results of the observations.

This city, in fact, has no proper sewerage system. Domestic waste is collected through open drains in some areas and individual cesspits located infront of each door in the rest of the areas, which results in a bad asethetic manifestation and insanitary environement. The drainage water from different sectors is collected in extra mural open drains and finally disposed in canals or agricultural fields, which ever feasible.

In this project a complete underground sewerage system and treatment of sewage has been proposed. The treatment of sewage will be effected through stabilization lagoons, the only process which is found suitable for the local conditions. The effluent produced will have 5-day B.O.D. value of around 20 P.P.M. In all 10-20 acre lagoons and 2-7.5 acres will be required by the end of year 2007. Only 4-10 acre units will be constructed for the present.

Concrete pipes for sewers and cast iron pipes for pressure sewers will be used. In order to avoid deep excavation and proximity to ground water table lift stations have been provided to lift the sewage to a pre-determined level and run, thereafter, under gravity. Two such lift stations have been provided. One station will be located on the sub-trunk line serving Zone B, and the other near the road bridge by the side of the canal. This will lift the combined sewage from the two zones viz A and B. A third lift station will also be required at the treatment plant site to lift the sewage from a sump well and feed it to the lagoons through a distribution box.

The total cost ofthe scheme for the water

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supply system comes out to be Rs. 1,320,250/- which is approximately equivalent to $ 3.5 per capita.

The total cost of scheme for sewerage system comes out to be Rs. 2,226,000/-, which is approximately equivalent to $ 5.5 per capita.

Thus the over all cost for the two schemes will be approximately equal to $ 9 per capita.

Of the total cost of the scheme 1/3rd of the capital cost will be paid by the Government as an aid to the municipality and 1/3 rd of the capital cost as a loan repayable in 20 years and the remaining 1/3rd will be paid by the local municipality. It is believed that this project will be implemented as soon as funds become available because of its greater need and urgency.

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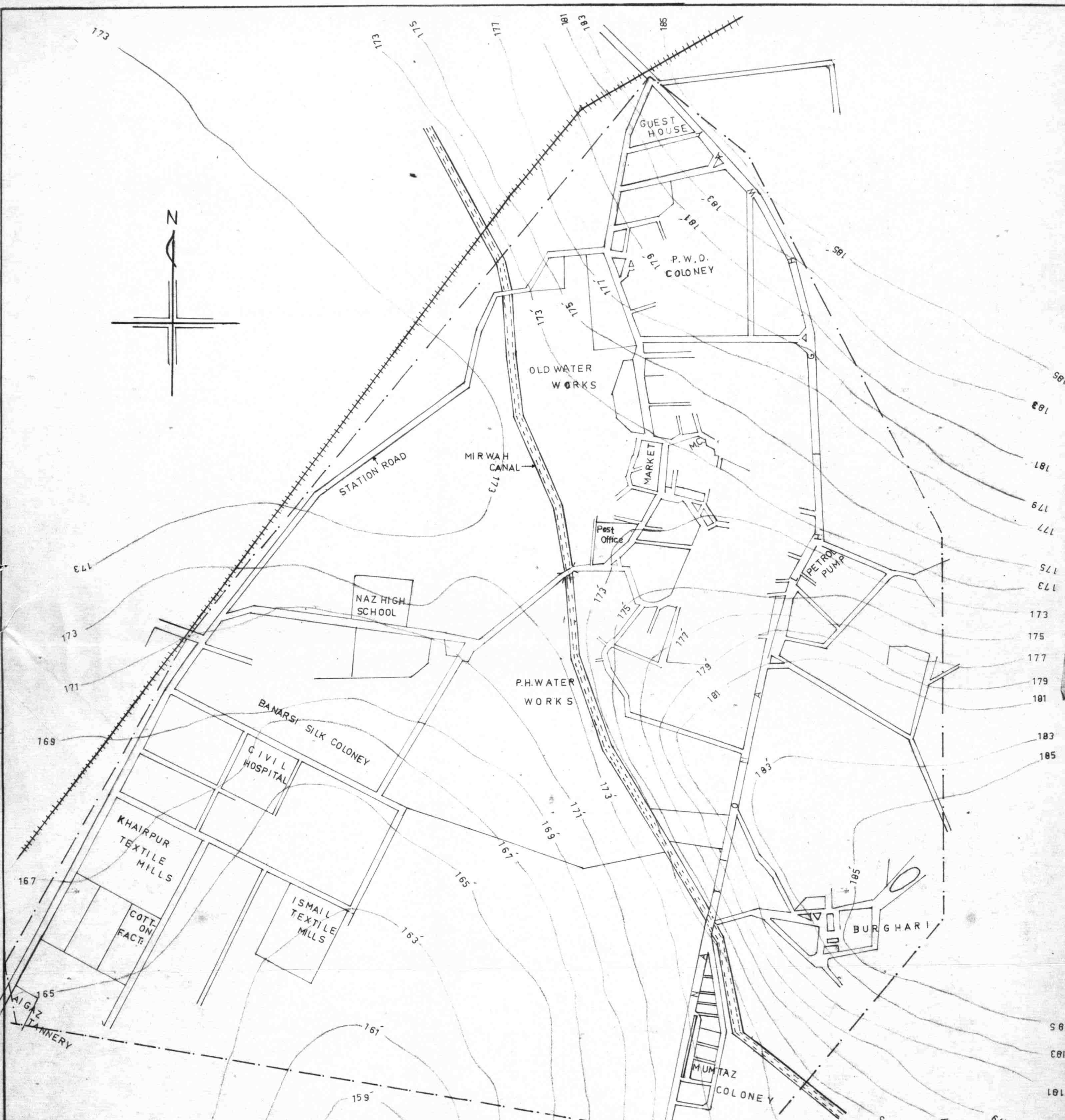
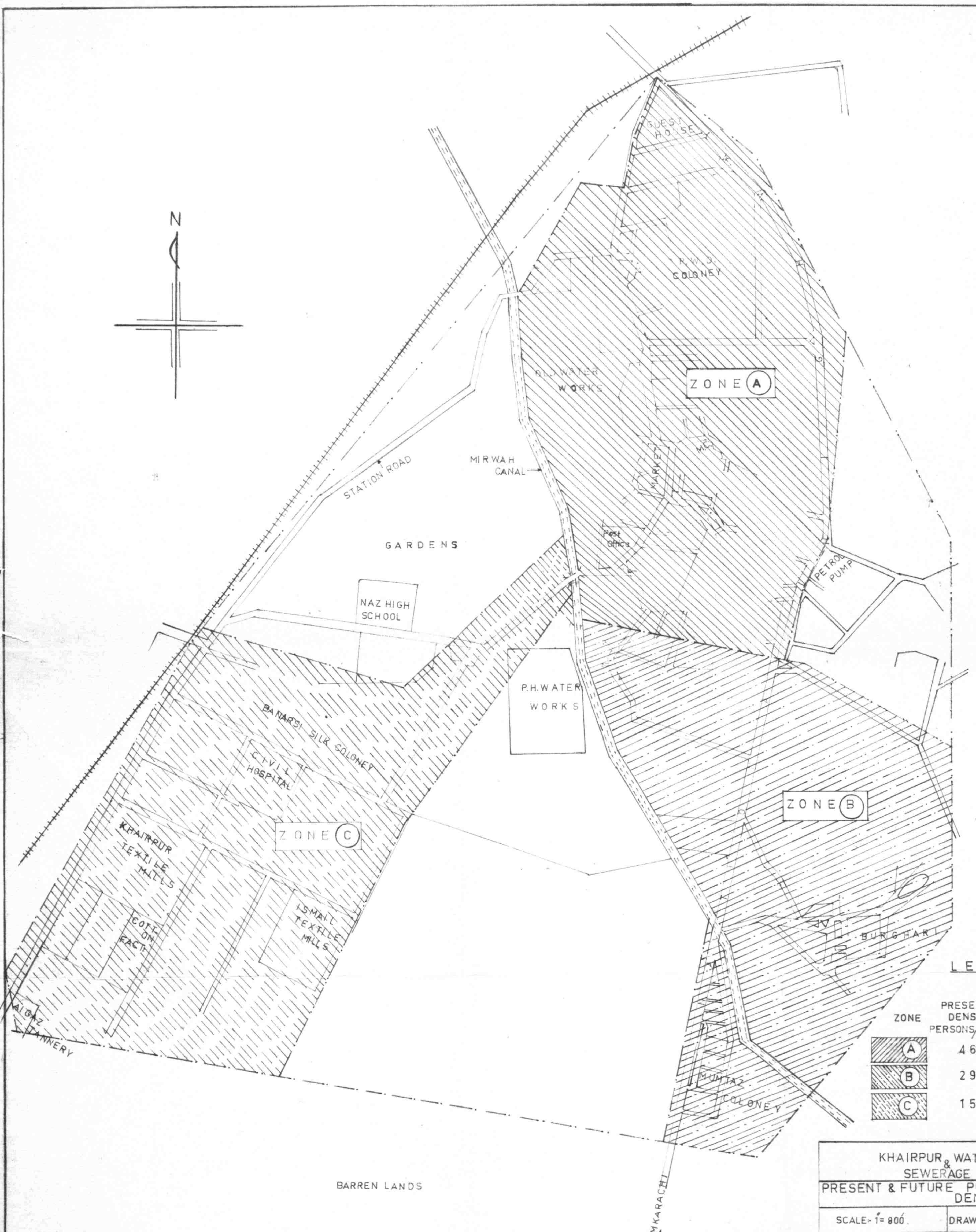


KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME	
TOPOGRAPHY	
SCALE- 1"= 800'.	DRAWING NO. 1
ENGINEER SHAIKH ALLAH BUX	DATE MAY 1967



LEGEND

ZONE	PRESENT POP. DENSITY PERSONS/ACRE	FUTURE POP. DENSITY PERSONS/ACRE
A	46	72
B	29	75
C	15	45

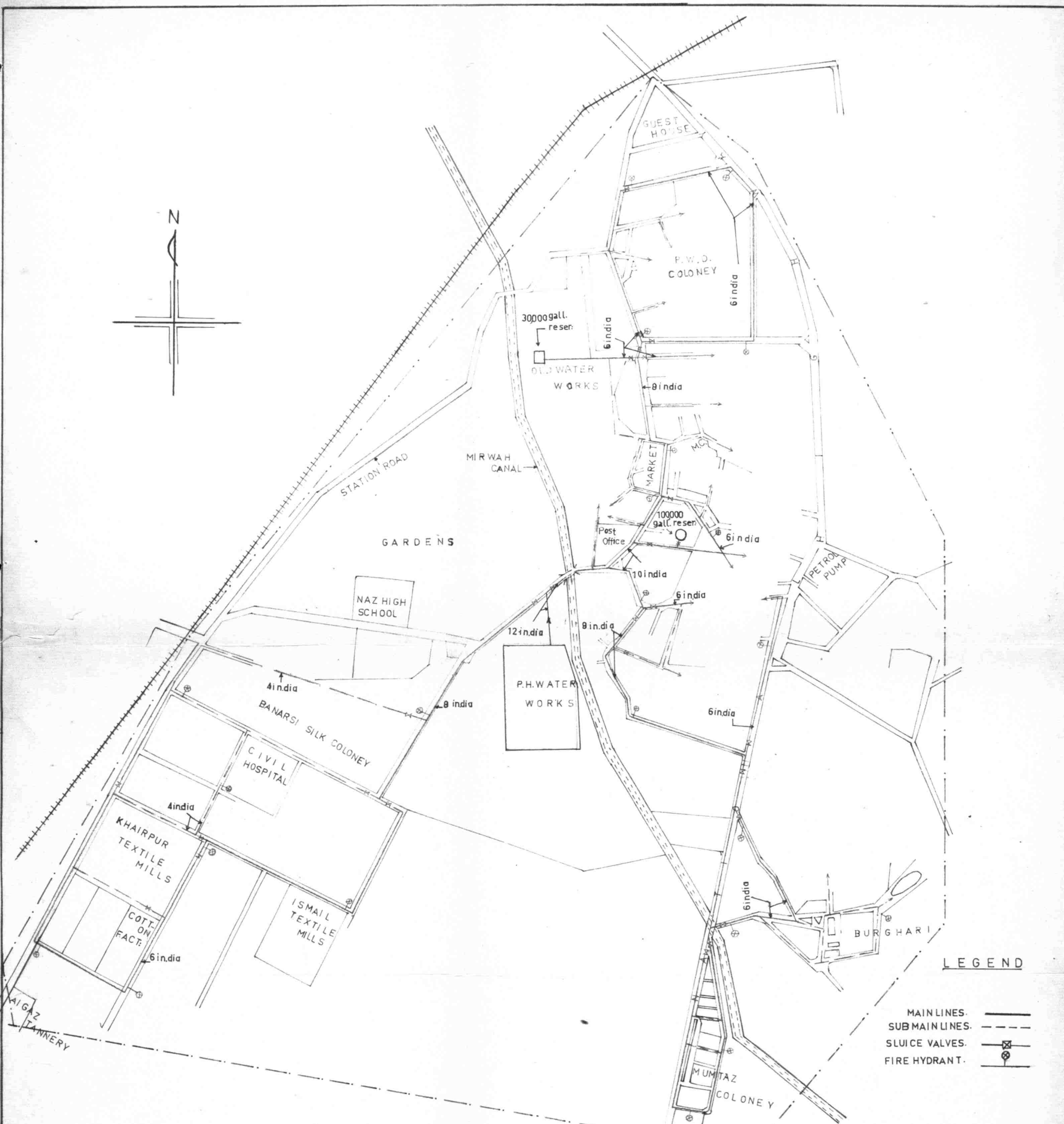
KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME

PRESENT & FUTURE POPULATION DENSITIES

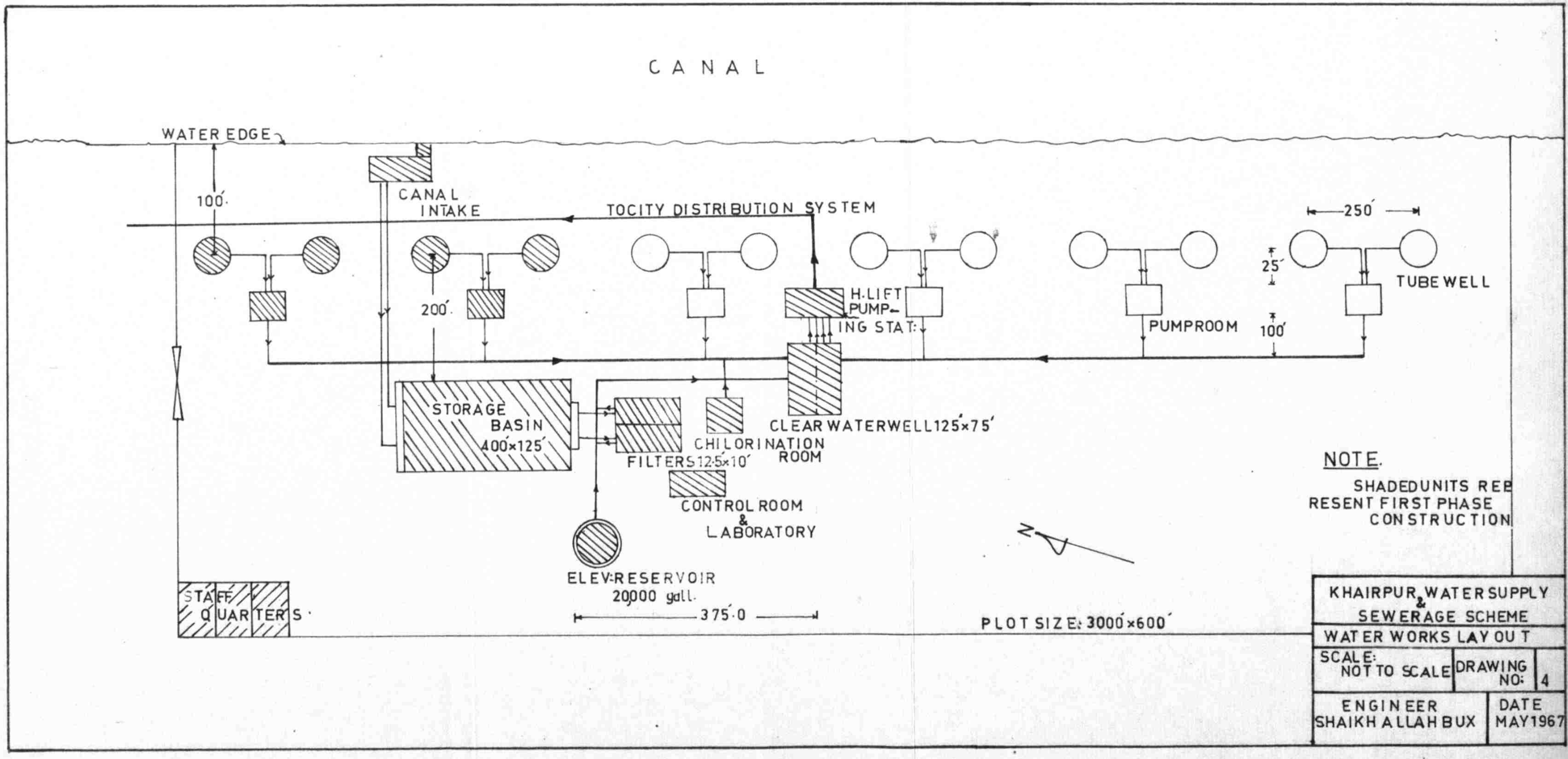
SCALE - 1" = 800'. DRAWING NO. 2

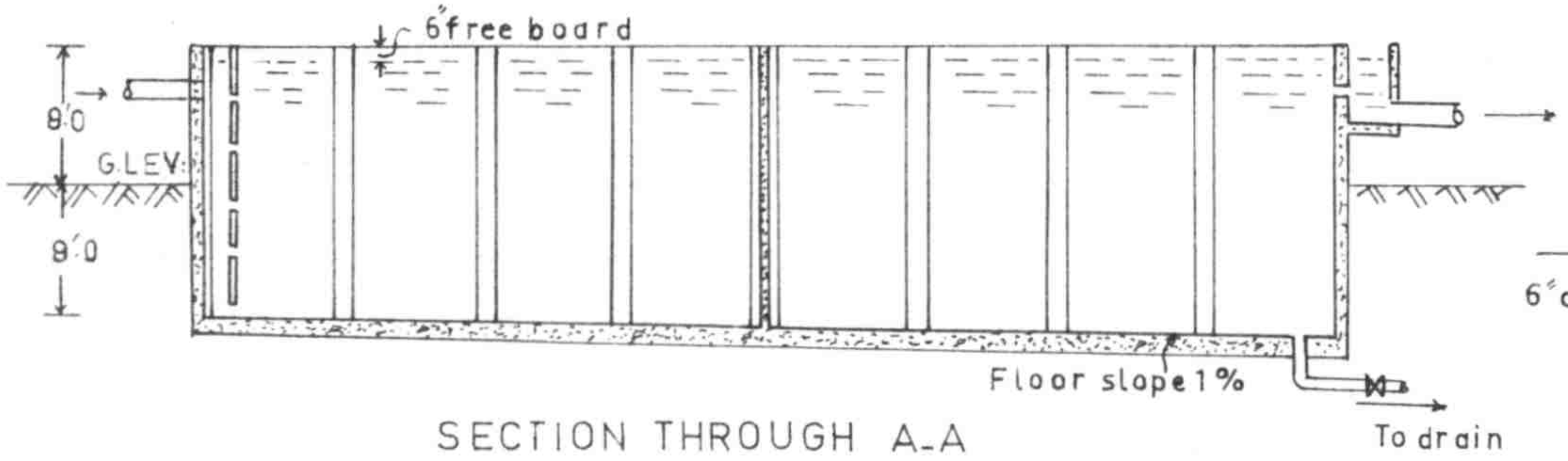
ENGINEER SHAIKH ALLAH BUX

DATE MAY 1967

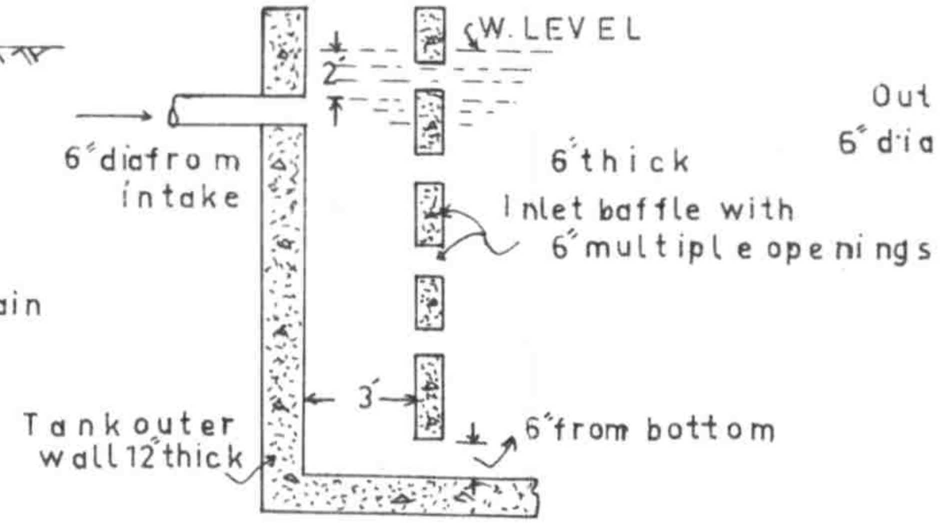


KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME	
WATER DISTRIBUTION SYSTEM	
SCALE- 1"= 800'.	DRAWING NO. 3
ENGINEER SHAIKH ALLAH BUX	DATE MAY 1967

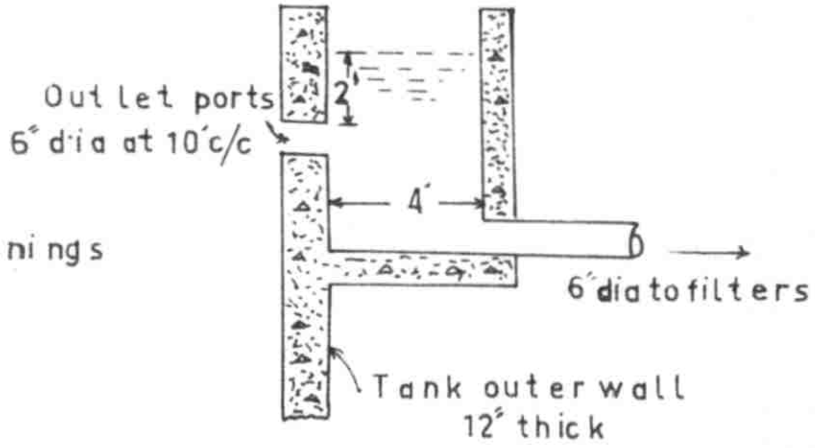

CANAL
WATER EDGE
100'
CANAL INTAKE
TOCITY DISTRIBUTION SYSTEM
250'
25'
TUBEWELL
200'
H·LIFT PUMP-ING STAT:
PUMPROOM
100'
STORAGE BASIN 400×125'
FILTERS 12·5×10'
CHILORINATION ROOM
CLEAR WATERWELL 125'×75'
CONTROL ROOM & LABORATORY
ELEV:RESERVOIR 20000 gall.
STAFF QUARTERS
375·0
PLOT SIZE: 3000'×600'
NOTE.
SHADEDUNITS REPRESENT FIRST PHASE CONSTRUCTION
N
KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME
WATER WORKS LAY OUT
SCALE: NOT TO SCALE
DRAWING NO: 4
ENGINEER SHAIKH ALLAH BUX
DATE MAY 1967



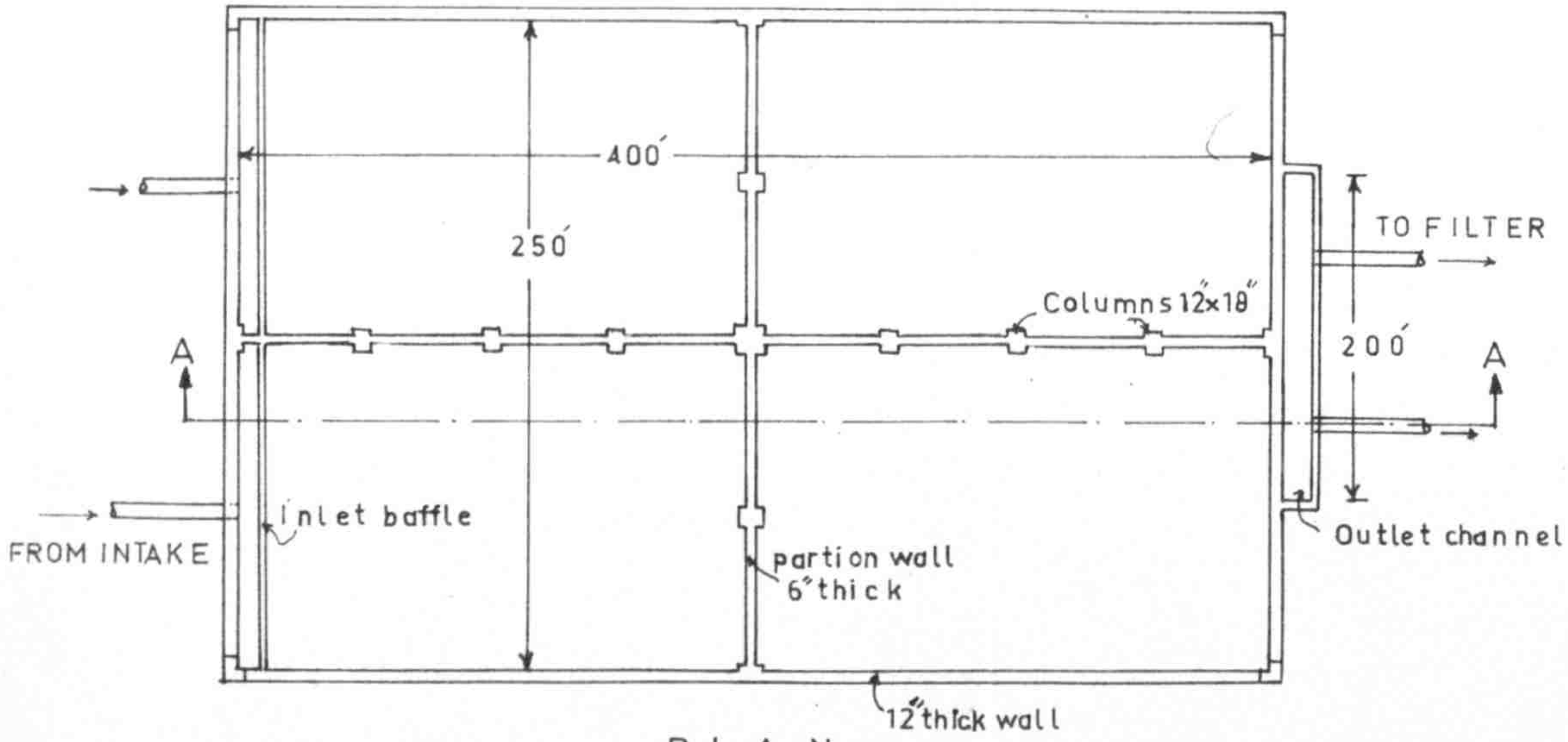
SECTION THROUGH A-A



DETAILS OF INLET

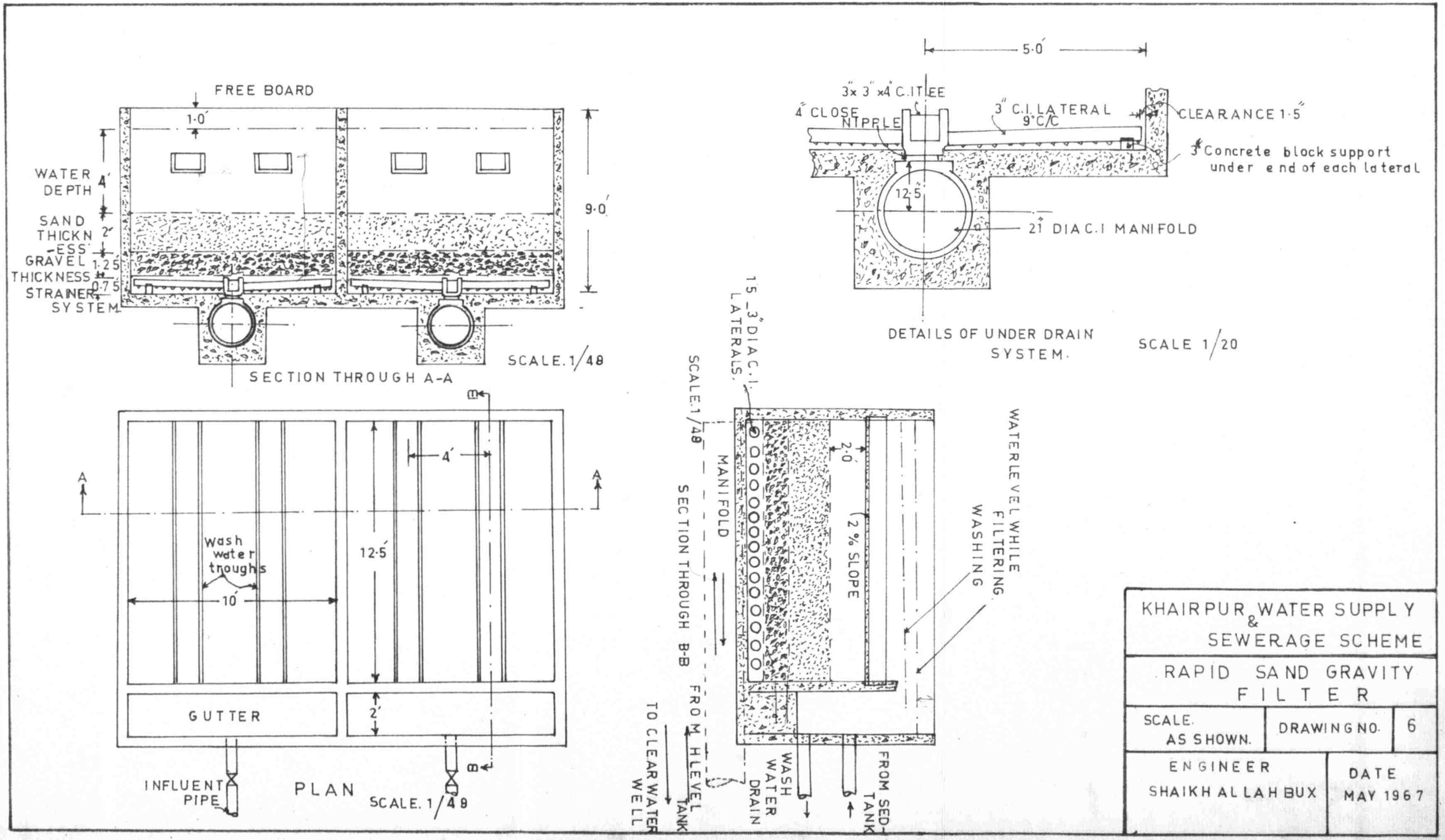


DETAILS OF OUTLET



PLAN

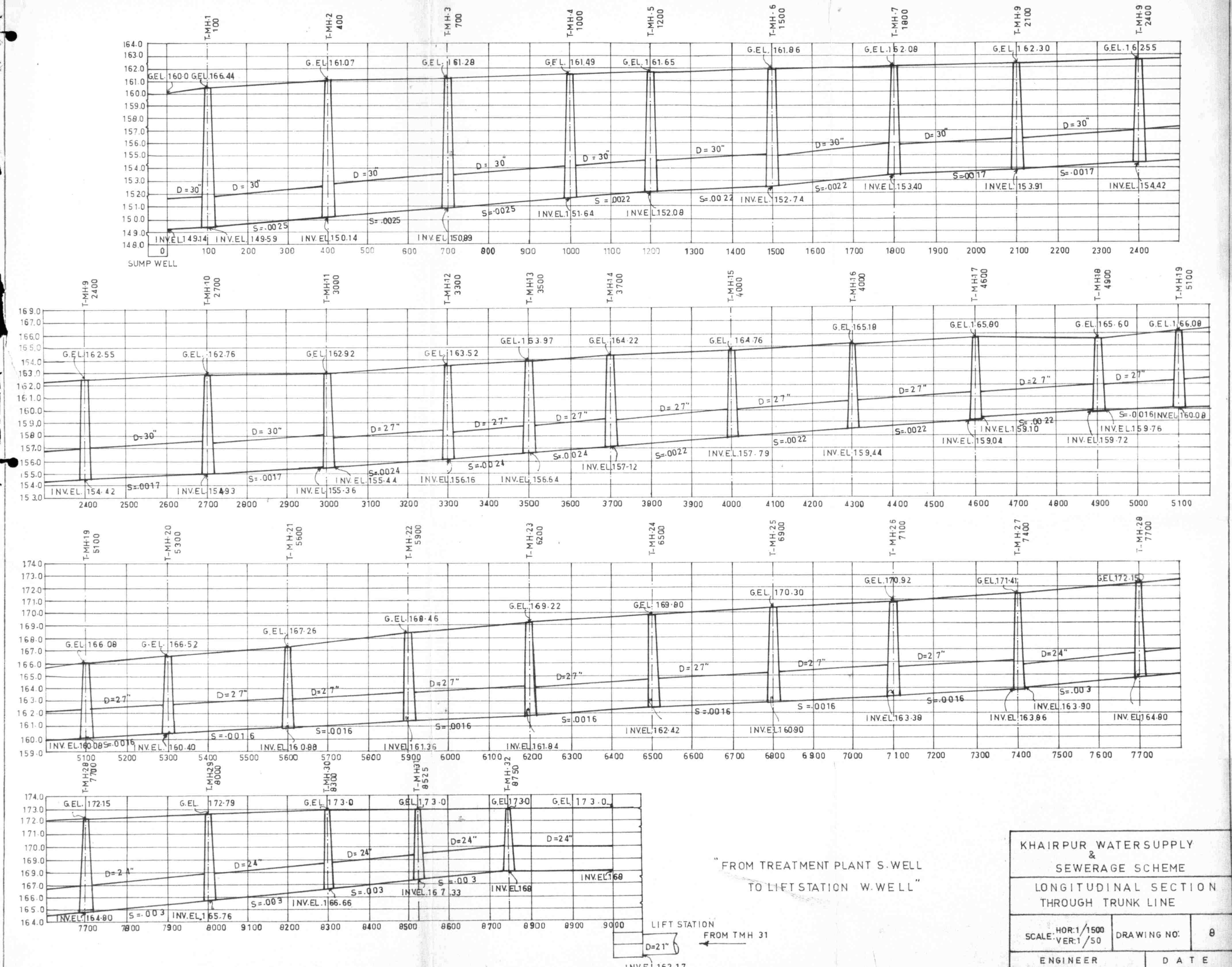
KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME	
SEDIMENTATION / STORAGE TANK	
SCALE: NOT TO SCALE	DRAWING NO: 5
ENGINEER. SHAIKH ALLAH BUX	DATE. MAY 1967


FREE BOARD
1·0'
WATER DEPTH 4'
SAND THICKN-ESS 2'
GRAVEL THICKNESS 1·25
STRAINER SYSTEM 0·75
9·0'
SCALE. 1/48
SECTION THROUGH A-A
5·0'
3" x 3" x 4" C.I. TEE
4" CLOSE NIPPLE
3" C.I. LATERAL 9" C/C
CLEARANCE 1·5"
3" Concrete block support under end of each lateral
12·5"
21" DIA C.I MANIFOLD
DETAILS OF UNDER DRAIN SYSTEM.
SCALE 1/20
A
A
B
4'
Wash water troughs
10'
12·5'
2'
GUTTER
INFLUENT PIPE
PLAN
SCALE. 1/48
B
15 _3" DIA C.I. LATERALS.
SCALE. 1/48
MANIFOLD
SECTION THROUGH B-B
2·0'
2% SLOPE
WATER LEVEL WHILE FILTERING
WASHING
FROM H LEVEL TANK
TO CLEAR WATER WELL
WASH WATER DRAIN
FROM SED-TANK
KHAIRPUR & WATER SUPPLY SEWERAGE SCHEME
RAPID SAND GRAVITY FILTER
SCALE. AS SHOWN.
DRAWING NO. 6
ENGINEER SHAIKH ALLAH BUX
DATE MAY 1967


KHAIRPUR & WATER SUPPLY SEWERAGE SCHEME.
CATCHMENT AREAS & LAY OUT OF MAIN & TRUNK SEWERS
SCALE 1 inch = 400 feet
DRAWING NO: 7
ENGINEER SHAIKH ALLAH BUX
DATE MAY 1967.
TO KARACHI
LUQMAN
NAZ HIGH SCHOOL
COL. SHAH HOSTEL
SHOPS
P.H. WATER WORKS
MARKET
PETROL PUMP
DABAR
BURGHARI
MUMTAZ COLONEY
R.C.C. PIPE FACATORY
CIVIL HOSPITAL
KHAIRPUR TEXTILE MILLS
COTTON FACTORY
AGRICULTURAL WORKSHOP
ISMAIL TEXTILE MILLS
BARREN LANDS
SUMP WELL
SEWEAGE TREATMENT WORKS
LIFT STATION
MAIN SEWER "F"
MAIN SEWER "M"
MAIN SEWER "G"
MAIN SEWER "H"
MAIN SEWER "I"
MAIN SEWER "J"
MAIN SEWER "K"
MAIN SEWER "L"
MAIN SEWER "Q"
MAIN SEWER "R"
MAIN SEWER "S"
MAIN SEWER "T"
MAIN SEWER "U"
MAIN SEWER "V"
1/2

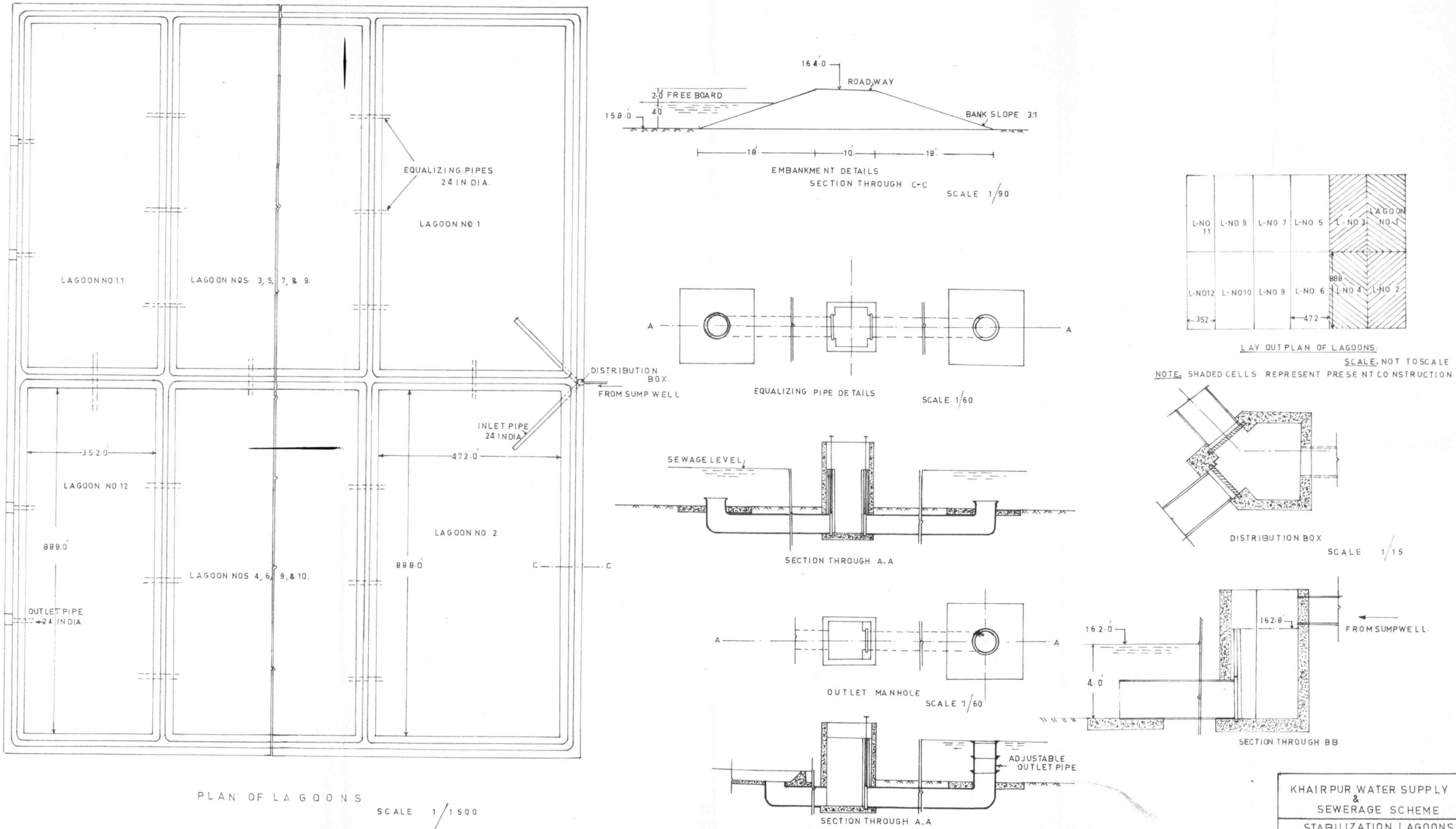
2/2

TO SUKKUR
KHAIRPUR STADIUM
CENTRAL JAIL
GUEST HOUSE
S.P. HOUSE
D.C. HOUSE
COMMISSIONER'S HOUSE
MAIN SEWER "A"
P.W.D. COLONEY
FAIZ MAHAL
MAIN SEWER "B"
LIBRARY
GYMKHANA
MAIN SEWER "C"
HUTS
OLD MUNICIPAL W. WORKS
MAIN SEWER "P"
WAPDA OFFICES
POLICE HEAD QUARTER
MAIN SEWER "D"
D.C. OFFICE
MAIN SEWER "O"
MAIN SEWER "E"
SCHOOL
AGRICULTURAL LANDS
N
E
W
TO LAHORE
STATION ROAD
GARDENS
M. COMMITTE
MAIN SEWER "N"
MAIN SEWER "F"
MARKET
MAIN SEWER "M"
MAIN SEWER "G"
TO KARACHI
LUQMAN
LIFT STATION
PETROL PUMP
NAZ HIGH SCHOOL
SHOPS
MAIN SEWER "H"
COL. SHAH HOSTEL
P.H. WATER WORKS
DABAR
MAIN SEWER "I"



"FROM TREATMENT PLANT S.WELL TO LIFT STATION W.WELL"

KHAIRPUR WATERSUPPLY & SEWERAGE SCHEME			
LONGITUDINAL SECTION THROUGH TRUNK LINE			
SCALE: HOR: 1/1500 VER: 1/50		DRAWING NO:	8
ENGINEER		DATE	
SHAIKH. ALLAH BUX.		MAY 1967	



KHAIRPUR WATER SUPPLY & SEWERAGE SCHEME	
STABILIZATION LAGOONS LAY OUT & DETAILS	
SCALE AS SHOWN	DRAWING NO 9
ENGINEER	DATE
SHAIKH ALLAH BUX	MAY 1967