IMPROVEMENT OF DESIGN, OPERATION AND MAINTENANCE OF FOUL SEWERAGE SYSTEM IN BAHRAIN

by

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ABSTRACT

A study for a foul sewerage system is being considered for Bahrain Island. There is a need for a system to cope with the development of the island and also to minimise the risk to public health and of environmental pollution.

An improvement study is needed for the existing foul sewerage system, which is to be incorporated with newly designed systems in the future.

The study will concentrate upon the existing system and on how it may be possible to up-grade the existing system so as to be able to combine it with the new design, which could lead to a much improved system overall. The intention is to minimise both operation and maintenance costs. In regard to these considerations the author felt it would be essential to make a study of improvements for the design, operation and maintenance of foul sewerage systems.

The conclusions reached in the project may help to control the problems which already exist in the foul sewerage system in Bahrain so as to prevent any further risk to the environment.

EXECUTIVE SUMMARY

1. INTRODUCTION

Bahrain consists of several low lying Islands in the Arabian Gulf located between the Saudi Arabian coast and the Qator coast. The largest Island is Bahrain Island (Figure 1.1) and the two other main Islands are Muharraq and Sitra. The Bahrain climate is hot in summer (June, July and August) with a temperature which rarely falls below 30°C. Rainfall usually occurs in the winter months (January, February and March) with an average temperature of 17°C. The population in Bahrain is 350 798 according to the Census in 1981 and the population for the year 2001 is expected to be 544,061 with an approximate 4% rate of growth.

Sewerage schemes are used in the populated areas of the Islands. These are mainly found in the northern part of Bahrain Island and over much of Muharraq and sitra Island. The excavation for sewerage are up to 5m below the ground level. This is because the artesian aquifers are situated at about 7m from the ground surface and this aquifer is the main source of the water supply to Bahrain Island. The main sewerage project started in Bahrain in 1977, when construction began on 60 km of trunk sewers and 34 associated pumping stations. Since then the sewerage mains have been developed and have been connected via the trunk sewer to the treatment plant in the Tabli catchment area (Figure 1.2), (Figure 1.3) and (Figure 1.4).

2.0 REVIEW AND IMPROVEMENT OF EXISTING FOUL SEWERAGE SYSTEMS

2.1 DESIGN

The average flow in sewers was recorded in different towns in Bahrain in 1974/75 and in 1984/85. The average flow per person including an allowance for abuse is 228l/d

excluding infiltration. The design peaking factors are based on international practises and also taking into account recorded sewer data. Formulas have been developed on this basis.

Design Peaking factor = $((1.5756 \times 10^7)/P)^{0.158}$

Daily Peaking factor = $((3.75 \times 10^7/P)^{0.078}$

where P is the design Population.

These two formulas are used for a population between 300 to 200 000 and for population above 200 000 a fix design Peaking factor of 2.0 and daily Peaking factor of 1.5 have been assumed. From recorded data it is found that the safety factors range from 1.8 to 2.4 (Figure 3.1).

A spreadsheet has been developed by the author to assist in the designing of sewers. This allows the designer to check that minimum self cleaning velocities are obtained at partial depths of minimum sewer gradients. This approach is based mainly on the Colebrook-White equation. This is illustrated in the solved example given below.

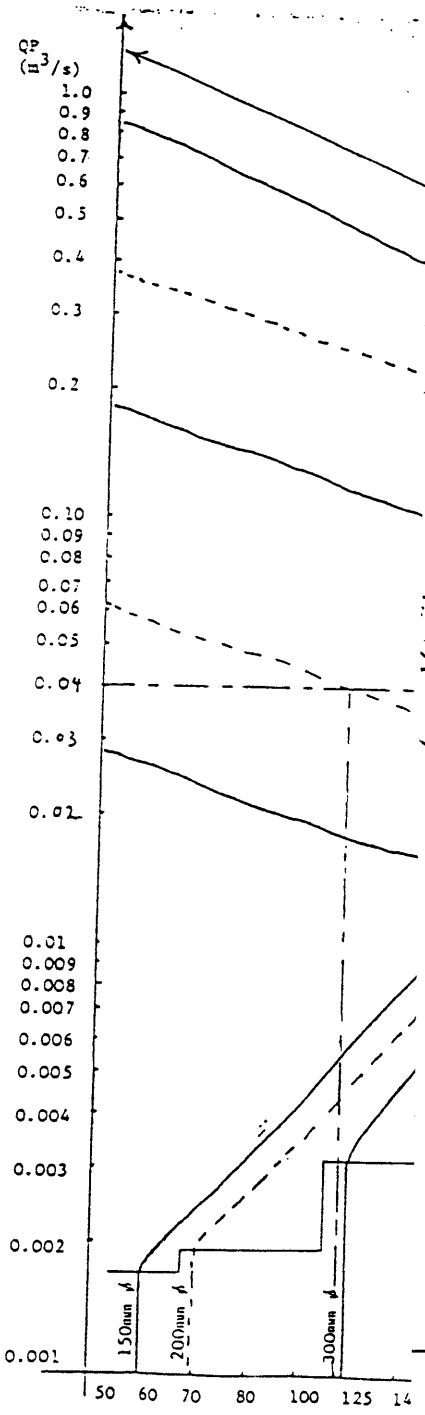
Example (Fig. 3.2)

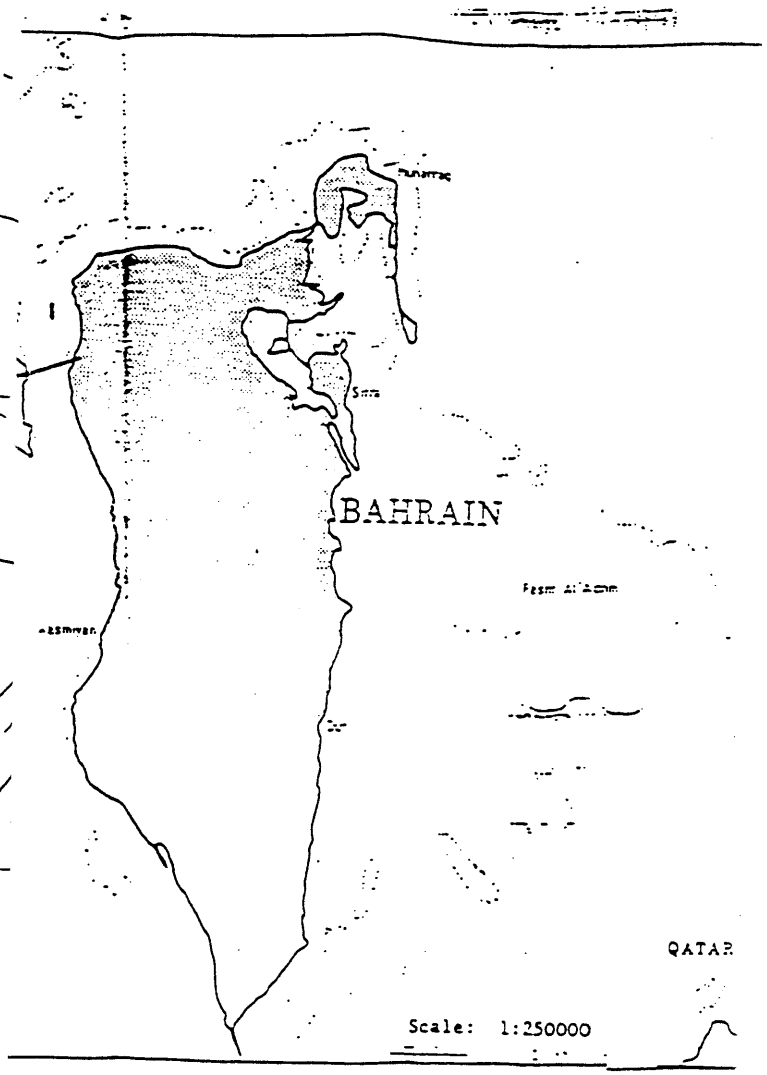
Usage of the graph in the Design Office

If the estimated flow is to be 40/L/s, then

1. For 200mm dia. is the maximum flow with 0.83% slope
2. For 300mm dia. is the maximum flow with 0.125% slope
3. For 400mm dia. is the maximum flow with 0.22% slope
4. For 500mm dia. is the maximum flow with 0.20% slope

This means that for a flow of 40 L/s, four different alternatives could be considered in designing a sewer line. This could be an advantage to a design office when selecting a suitable pipe diameter with minimum slope that could suit their conditions.





Source: (Hawksley, 1985)

Figure 1.1
Map of Bahrain

2.2 ACCESS CHAMBERS

2.2.1 HOUSE CONNECTIONS

The access to the sewerage from the house drain is provided through an inspection chamber which is located outside properties.

2.2.2 MANHOLES

Circular manholes constructed from concrete with GRP liners are used in Bahrain.

2.3 PUMPING STATIONS

2.3.1 DISTRICT PUMPING STATIONS

The circular type of station with GRP liner has generally been used for pumping lift and rising main stations with a maximum flow of 300 L/s.

2.3.2 MAIN PUMPING STATIONS

Main pumping stations are defined as stations operating on trunk sewer lines. They could be split into:

1. Lift station using submersible pumps to lift the sewage into a collector channel which discharges the sewage into the next trunk sewer.
2. Rising pressure main stations consisting of wet well/dry well.

2.4 GRIT HANDLING

Grit handling in the sewerage systems in Bahrain varies from large pumping stations small pumping stations. The amount of grit removal depends upon four factors, which are the stations' location, the degree of the development in the catchment area, the

height of the water table in respect of the sewer line and surface water (since the environment is full of grit).

2.5 ODOUR CONTROL

Due to high temperature, the odour problem is high in Bahrain and several methods have been used. The author recommends another type of odour control which is air injection.

2.6 OVERFLOW

Overflows are only installed at main or trunk pumping stations because if there is a pump or power failure it will result in flooding low lying areas. Overflow in a district pumping station is not necessary because the flow can be stored for a short time in the sewerage system itself.

3. CONCLUSION AND RECOMMENDATION

3.1 CONCLUSION

As has been seen from this report, the Bahrain Islands are low lying areas with very little rainfall and high temperature (28°C-40°C). Due to the flatness of the ground a flat sewer system is being constructed with a large number of pumping stations, which need many suction tankers and a large number of labourers. Due to the lack of labourers and machinery in respect of the number of pumping stations. This causes delays to the desilting process from the sump and increases the amount of silt in the sump before it can be removed (lack of rainfall caused the presence of a high amount of grit in the environment). Therefore, accumulation of silt or grit in the sewer line

will prevent movement of sewage in the system.

In order to minimise the operation and maintenance requirements of the sewer network, the deposits can be removed at various points and by applying various methods. Also, a well managed system of removal is required. Because of delay of sewage in the sewer line with the presence of high temperature, this means that the septicity problem will be high. Different types of equipment have been used to solve the septicity problem but none of them are very satisfactory.

In order to improve the sewerage system in Bahrain a number of recommendations have been recommended by the author.

3.2 RECOMMENDATIONS

- In designing any sewerage system for Bahrain, consideration should be taken into account so that the sewerage should be kept moving continuously to the treatment works (Section 1.3).
- Each town should have its own treatment works so as to reduce the grit and septicity problems and also utilise a smaller land area for treatment plants (Section 1.5).
- There were no significant changes in average daily flow in a 10 year period; therefore, it is preferable to retain the safety factors at their present levels in order to guard against such uncertainties as pump performance, abuse, rainfall and surcharging, etc (Section 3.12).
- A spreadsheet has been developed to assist in the designing of sewers. This allows the designer to check that minimum self cleaning velocities are obtained

at partial depths of flow in a minimum sewer gradient (Section 3.2).

- Initial and long term velocities should be considered when designing any sewer line (Section 3.2).
- Inspection chambers which are not located in the road, could be constructed from PVC material. Also, an inspection point could be constructed if the depth is shallow (Section 3.3.1).
- To prevent hydrogen sulphide and acid gaining ingress into the concrete in manholes, a number of aspects should be considered and are as follows:

  Specified GRP liner thickness and clay wave channels are to be used, then application of epoxy mortar should not be less than 10mm thickness, sharp edges should be avoided, attention should be paid to the detailing of joints between and in the incoming channels, coal tar coatings are not to be applied on benching, no ladder or iron steps are to be installed in manholes and sealing plates of GRP material. They should be provided below the manhole covers (Section 3.3.2).

- To assist accurate future planning, flow measurement equipment should be installed in the main pumping stations, which will enable the MWPW and MOH to proceed with the design of new stations and the renovation of existing stations. Also it will introduce new designs, or confirm old ones, concerned with grit removal from the pumping stations or the rising main (Section 3.4.3).
- Three different types of grit handling should be used in Bahrain so as to

simplify the desilting process from the pumping stations, as follows:

A) Pump operation and intake.

B) Desilting of Manholes.

C) Desilting Systems. (Section 3.4.3.1)

- Better management could be achieved by providing an inventory of time, data, quantity of silt and location of each station. This not only gives management a better idea of performance, but it improves that performance by demonstrating that it is being monitored. (Section 3.4.3.1).
- Air injection should be used instead of oxygen injection or scrubbing plants in order to control odour (septicity) problems in the sewerage system in Bahrain. (Section 3.4.3.2).
- In constructing any open overflow reservoir for a pumping station, it should be built so that it can be used as an overflow for the pumping station and at the same time for land application purposes and finally a storm basin. (Section 3.4.3.3).
- A close overflow reservoir should be used for a pumping station which has no overflow facilities or for a pumping station which is discharging its overflow into the sea. (Section 3.4.3.3).
- Routine maintenance to be carried out at the same time as desilting and cleaning. (Section 4.2.4).
- Instrumentation, control and Automation (ICA) studies are to be carried out. (Section 4.3.4).
- For better institutional development there should be direct cooperation between the MOH and MWPW engineers. (Section 5.0).

the MOH and MWPW engineers. (Section 5.0).

- Standard details and design parameters should be adopted at the beginning of any project. (Section 5.1).
- Ensure adequate supervision of service connections to the main sewer. (Poor branch connections can result in the blockage of the main sewer). (Section 5.2).
- Establish an improved system of recording and analyzing by computer which would reduce design time, improve efficiency of operation and maintenance and provide a better means of costing and budgeting for preventative maintenance and remedial work. (Section 5.3).

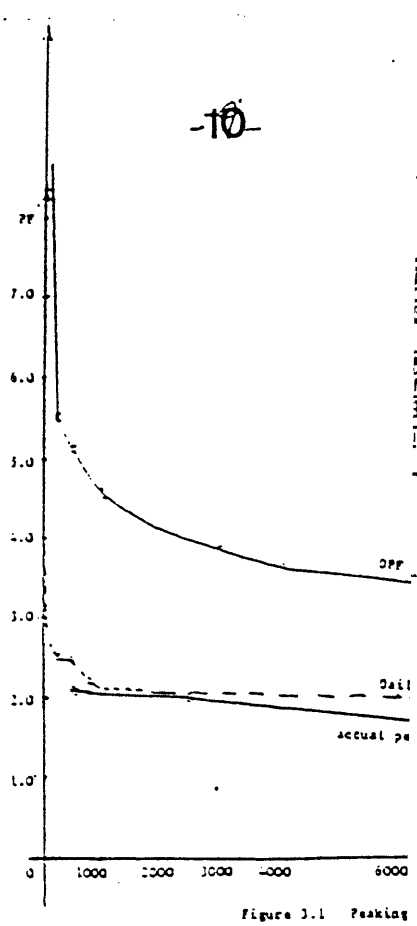
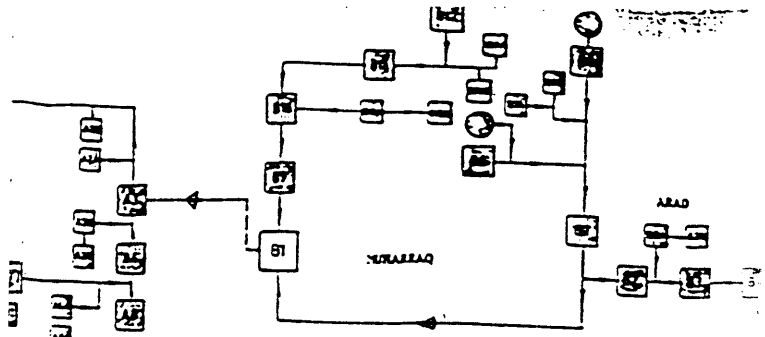
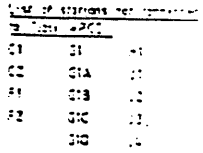
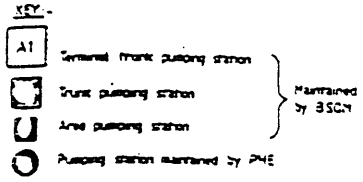


Figure 3.1 Peaking

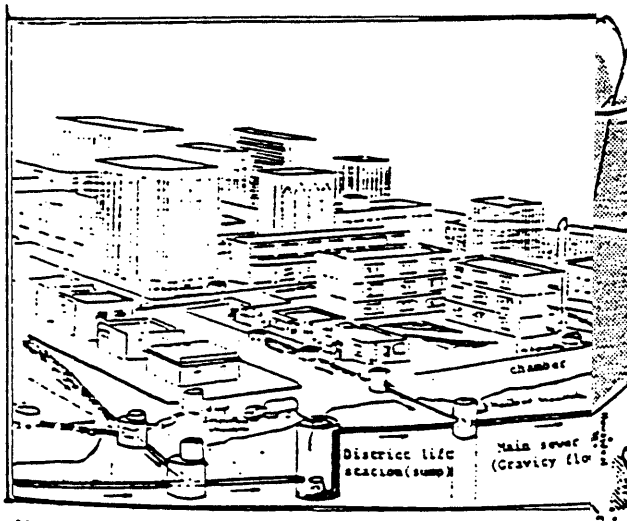




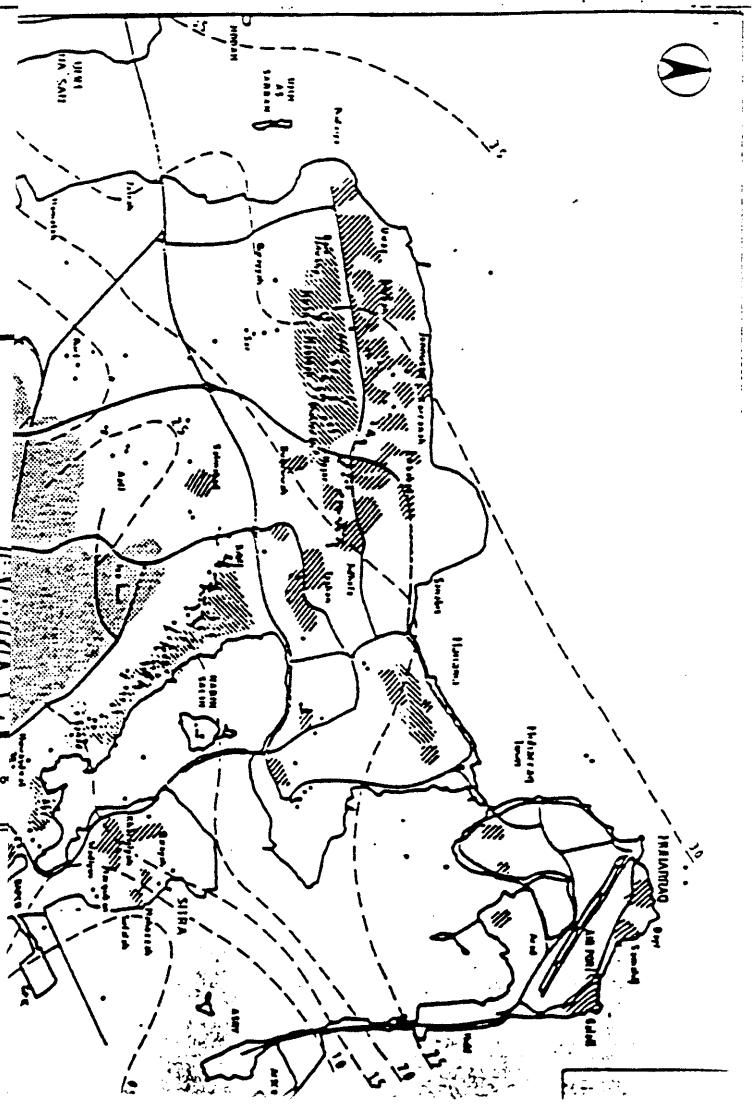


1.1 Typical diagram showing pumping scation layout
(Hawksley, 1985)

FIG.1.



Blue colour (pumped flow)
Orange colour (gravity flow)



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(2) Spreadsheet details

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CHAPTER ONE

1. INTRODUCTION

1.1 GEOGRAPHY AND TOPOGRAPHY

"Bahrain consists of several low lying islands in the Arabian Gulf located between the Saudi Arabian coast and the Qatar Coast. The largest island is Bahrain Island. (Figure 1.1).

A) Bahrain Island

Bahrain Island measures 48 km from north to south and 16 km from east to the west at its widest point. (Figure 1.1).

The northern coastal strip of Bahrain Island is low lying, rarely exceeding 5m above mean sea level, some of the highest ground being in the centre of Manama itself, the capital city.

South of the northern coastal strip is the central region, which is despite its name, within the northern quarter of Bahrain Island.

From the west coast which has some vegetation and agriculture, the ground rises to a maximum of 20m above mean sea level to an old area, which was used as burial mounds, 3km inland.

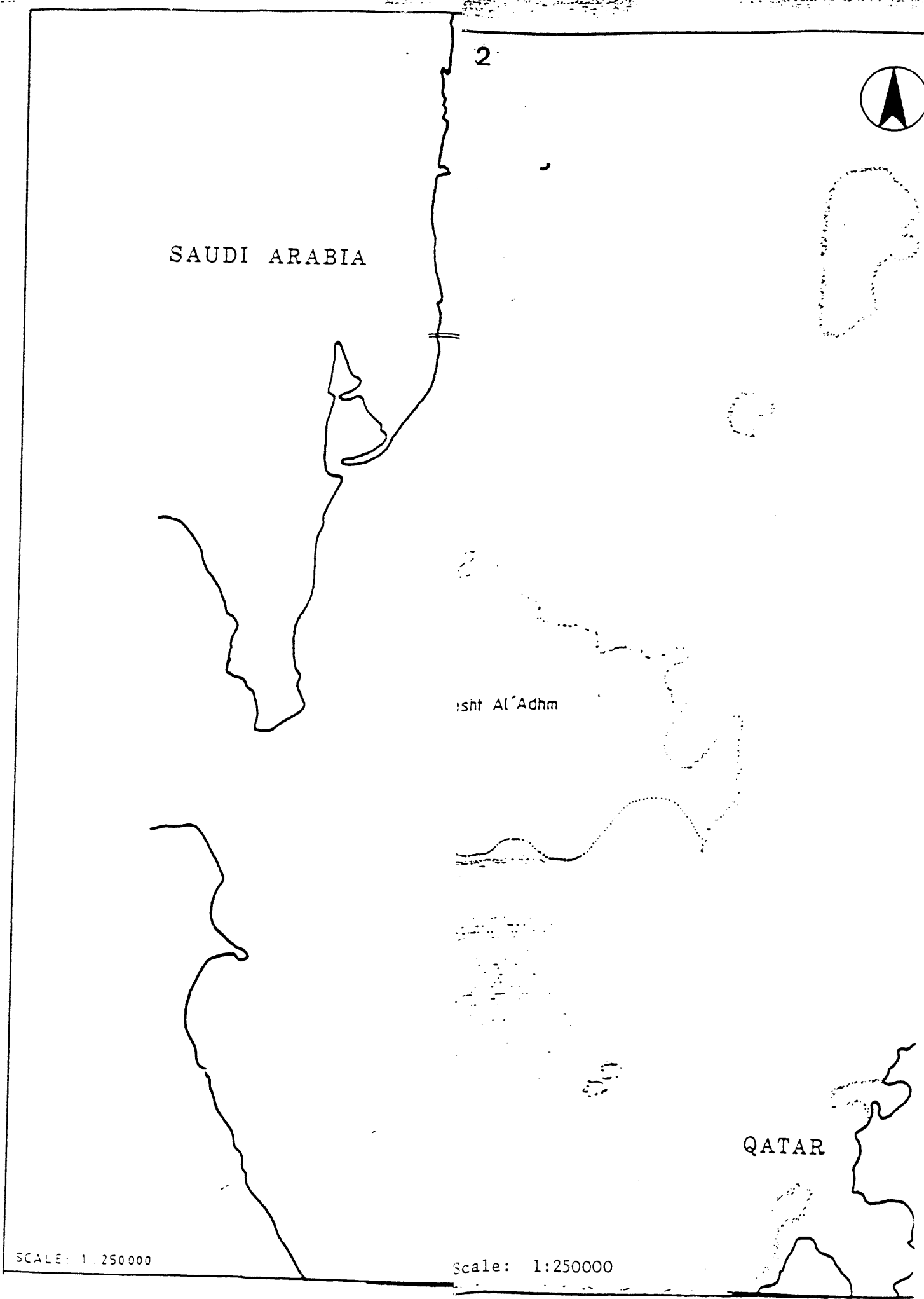


Figure 1.1
Map of Bahrain

85)

Due south of Isa town, is a depression area 3m above sea level and the start of the gradual slopes up to Riffa, 5km to the south resting place on the rim rock. The rim rock, 40 to 50m above mean sea level and forms a totally enclosed, well-defined escarpment, containing an area measuring 22km from north to south and 7km from east to west. The area is the central region of Bahrain Island. Around this area the ground slopes down towards the sea. Inside, it drops steeply to 20m, before rising towards the centre and the Jebel Dukhan, height 138m above mean sea level and the highest point in Bahrain". (Hawksley, 1985).

B) Muharraq Island

Muharraq Island is linked to Manama (or Bahrain Island) by a causeway and is the second major inhabited Island in the Bahrain group of Islands.

It is small compared to Bahrain Island, measuring no more than 6km from east to west and 5km from north to south. The whole of Muharraq Island is low lying, rarely exceeding 5m above mean sea level". (Hawksley, 1985).

C) Sitra Island

Sitra Island is linked to the east coast of Bahrain Island. The Island is low lying, never exceeding 4m above mean sea level and the southern part of the Island is extremely low lying, 1m above mean sea level. At its widest the Island measures 3km and runs approximately 6km from north to south, (Hawksley, 1985).

This means that implementation and design of sewer network for low lying areas will

produce nearly flat system which will mainly require lifting stations, so that the sewage can be transferred to the treatment works.

Therefore proper design is needed which would suit the situations so as to minimize operation and maintenance requirement.

1.2 GEOLOGY AND HYDROGEOLOGY

A) Geology

Excavation for sewerage schemes has generally been in the northern part of Bahrain. It is confined mainly to Manama and Muharraq. The conditions encountered are extremely variable but normally they involve excavation in superficial deposits of sand or silts. Underlying these are sporadically occurring layers of Caprack which are further underlain by yellow siltstone. The yellow silt and sand deposits have reached a depth of about 7m. Caprack in Manama and Muharraq often occurs in densities of 0.3m and 1.0m thicknesses and usually has a high permeability. Where siltstone directly underlines the Caprack, it can act as a carrier for groundwater and extensive de-watering can be required for excavations. The siltstone, is impermeable but may contain fissures which act as paths for the water under artesian pressure reaching the surface. It is believed that no sewerage excavations have penetrated the siltstone through to the underlying aquifers. (Hawksley, 1985).

B) Hydrogeology

Bahrain Island is fed by the Eocene aquifer system which carries water from the Saudi Arabian mainland. The system consists of three aquifers:

a) The Alat or "A" aquifer nearest the surface.

b) The Khobar or "B" aquifer below "A" but close to it.

c) The Umm or Radhuma or "C" aquifer which is the deepest.

Above the aquifers and the overlying Capracks, old water tanks are common, particularly in the low lying area nearest to the coasts.

(Table 1.1) gives details of ground water levels and analyses, (Figure 1.2) (Hawksley, 1985).

As has been mentioned, the sewage schemes are built in areas where an old ground water table exists. This will cause the pipe material to deteriorate and other infrastructure. Hence, appropriate pipe material should be used so as to stand against external deterioration and also special protection material or coating should be applied on manholes and other infrastructure to give it a longer life.

Table 1.1 Groundwater Records

Location	Water Depth (m)	Permeability (m/s)	Ground Water Analysis So_3 (g/l)	Cl (g/l)	pH
Budaiya	0.50	1.72x10-5	2.14	6.05	7.5
Budaiya	0.60	5.82x10-5	1.24	6.55	8.0
Salmabad	0.60	Nil	2.16	11.80	7.5
Hamalah	0.75	5.35x10-5	3.69	22.45	8.3
Malikiyah	0.80	4.50x10-5	2.05	4.15	9.1
Malikiyah	0.90	7.15x10-5	2.07	3.80	8.1
Karbabad	0.95	2.58x10-5	1.11	5.95	7.7
Dimistan	0.95	6.13x10-5	2.66	12.30	7.9
Barbar	1.00	5.86x10-5	3.86	13.00	7.9
Zallaq	1.25	2.18x10-5	E.71	21.20	7.1
Karranah	1.30	1.30x10-5	4.15	19.85	7 5
Karzakkan	1.30	7.57x10-5	3.65	33.35	8.8
Dar Kulayb	1.60	2.28x10-5	2.19	4.10	8.0
Hidd	2.00	3.94x10-5	2.66	21.45	7.8
Karranah	2.00	6.14x10-5	3.81	22.25	7.4
Shahrakkan	2.00	1.90x10-5	2.17	4.25	7.8
Tashan	2.50	4.35x10-5	1.87	9.30	7.6
Sar	2.93	2.93x10-5	-	-	-

Source: Hawksley, 1985.

1.3 CLIMATE

The Bahrain climate is hot and dry, with relatively high humidity in the summer months. Rainfall has never occurred during the summer months which are June, July and August. Most of the rainfall occurs during the months of January, February and March. The hottest months of the year tend to be July and August and the temperature rarely falls below 30°C. The coldest month tends to be January and February and the average temperature will be 17°C. (Hawksley, 1985).

For three months of the year the temperature is normal (average 17°C), while the other nine months the temperature is high (28°C - 40°C) with no rainfall.

High temperature with delayed sewage in a sewage system will lead to high septicity problems. Therefore in the designing of any sewage system for Bahrain this consideration should be taken into account so that the sewage should be moving continuously until it reaches the treatment works.

1.4 POPULATION

"The last census in 1981 estimated the population to be 350,798 (238,420 Bahrainis and 112,378 non-Bahrainis). The growth rate of Bahrainis between 1971 and 1981 averaged at 2.85% per annum whereas that of non-Bahrainis averaged 11.49% per annum. Under-estimation of the Census figures has been investigated and found not to be significant. The under-estimation was found to be about 4.4% overall, and hence it is unlikely to be of significance in the overall national strategy planning for sewerage and treatment, the programme of which would have to be adjusted according to the actual population achieved. (Table 1.2) (Hawksley, 1985).

Table 1.2 1991 and 2001 Population by Census Region

Census Region	Population 1981	Population 1991	Population 2001
Hidd	7,280	7,567	8,500
Muharraq	63,075	67,652	71,250
Manama	125,156	138,833	165,366
Jidhafs	35,166	42,732	68,926
Northern	22,372	26,707	30,500
Sitra	23,790	25,487	28,250
Central	17,294	20,921	25,100
Isa Town	22,004	26,707	28,300
Riffa	28,910	33,384	37,634
Western	15,080	18,695	23,735
Hamad Town	-	35,850	56,500
TOTAL	*360,127		544,061

*Adjusted for under-enumeration (Source: Hawksley, 1985).

From the last census it is shown that non Bahraini growth is much more than Bahraini growth. This is caused by the movement of Bahrainis from low income homes in the central Manama and Muharraq and non Bahraini families placing them in old houses. Each Bahraini family was staying in a house while two or three families of non-Bahraini nationality were staying in the house. This is an increase in the number of persons by two or three times. Therefore, a safety factor has to be considered for this reason in designing any sewerage system for Bahrain.

1.5 EXTENT OF SEWERAGE

Since 1977, when construction under the Bahrain sewerage project began, 60km of trunk sewers and 34 associated pumping stations have been constructed.

Construction of mains sewerage commenced in 1979. By the end of 1984, the connected population to the Tabli Water Pollution Control Centre (WPCC) was estimated to be 104,000 in Manama, Greater Manama, Isa Town, Riffa and Aali including those connected under other ministry schemes such as Hamad Town. A further population of approximately 19,000 had been connected in Muharraq, Arad and Hidd. Planned Muharraq flows have not been connected to the Manama system because of temporary treatment capacity limitations. Instead they are discharged into the sea. Outside the Tabli catchment area, sewerage mains have been provided in Nuwaydiratu and Bani Jamrah to the local Sewage Treatment works. The areas not yet served include parts of Manama and Muharraq, most of Riffa and the majority of the villages. (Figure 1.3) (Hawksley, 1985).

A typical sketch showing how foul sewerage systems operate in Bahrain. (Figure 1.4).

For increasing treatment capacity three options should be proposed.

A) Extending the existing sewerage treatment plant could be limited because of limited land available around the treatment plant. Pumping sewage from different areas might face different problems like building extra pumping stations and septicity. Hence some town and villages will be discharging into the sea which will cause an

effect on the environment.

B) Building a new treatment plant in other areas. This treatment plant might not serve all parts since the Towns and Villages are not located in one area. Hence, this is a similar problem to the first one.

C) Building a separate treatment plant for different Towns or Villages. This will save pumping costs including a number of pumping stations and septicity problems. Also the treated water could be used for either Irrigation or Landscaping, since it will be very near to that town or village.

An individual treatment could be dealt with much more easily than in a large treatment works. Therefore each town should have its own treatment works.

CHAPTER TWO

2. REVIEW OF EXISTING FOUL SEWERAGE SYSTEMS

2.1 DESIGN PARAMETERS

2.1.1 AVERAGE DAILY FLOW (ADF)

The average flow in sewers was recorded in different towns in Bahrain in 1974/75 and in 1984/85. (Table 2.1). The method was to take the average reading of the flows over each hour during a 7 day period (Table 2.2). Values for average daily flow (ADF) per person were then calculated from the known population figures for each town according to the census results. However, in some cases where a per capita flow had a high value, it was related to the high base flow (minimum flow during night). (Table 2.3).

Night flows usually give an indication that one of the following is occurring.

a) Infiltration of ground water into the sewerage system.

Infiltration of the ground water into the sewerage system occurs only when there is ground water, therefore, it cannot be considered as a night flow.

b) 'Abuse' (i.e. wastage of water).

Abuse in the sewerage system is often found to be from leakage of house plumbing by which the water flows directly into the sewerage system. This can be minimised by controlling the leakage occurring in the house fittings. Usually this can be controlled by applying a water meter in each house, therefore it cannot be considered as permanent night flow.

c) Normal usage by night shift workers.

Normal usage by night shift workers, or by night users of water cannot be controlled, therefore it is to be considered as night flow.

d) Industrial or commercial operations during the night.

If industrial or commercial premises are situated in the area being measured, they are also considered as night flow.

e) Draining down of the upstream systems.

Draining down from the upstream systems the flow travelling from a source to the treatment plant could take a few hours, so a safety factor should be considered in this situation. This flow is assumed as a part of permanent night flow.

f) Occasional pumping of ground water from excavations into the sewerage systems.

Sometimes the pumping of ground water from the excavation into the sewerage system will be not continuous. Pumping occurs intermittently when construction is taking place and therefore it is not considered as permanent night flow.

g) Connection of the storm water system into the sewerage systems.

Connection of the storm water system into the foul sewerage system is not allowed in Bahrain because the system is not designed for storm drain and also to prevent the sand from entering the system which usually exist in the road, but in the some places where flooding occurs during heavy rainfall, the manhole covers are removed so that the storm water can flow into the manhole.

Table 2.1 Recorded Average Daily flow

Data 1974/75 town	l/p/d	Data 1984/85 Town	l/p/d
Manama & Greater Manama	220	Manama	(189+→259*)=230 Av
Muharraq	197	Isa Town	234
Hidd	200	East Riffa	252
Isa Town	280	Nuwaydivat	250
Average	225	Isa Townwestern Appartments	189
		Average	231

+ minimum flow (l/p/d)

* maximum flow (l/p/d)

Av. is average flow

Source: Hawksley, 1985

Table 2.2 Average Daily and Per Capita Sewage Flows for Various Catchment Areas

LOCATION	PUMPING STATION	MONITORING PERIOD	AVERAGE DAILY FLOWS (l/sec) Hours run meters <--Recording Ammeter--> (Over 7 days)	7-day	6-day	Friday	*	ESTIMATED POPULATION	PER CAPITA FLOW (l/head/day)	REMARKS
Old Gufool	A1C	13/10-21/10	2.26	1.98	2.01	1.75		840	218	
Kanoo Gardens	A1D	27/8- 3/9	2.77	2.91	2.93	2.80	2.98	855	287	*3/9/84- see A1E. High base flow. S Burhamah and Kanoo Gardens flows combined
Kanoo Gardens	A1D	3/9	-	-	-	-	0.72	305	201	Kanoo Gardens flow only
S. Burhamah	A1E	21/10-29/10	2.65	2.67	2.67	2.66	-	610 p.e.	375	Very high base flow
S. Burhamah	A1E	3/9	-	-	-	-	2.27	610 p.e.	322	24 hour manual survey to compare with A1D
S. Manama	A5C	27/11- 3/12	1.55	1.51	1.48	1.63	-	310	421	Affected by power station
Hoora	A7C	5/9-16/9	9.18	8.42	8.69	9.64		2500	305	* Eld value. High base flow
E. Sanabis	A17D	8/11-15/11	3.36	3.00	2.99	3.06	-	1000	259	
Isa Town Nos 1-5		3/1-10/1	39.8	-	-	-	-	12450	276	High base flow. Hours run survey only
Isa Town	D6	25/11- 2/12	1.51	-	-	-	-	690	189	New flats. Friday 1/73 l/s. hours run survey only
E. Riffa	F2	19/11-25/11	-	1.24	1.23	1.31	-	425	252	New housing
E. Riffa	STW	25/11- 2/12	3.09	-	-	-	-	1060	252	Friday 3.34 l/s. Hours run survey only
Nuwaydirat	J1	3/10-13/10	9.24	10.09	9.75	10.89	9.57	3200	261	* Eld value
Nuwaydirat	J3	30/10- 7/11	-	-	-	3.32	2.70	820	294	* Only 3 weekday interpretable due to power cuts

Note: p.e. = population equivalent

(Source: Hawksley, 1985)

Table 2.3 Night Flows

Location/P. Station	Monitoring' Method	Average Daily Flow ADF (l/sec)	Night Flow (l/sec)	Night flow / ADF
Old Gufool: A1C	Ammeter	2.12	0.74	0.35
Kanoo Gardens: A1D + A1E areas	Ammeter	2.84	1.19	0.42
A1D area only	Manual/ammeter	0.72^	0.15^	0.21
S. Burhamah: A1E	Ammeter	2.65	1.51	0.57
S. Manama: A5C	Ammeter	1.51	0.89	0.59
Hoora: A7C	Ammeter	8.83	3.62	0.41
E. Sanabis: A17D	Ammeter	3.00	0.72	0.24
E. Muharraq: B6	Ammeter	16.24	10.25	0.63
Isa Town: D7 & D6	Hours run	1.51	0.36	0.24
Isa Town: 1-5	Hours run	39.80	14.40^	0.36
E. Riffa: F2	Ammeter	1.24	0.29	0.23
E. Riffa STW	Hours run	3.09	0.80	0.26
Nuwaydirat: J1 etc	Ammeter	9.67	2.71	0.28
J3	Ammeter	2.79	0.69	0.25

Note: Figures marked ^ are based on a 24 hour survey only.
All other figures are based on 7 day results.
' For determining night flow.

Source: Hawksley, 1985

Therefore the actual night flow could be considered to be only in such cases as c, d and e.

Currently, when designing town sewers, the average flow per person, including an allowance for abuse is 228l/d. This figure includes all the flows into the sewers excluding infiltration. Where the ground water level is lower than the sewer level, there will be no infiltration. Hence for the design of a sewerage network, the consideration of infiltration should be according to the highest expected ground water level in relation to the sewer pipe.

2.1.2 PEAKING FACTORS AND PEAKING FLOW (PDF)

The design peaking factors are based on international practices and they also take into account recorded data from the sewer lines in different places and at different periods in Bahrain (Table 2.1). Therefore the Ministry of Works, Power and Water (MWPW) recommends peaking factors to be applied to sewage flow before the addition of the infiltration. Using (Table 2.1) as a guideline the following formula was found to "curve fit" the values given:-

"Design peaking factor = $((1.5756 \times 10^7)/P)^{0.158}$ 2.1

where P is the design population". (Hawksley, 1985)

This formula is used only for a population above 300, so for a population below 300, a fixed pipe diameter of (150mm) and a minimum sewer gradient were considered. (Table 2.4).

MWPW has introduced a formula which is based on the recorded daily flow from different areas.

The following formula has been developed for a population between 300 to 200,000 (exclusive of infiltration).

"Daily peaking factor = $((3.75 \times 10^7)/P)^{0.078}$ 2.2

P is the Design Population.

"For population above 200,000 a daily Peaking Factor of 1.5 and a Design Peaking Factor of 2.0 have been assumed". (Hawksley, 1985).

2.2 HYDRAULIC DESIGN

2.2.1 PIPE ROUGHNESS

The hydraulic and the roughness coefficient designs in Bahrain are based upon those of the Hydraulic Research Station (England), which are derived mainly from the Colebrook-White equation. (Appendix B).

2.2.2 MINIMUM GRADIENT

Bahrain Island is very flat. A minimum slope is often used to minimise excavation but sufficient to avoid the blockage of sewer lines, and to achieve the minimum self cleaning velocity of 0.69m/s recommended by the MWPW. The minimum pipe diameter of 150mm is used for gravity main sewers and a minimum diameter of 100mm for pressure mains. For minimum gravity sewer gradient and recommended design parameter (Tables, 2.4 & 2.5).

In warm areas, hydrogen sulphide will often develop as waste water is transported in a sewer which is laid along flat slopes. This waste will often cause blockage of sewer lines and hence more generation of hydrogen sulphide. (Section 2.3).

Also flat land needs more pumping stations to lift the sewage up, so that it can be transported to the treatment plant.

Table 2.4 Recommended Design Parameter (MWPW)

For populations greater than 300:

Hydraulic Theory and Roughness Coefficients	Colebrook-White/Hydraulics Research Station
Minimum pipeline sizes:	150mm dia. gravity, 100mm dia. pressure
Minimum velocities:	0.69m/sec at normal peak daily flow. Full flow velocity 0.9m/s (gravity). Pressure mains 1.2m/s
Design flows:	Design peak flows obtained by applying peaking flows to average flows, adding allowances for infiltration where appropriate.

(Source: Hawksley, 1985)

Table 2.5 Recommended Design Parameters (MWPW)

For populations less than 300, for which the peaking factor is unreliable and inappropriate, the following criteria for gradients were adopted for 150mm lateral sewers:

Number of Housing Units	Minimum Lateral Sewer Gradient
1 - 10	1 in 60
10 - 30	1 in 80
30 - 60	1 in 100

(Source: Hawksley, 1985)

2.3 PIPE MATERIALS

In Bahrain a flat sewerage network is constructed. Therefore hydrogen sulphide generation is high, due to the delay of the sewage in the system. This has meant that in Bahrain, close attention was given to the type of pipe materials chosen and their degree of corrosion resistance.

"The most important chemical characteristics of a pipe material are resistance to dissolution in water and corrosion. The pipe surface must be able to withstand both electrochemical reactions from the surrounding soil and waste water conveyed in the pipe. Figure 2.1 illustrates the process of crown corrosion in sanitary sewers. Bacterial activity in anaerobic waste water produces hydrogen sulphide gas, particularly in warm climates when sewers are laid on flat ground. Hydrogen sulphide absorbed in the water condensed on the crown is converted to sulphuric acid by aerobic bacteria action. If the pipe is not chemically resistant, then acid deteriorates it and eventually results in the collapse of the crown."

(Source:Hammer, 1986).

Vitrified clay, GRP and uPVC pipes are used since they have high resistance to hydrogen sulphide corrosion.

Vitrified clay pipes are mainly used for the main sewer lines and also for lateral connections to the inspection chambers. The sizes of pipes which have been used, range from 150mm to 250mm in diameter. These pipes are usually imported. (For example from U.K.).

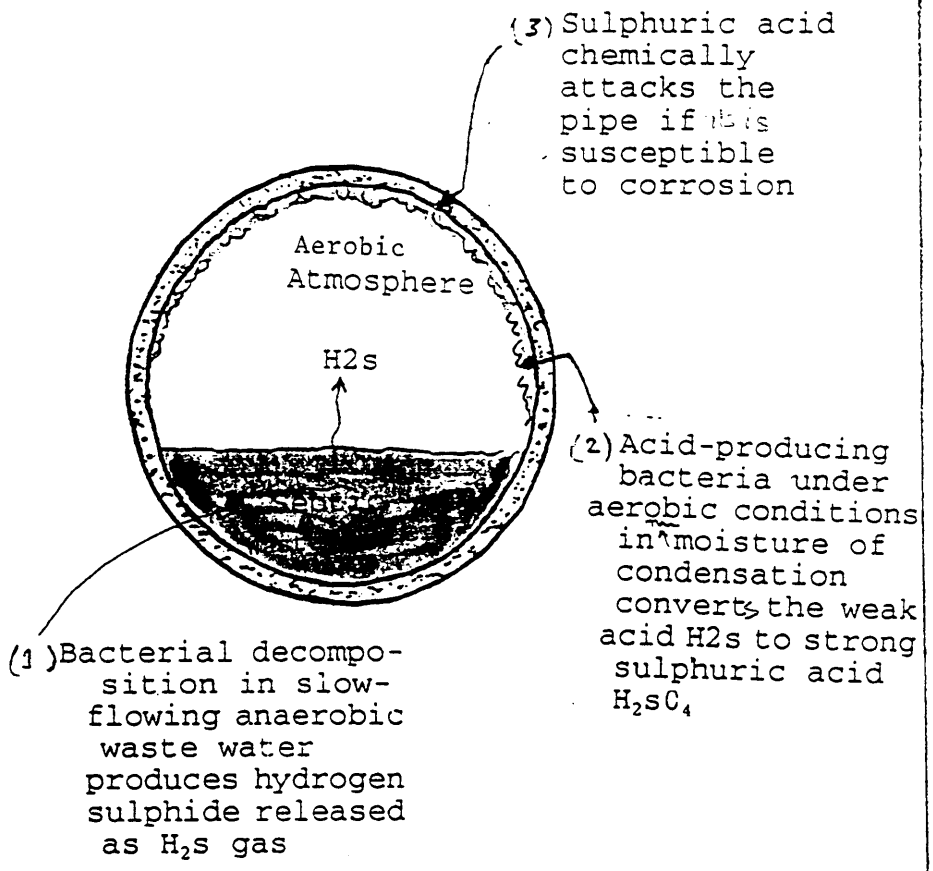


Figure 2.1 Environmental condition leading to crown corrosion occur in sanitary sewer as a result of flat grades, warm climates and an appropriate material.

GRP (Glass Reinforced Plastic) pipes are used for both gravity sewers and for pressure sewers. The GRP pipes are manufactured locally, which makes them very competitive with clay ware pipes of 300mm diameter and above. Recently GRP pipes have been used for pipe-work in lift stations and in pressure stations.

UPVC pipes have become much more widely used for plumbing work inside a house. The reason for this is that they can be easily transported, installed and cut to the required size; these pipes are also manufactured locally.

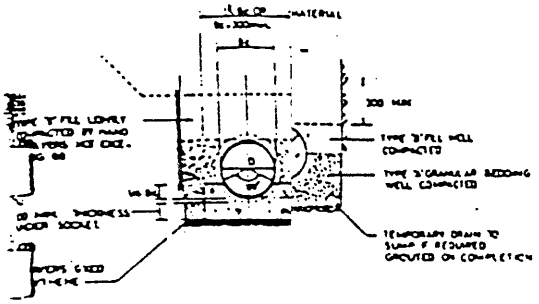
2.4 STRUCTURAL DESIGN

2.4.1 INTRODUCTION

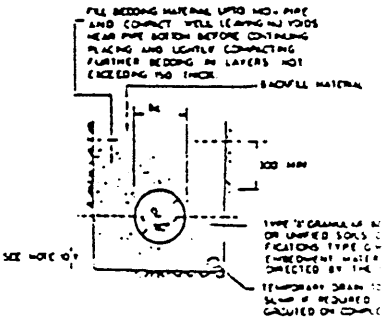
The Marston theory is used for the structure design of waste water pipe lines. For pipe bedding and joint details (Figure 2.2).

This has proved to be adequate with no cases of failure for rigid pipes, even where inadequate pipe bedding has been provided. However, additional thought should be given to joints outside manholes, because failure of clay ware pipes has been observed during CCTV surveys. This damage has been due to inefficient compaction of ground around the manholes during construction.

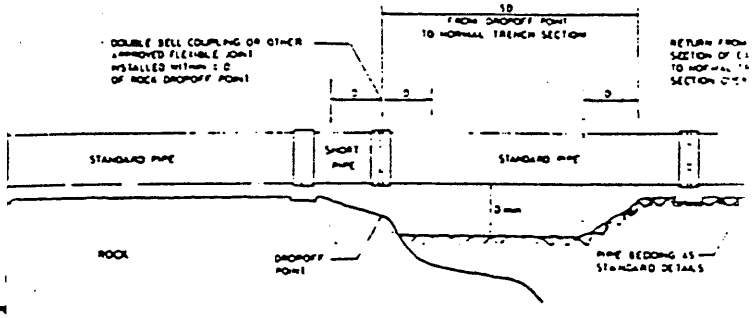
Problems have been experienced with double flexible joints of GRP pipe line outside manholes. This is caused by short lengths of pipe between two joints. However this problem has been overcome by having a transition zone at the edge



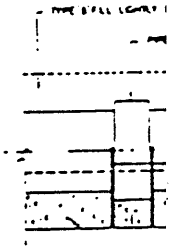
CLASS 'A' BEDDING



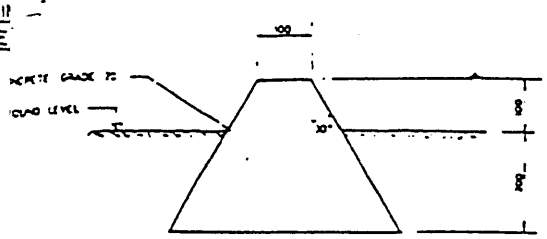
CLASS 'S' BEDDING

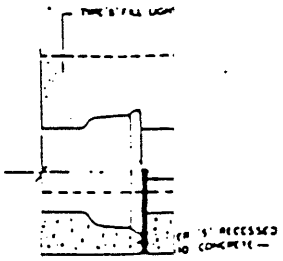


PIPE BEDDING DETAIL AT
MAJOR CHANGE IN BEARING CAPACITY OF FORMATION

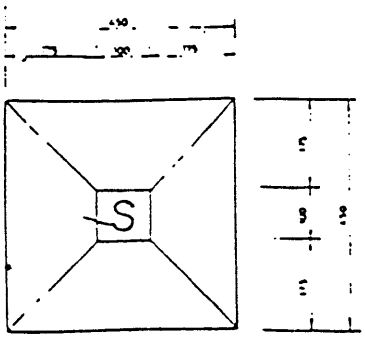


DETAIL AT COLLAR
CRADLE





DETAIL AT SPIGOT
CONCRETE



ELEVATION

PLAN

SEWER STUB SURFACE MARKER

Figure 2.2

Bedding and joint detail

of the structure, with reoprene rubber around the pipe.

2.4.2 DEPTH OF SEWERS

Knowledge of the soil and site conditions is very important in order to determine the depth of the sewer line.

Depth of sewers depends upon the type of pipe material and its crushing strength, bedding and materials, back filling material, width and depth of the trench and finally the type of loading on the pipe lines. MWPW recommends that the depth from the finished ground surface to the top of the sewer pipe must be in the range of 1.0m minimum to 5.0m maximum. An experimental approach has been taken in one of the non-MWPW village schemes, with a maximum excavation depth of 3.0 metres. The result was the construction of four more pumping stations, which increased operation and maintenance requirements. These pumping stations generally lift the sewage by only 1m to 2m.

2.4.3 MAIN SEWERAGE

In the past in some towns in Bahrain the main sewerage networks were not built of appropriate pipe material. This caused deterioration of the pipe material although it had been designed for adequate capacity. As a result, the rate of corrosion has accelerated causing a rise in siltation and deposition of solids in the pipe line. This leads to blockage and the rise in septicity conditions near the head of the main sewerage systems.

Deterioration had reached a point where a major renewal or rebuilding of the main sewerage line was required. After renewal of the deteriorated parts, the flow in the main sewerage systems was much improved and no further blockage was recorded. Some blockages were recorded only in certain situations as where the lateral sewers entered the main sewer from individual properties.

2.4.4 TRUNK SEWERS

The trunk sewers operating in Bahrain are used to convey main sewer flows from different towns to the treatment plant. (Figure 1.4). All trunk sewer pipes are made from GRP. There were some problems with sewage coming from the old MWPW drainage systems. This was due to high septicity which had caused minor corrosion problems in the new trunk sewer.

An inspection in 1983 was carried out as part of a study by CIRIA (UK).

They found that no significant deterioration had occurred in the trunk sewer length, but that there was sliming approximately 5mm deep on the pipe wall below the normal flow line. Sliming depths were similar in all lines according differing flow velocities.

2.5 ACCESS CHAMBERS

2.5.1 HOUSE CONNECTIONS

Existing house sanitation where there are no sewerage systems, commonly consist of septic tanks located in adjacent access roads. These septic tanks retain solid matter

and allow liquid to overflow to the ground. When a sewerage system is built in an area, all the houses are connected to the main sewerage system through inspection chambers.

The inspection chambers are usually built in the footpath outside the property so that it has easy access for maintenance.

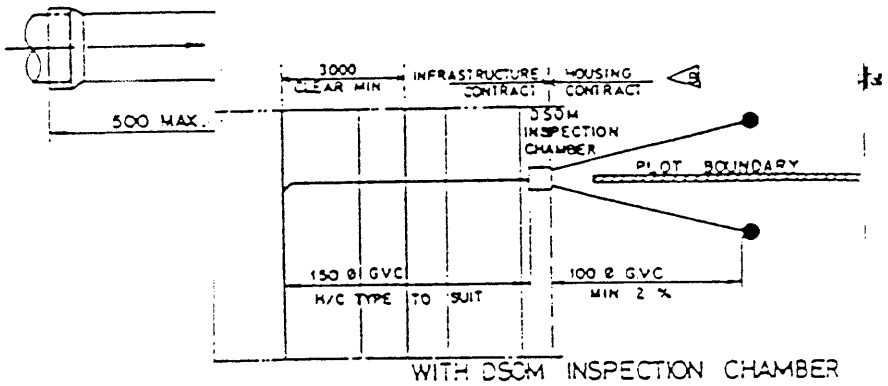
In some old sewerage schemes in Bahrain, the inspection chambers were built from rendered block work. These inspection chambers did not have protection against sulphide, which led to cracking, spalling of walls and blockage of channels in the chamber. MWPW had changed concrete benching to clay ware channels. This step has made a significant improvement in inspection chamber flow characteristics.

Ministry of Housing (MOH) and Ministry of Work, Power and Water had recommended a concrete circular shape with different sizes of chambers according to their depth (Figure 2.4).

For different depths of inspection chambers different types of connections had been recommended by Ministry of housing (MOH) and MWPW (Figure 2.3).

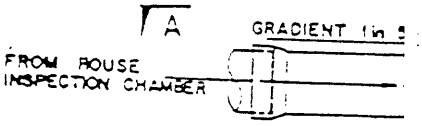
2.5.2 MANHOLES

In some of the old sewerage schemes in Bahrain, manholes were constructed from



(-FROM A-A WITH CON

FINIS
PAVE



45'

DIMENSIONS OF CONCRETE SURROU VERTICAL PIPEWORK.		
DIA. OF PIPE	MAX. PERMISSI-BLE WIDTH	MIN
150	800	
200	1000	

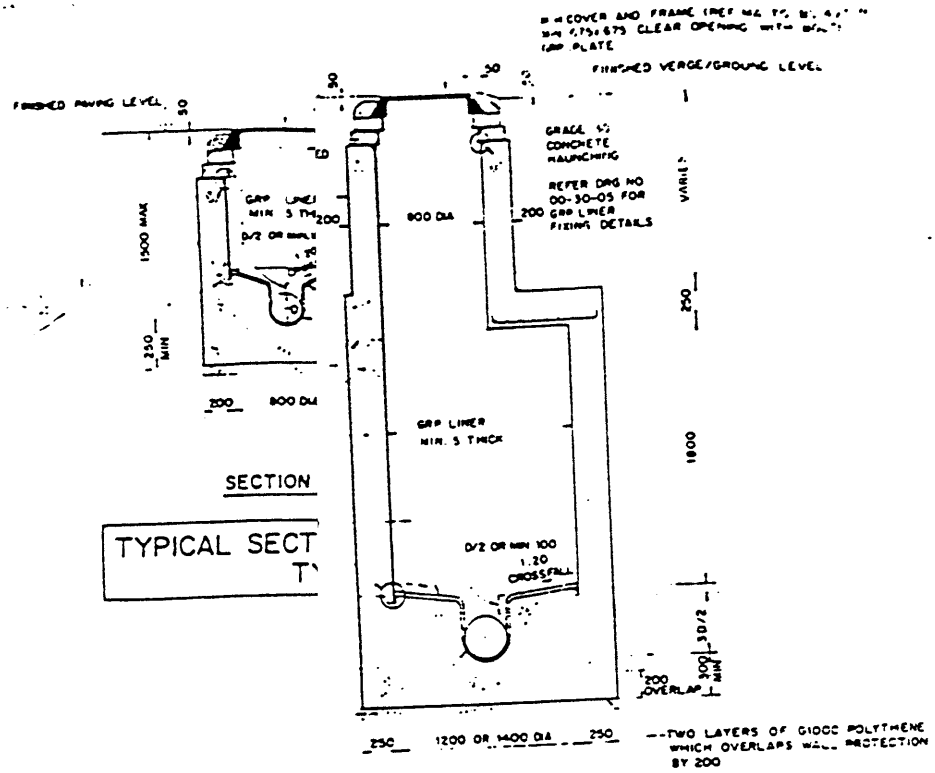
ELEVA

(FROM B-B WITH CONCRETE LOW CONNECTION (S)

TYPICAL DETAIL

Figure 2.3

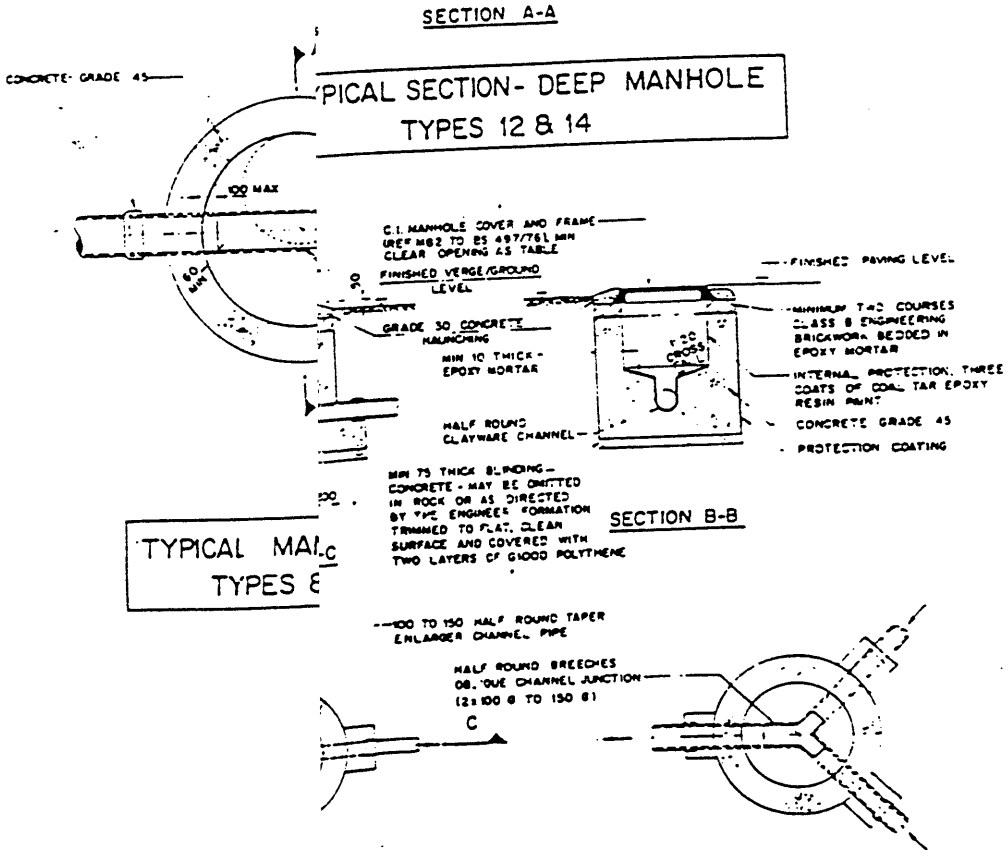
Sewer lateral and house connection details



SECTION

TYPICAL SECT
T

SECTION A-A



TYPICAL MA
TYPES 8

SECTION B-B

SCHEDULE OF MANHO				
MANHOLE TYPE	(1) MANHOLE CHAMBER		(2) MANHOLE S	
	INTERNAL DIAMETER	MIN WALL THICKNESS	INTERNAL DIAMETER	MIN THIC
8	800	200	-	
12 S	1200	250	-	
12 D	1200	250	800	
14 S	1400	250	-	
14 D	1400	250	800	

NOTE: IF CONDITIONS REQUIRE THE FIGURI WITH THE ENGINEERS AGREEMENT.

* MANHOLE DEPTH IS MEASURED A BETWEEN COVER LEVEL AND MAIN

(SINGLE ENTRY)

PLAN (TWO ENTRIES)

INSPECTION CHAMBER DETAILS

SCHEDULE OF INSPECTION CHAMBER TYPES					
	INTERNAL DIA.	MIN. WALL THICKNESS	MIN. M.H. CLEAR OPENING	MIN. * DEPTH	MAX. * DEPTH
	900	150	500 DIA	500	750
	600	200	600 DIA OR 600 x 450	750	1000
	750	200	600 DIA OR 600 x 450	1000	1200

ALL DEPTHS MEASURED FROM COVER LEVEL TO MAIN INVERT LEVEL

Figure 2.4 Manhole and inspection chambers

rendered block work. These manholes did not have protection against sulphide and this led to cracking especially in the walls of the manholes and also blockage of the channels. MWPW changed the concrete benching to clay ware channels. This step has made a significant improvement in manhole flow characteristics. Therefore, for any newly constructed sewerage schemes MWPW has recommended that all manholes should be of a circular shape and constructed from concrete. Their size to be in the range of 1200 to 1400mm internal diameter, with a reducing slab of 800mm internal diameter used as the access shaft for deep manholes. (Figure 2.4).

In addition MWPW introduced the application of an epoxy mortar skin to the benching; no noticeable corrosion has been recorded since, but some failures occurred. These were found to be due to poor surface preparation and only a thin layer of epoxy having been applied. All the walls and soffits of the sewerage manholes have been protected from corrosion either by the application of local tar epoxy paint systems or by using a GRP liner.

2.6 PUMPING STATIONS

2.6.1 DISTRICT PUMPING

The circular type of station with a GRP liner has generally been used for pumping lift and rising main stations (Figure 1.4, 2.4). The structural details have varied according to the designer.

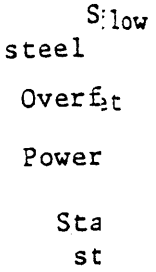
A mass concrete wall structure was proposed by the MWPW in 1979. Some circular

stations have been constructed with reinforced concrete walls, while others are constructed with a structural GRP liner without a concrete surround. Before 1980, cast iron pipes were used and corrosion was the main problem due to high septicity levels in the sump.

MWPW has tried to solve these problems by using heat shrinkable corrosion protection sleeves. As well as the problems of cast iron pipe corrosion, there have been several instances of the loosening of pump fixing bolts on their nuts, allowing movement of the pump as well as damage to the pipe. (Figure 2.5). These problems were solved by using GRP pipe work and also a number of standard lengths for the pipe work were specified by MWPW which made replacement of the pipes much easier.

To assist the progress of maintenance, there should be provision of an extra single manhole inside the pumping station into which all incoming sewers are collected, so that there is only one inlet to the sump itself.

This has simplified the problem of over pumping sewage in the case of a major failure in the pumping station. With regard to safety factors MWPW recommended that the pumps and the associated control together with the switchgear operate at a maximum of ten starts per hour, although the equipment itself is specified to operate up to 15 starts per hour.



holding the pump

Holding bolt

Figure 2.5

District pumping station

wer & Water, 1990

The majority of control systems installed in district pumping stations are based on the air bell principle and are backed up by stainless steel electrodes in case of failure. Adjustment of the air bell controls has been too accessible allowing incorrect setting of the controls and as a result, incorrect operation of the pump.

This could be overcome by covering each control adjustment knob with a lockable tough plastic material door to be opened only by a suitably qualified operator.

Ultrasonic sensors are used instead of air bells in some of the new pumping stations on a trial basis.

District pumping stations are usually small stations with a peak flow of not more than 300 l/s (as maximum flow).

Two pumps are usually installed, each one is able to meet the maximum inflow and they operate alternately. District pumping stations are usually operated on a main sewer line.

2.6.2 MAIN PUMPING STATIONS

Main pumping stations are defined as stations operating on trunk sewer lines.

They can be split into:

1. Lift stations using submersible pumps to lift the sewage into a collector channel

which discharges the sewage into the next trunk sewer.

2. Rising pressure main stations consisting of a wet well/dry well with variable speed pumps.

Both types of stations require a sufficient boundary of land. They are equipped with a superstructure to house control, switchgear and accommodation for personnel on full-time attendance.

Main pumping station sumps have been constructed in reinforced concrete and are of rectangular shape.

The lift stations have either one well or two for submersible pumps which discharge to a central channel which feeds to the head of the next trunk sewer.

These wells are usually lined with PVC and perform satisfactorily provided that they have been installed with care, so that there is no possibility of later damage to the concrete beneath them. Flap valves have been installed at the outlet to each pump discharge within the central channels. Plastic flaps were used but it was found that there was too much turbulence in the channel, which caused the flaps to break off. These were replaced by cast iron flaps. Turbulence would be reduced by alternating the discharge from each pump. Another improvement could be made by turning the discharge slightly into the direction of the flow within the central channels.

All pressure main pumping stations have both wet wells/dry wells. These wells are lined with PVC. All of the pipe work in the dry well is of stainless steel.

Although the wells are completely closed there is an odour problem at the main pumping stations. These problems will be a more serious nuisance for the population if the main pumping stations are located in a residential area or even close to it.

2.6.2.1 GRIT HANDLING

Grit handling in the sewerage system varies from large pumping station to small pumping stations. More grit problems in sewerage systems in Bahrain mainly appear in the low lying areas which are Manama and Muharraq.

The amount of grit removal depends upon two factors, which are:

a) The stations location within the sewerage system.

b) The degree of development in the catchment area.

c) The height of the water table in respect to the sewer line and hence the amount of infiltration entering the sewer.

d) Surface water (presence of sand in the road) because there are a lot of roads which do not have any side footpath and hence allow a lot of sand to enter the road and from there into the sewerage system. All the incoming sewers from the different parts of the catchment area towards the pumping station or life station appear clear of silt and run freely with adequate velocity at the pumping station the sewage will be

steady for some time until the pump starts. This will cause some of the silt or grit to settle down and become trapped in the sump and the other will be pumped onwards downstream until it reaches the treatment plant. The remaining grit or silt is removed by suction tanker and manually (by sending a person down the sump). (The process will take approximately one to two hours). [MWPW].

Since in Bahrain there are no degritting wells, then all the grit or silt will be trapped in the pumping sump itself, which will reduce pump effectiveness by occupying capacity and creating blockage and hence causing the pumps to stop completely. Therefore frequent visits to the station are needed. These visits are scheduled according to their grit or silt accumulation (Table 2.6).

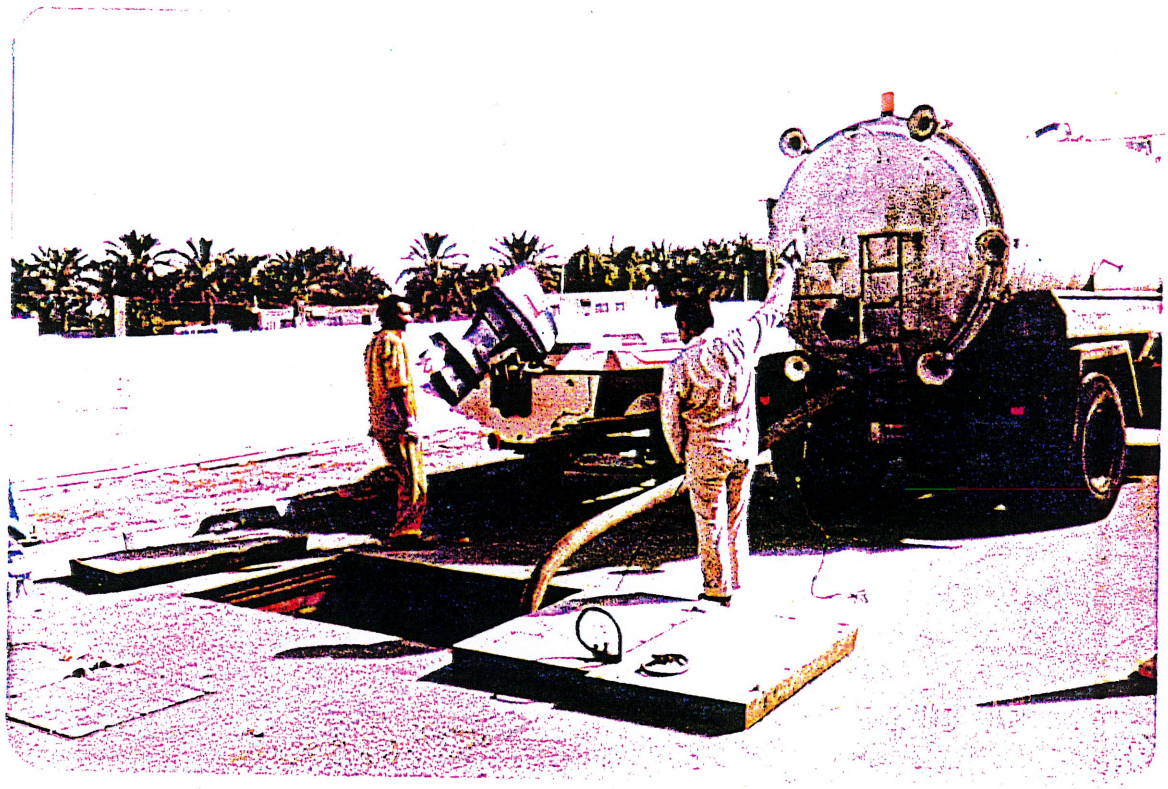
However, in Bahrain there are large number of pumping stations which need numbers of suction tankers and numbers of labour, due to lack of labour and also machinery with respect to the numbers of pumping stations (usually eight tankers working from fourteen tankers). This will cause delay to the desilting process from the sump and hence will increase the amount of silt in the sump (photo 2.7) before it can be removed.

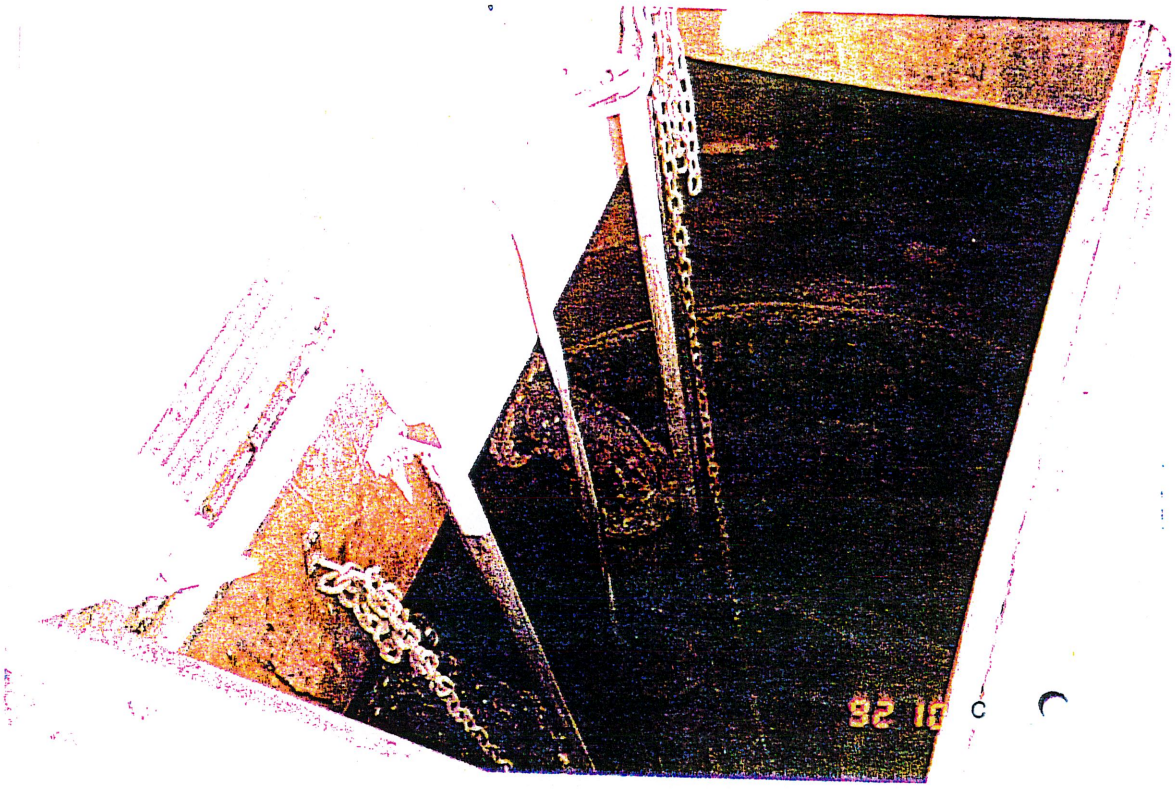
Therefore an improvement is proposed by the author in section (3.4.3.1).

Table - Desilting Operations in Pumping Stations (Jan 1991-Jan 1992)

Station	M^3/day	Visits each Two months M^3	Total Visits	Total Grit Removed m^3/year
A1	60.00	4.0	6	2.4
A1F	356.55	0.5	6	3.0
A7	5237.08	0.7	6	4.2
E2	4033.33	1.5	6	9.0

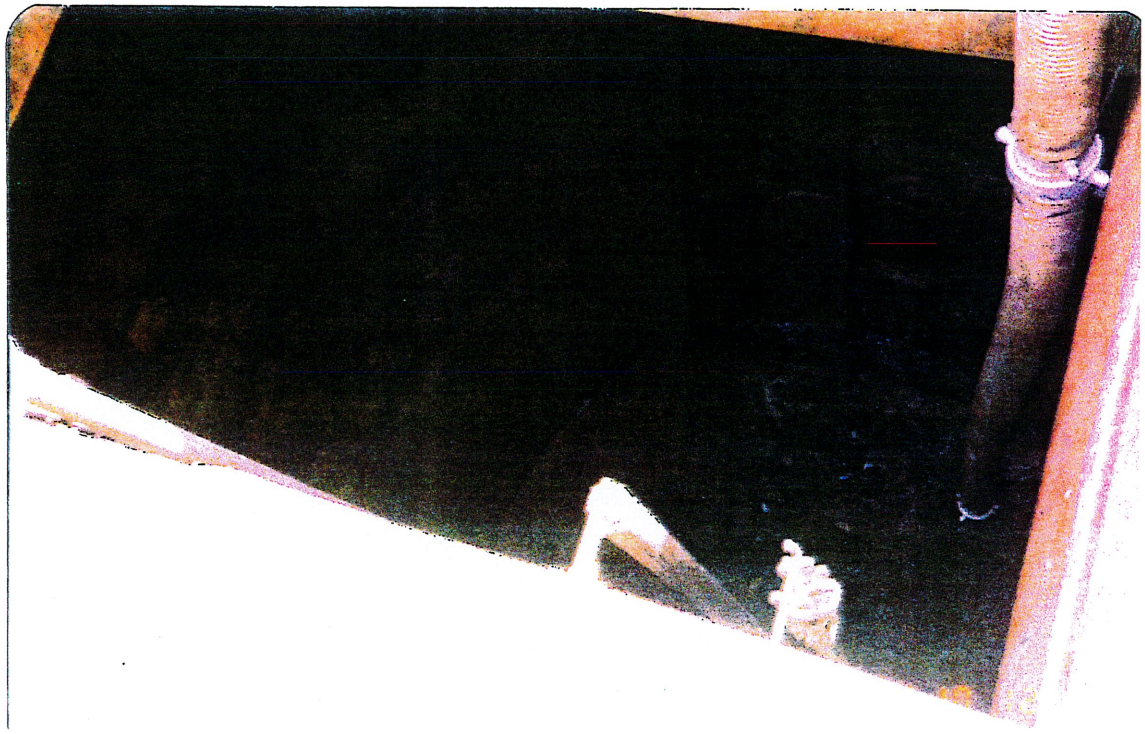
MWPW 1992





Grithanding from pumping station sump

FIG 2.6





Grithanding from pumping station sump

FIG 2.7

2.6.2.2 ODOUR CONTROL

Generation of hydrogen sulphide in pumping stations is at present due to wet storage, the retention time in rising mains and the existing high temperature situation.

Proper design and a regular programme of odour control systems should be followed, so that the concentration of odourant is minimised, or the septicity level kept to the lowest level which will not be harmful to nearby residents. The problem can be solved by preventing anaerobic process in waste water and this depends upon the amount of dissolved oxygen present in the flow (waste water).

As building development reaches the perimeter of the existing main pumping station, it will be necessary to minimise odour problems. These will cause nuisance smells and toxicity gases.

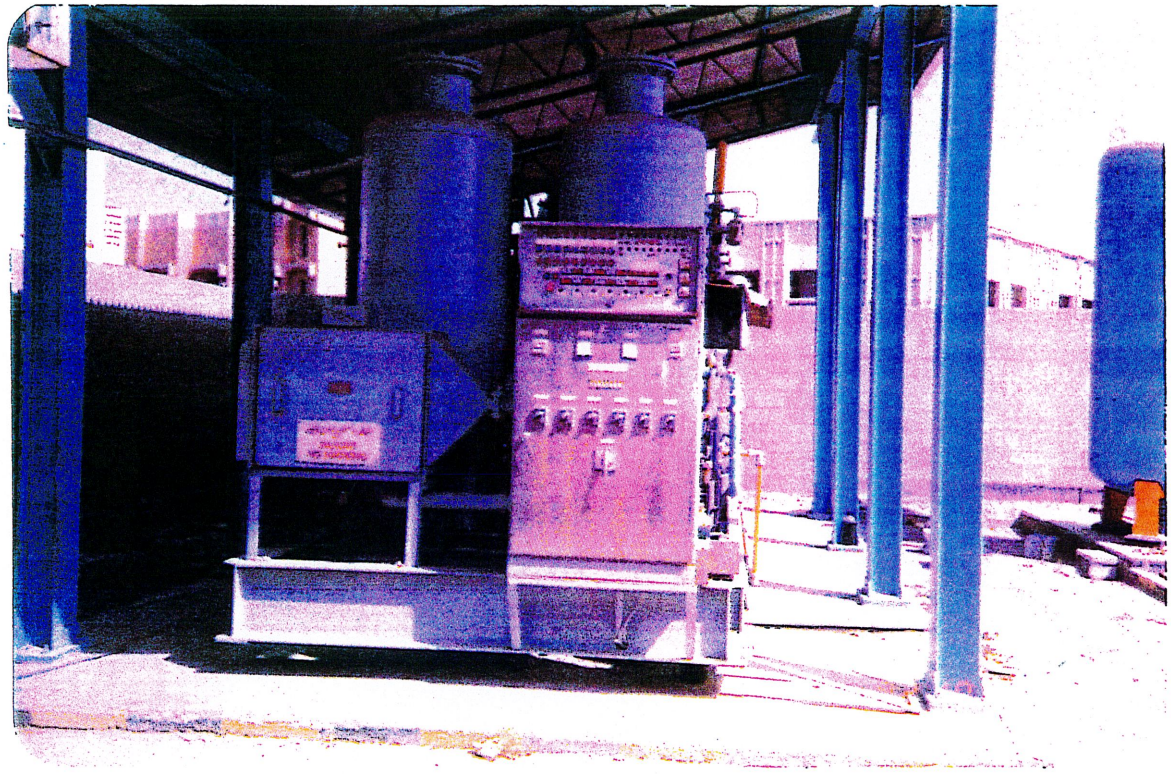
Three different methods have been used by the MWPW to control odour in pumping stations, which are as follows:

A) Oxygen injection.

Two oxygen injectors were installed in two different pumping stations, (Fig 2.8), injection of pure oxygen into a rising main can prevent or significantly reduce further sulphide generation from occurring and allow biochemical oxidation of the existing dissolved sulphides. Also the sewerage system will be acting as a treatment works while the sewerage is moving to the treatment plant (self purification).

The disadvantages of oxygen injection are:

1) Very complicated equipment which needs an expert technician to operate it.





Oxygen injection equipment

FIG 28

2) Danger of explosion.

3) Is only practicable on long pressure mains (Because of the low solubility of oxygen in sewage).

4) High operation and maintenance requirement.

Due to these disadvantages the oxygen injection plants have been used only for very short periods.

B) Gas scrubbing plant (Fig 2.9)

The wet scrubbing gas absorbtion plant has been in service for approximately twelve years in Bahrain .(Fig 2.10)

The implementation of scrubbing plant is to reduce the nuisance (odour control) which is coming out from the sump.

Hydrogen sulphide is passed by air blown through the towers. A mixture of water (1000 PP) plus caustic soda is spread from the top of the tower. The Hydrogen sulphide which is coming out of the vent will be diluted.

Disadvantages for gas scrubbing plant

1) The tower always needs careful maintenance. (Blockage Problems).

2) The dosing pumping of scrubbing equipment is unreliable and subject to frequent delivery flow fluctuations.

3) The need for fresh water is 1000 PP minimum, therefore a reverse osmosis package plant was installed in the pumping when the scrubbing equipment was installed or water is supplied by tankers.

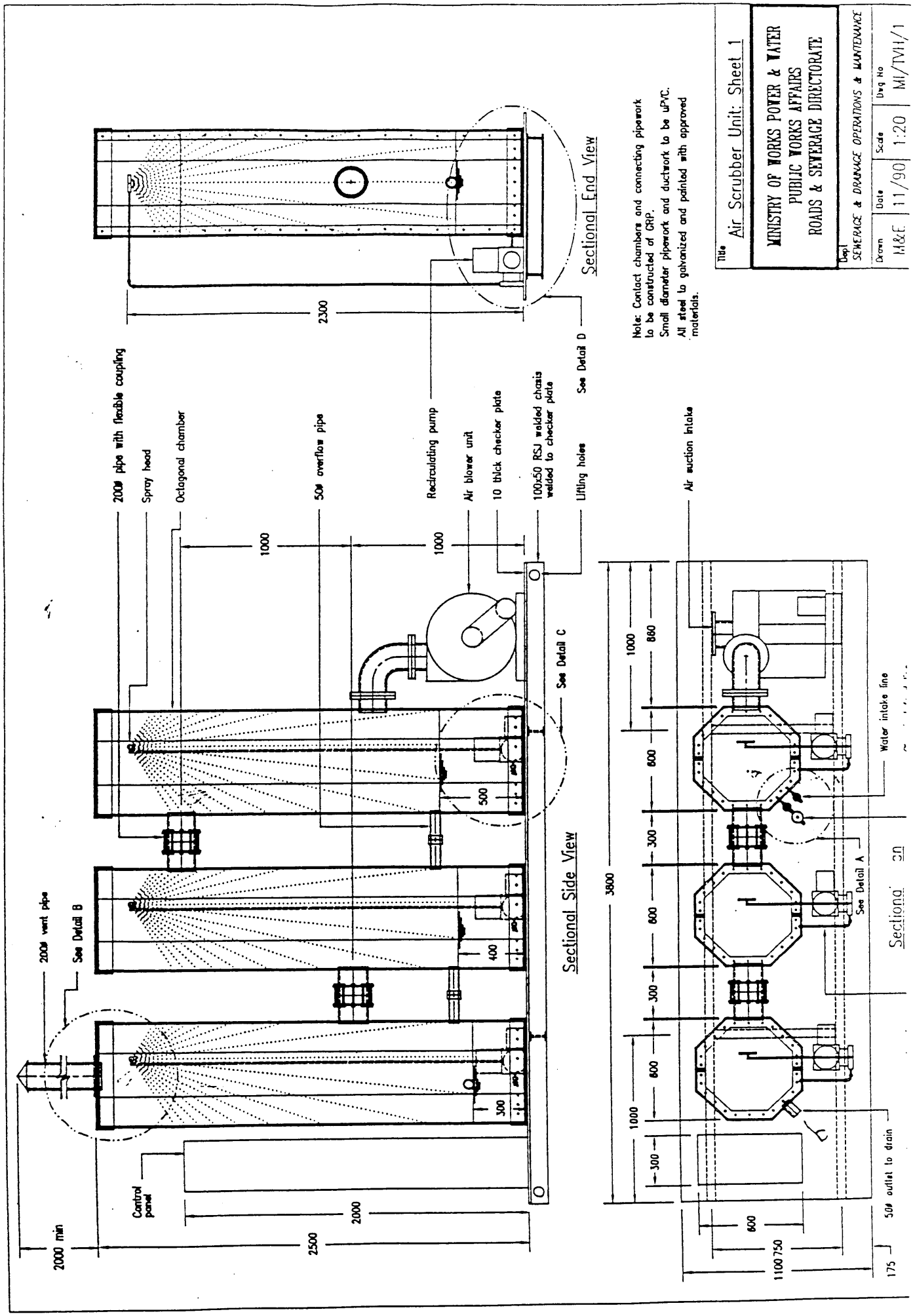

Sectional End View
2300
200ø pipe with flexible coupling
Spray head
Octagonal chamber
50ø overflow pipe
Recirculating pump
Air blower unit
10 thick checker plate
100x50 RSJ welded chais welded to checker plate
Lifting holes
See Detail D
Air suction intake
Note: Contact chambers and connecting pipework to be constructed of GRP.
Small diameter pipework and ductwork to be uPVC.
All steel to galvanized and painted with approved materials.
Title
Air Scrubber Unit: Sheet 1
MINISTRY OF WORKS POWER & WATER
PUBLIC WORKS AFFAIRS
ROADS & SEWERAGE DIRECTORATE
Dept
SEWERAGE & DRAINAGE OPERATIONS & MAINTENANCE
Drawn
M&E
Date
11/90
Scale
1:20
Dwg No
MI/TW1/1
1000
1000
500
400
300
2000 min
2500
2000
Control panel
200ø vent pipe
See Detail B
Sectional Side View
See Detail C
3800
1000
880
800
600
300
Water intake line
See Detail A
Sectional on
500 outlet to drain
600
1100
750
175

FIG 2.9

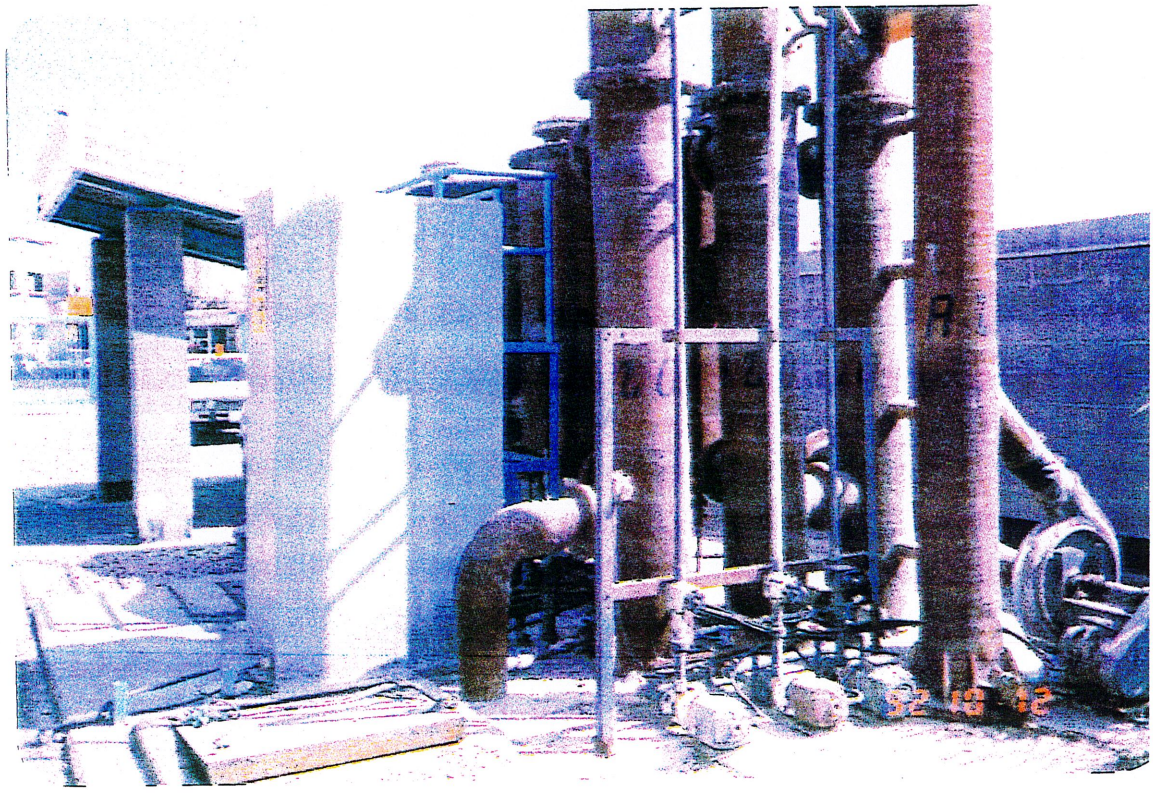




Scrubbing equipment

FIG 2.10





Scrubbing equipment

FIG. 2.10

4) Dependent on performance of the refurbished system and also environmental restrictions, there may be a requirement for secondary deodorisation.

5) Gas scrubbing chemical

The Chemical used is sodium hydroxide (caustic soda) in powder. This chemical is delivered to the site in bags and difficulties arise with regard to:

1) Handling and personal hazards.

2) Storage difficulties.

3) Mixing inconsistencies.

The aim of this scrubbing plant is that the final exhaust gas concentration should not be in excess of 2 ppcs to guarantee an odour free environment. (MWPW).

As has been shown in the Experimental Data (Table 2.7) shows that the scrubbing plant is not achieving the aims which it has been installed for. Hence the author believes that using a scrubbing plant is a waste of money and effort.

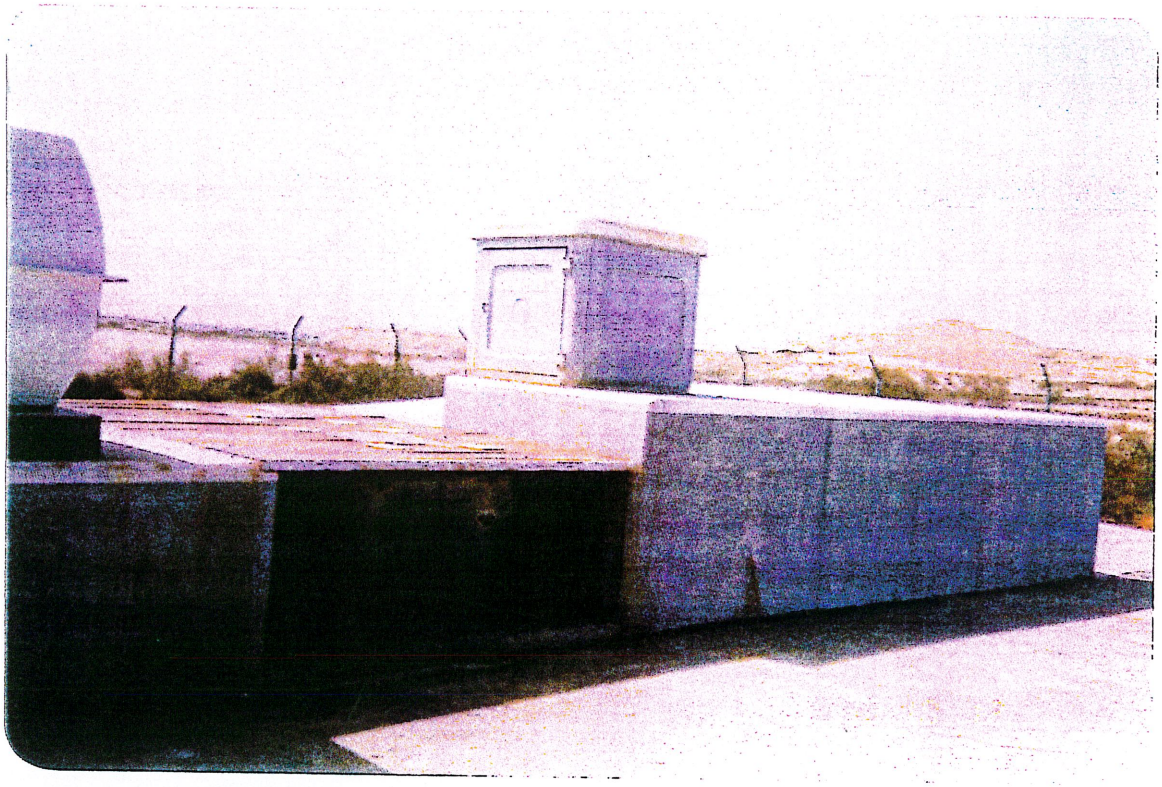
C) Chemical dosing (Fig 2.11)

Ferric iron solution is used as chemical dosing in small pumping stations to control odour problems.

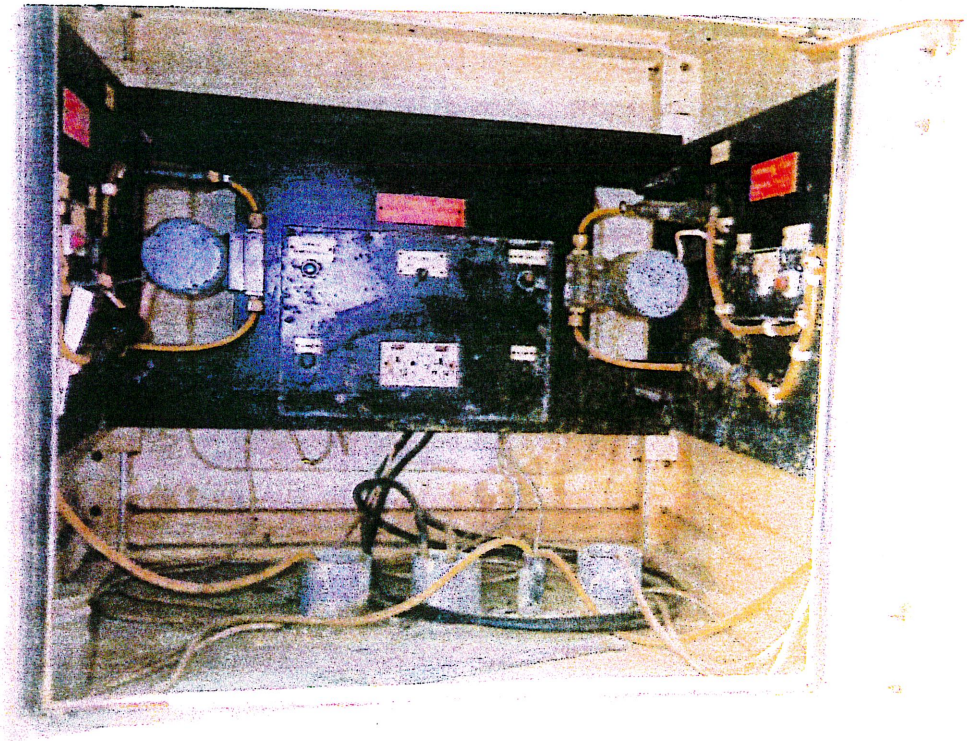
Lack of chemicals had caused difficulty in the dosing process because the chemical is usually imported from other countries and when it is finished they have to wait for another shipment. Also, lack of spares for dosing pumps has stopped the dosing process in other pumping stations. Such problems (Lack of materials and Breakdown) give a clear picture that chemical dosing is not suitable to be used for odour control. Hence, another method should be used to control odour and septicity in sewerage

systems. Therefore all three methods which have been mentioned are not satisfactory so are not suitable for Bahrain use.

This is an indication that a new method should be introduced which can solve both the odour and septicity problems. Also with lower O & M requirements. Therefore an improved method has been recommended by the author in section (3.4.3.2).



Chemical Dosing into rising main



chemical dosing into rising main

FIG 2.11

2.6.2.3 OVERFLOWS

Different overflow systems are used in large pumping stations in Bahrain. An overflow system is very important in any pumping station because it caters for any emergency states such as pump or power failure which will result in flooding in low lying areas.

Overflow is used only for large pumping stations where the flows are high and over-pumping is very difficult. An overflow is not required in small pumping stations. This is because the smaller flows tend to take a longer period to fill the sewage system itself, until flooding occurs. Also in the case of failure over-pumping could be arranged.

Three systems used in Bahrain are as follows:

A) Overflow using pipe connections from pumping stations to the outflow into the sea. (Fig 2.12). This type of outflow is only used in pumping stations near the sea or close to it. Hence all pumping stations in Manama and Muharraq are with this type of overflow.

The advantage of this system is that it does not require any open space its discharging directly into the sea when failure occurs or when pumps are not working in their full performances.

The disadvantage of this system is that it requires a right way in which the pipe can be laid. It needs frequent maintenance of the gate valves at the outfall. It also causes an environmental pollution when discharging waste water into the sea. This is dangerous to aquatic life.

LOG FOR MONITORING H2S GAS AT B1 PL

Date	17/6/90		
Time	09.30	14.30	(
Description			
H2H P.P.M. INLET	120	90	1
BOX NO	1/89	1/89	1.
H2S P.P.M. EXIST	50	30	.
BOX NO	1/89	1/89	2/
S.G. OF C/NaOH	–	–	.
P.H. OF C/NaOH	–	–	-
S.G. OF DOSED H2O	–	–	-
P.H. OF DOSED H2O	–	–	-
S.G. OF USED H2O	–	–	-
P.H. OF USED H2O	–	–	-
S.G. OF H2O TOWER-1	–	–	-
P.H. OF H2O TOWER-1	–	–	-
S.G. OF H2O TOWER-2	–	–	-
P.H. OF H2O TOWER-2	–	–	-
S.G. OF H2O TOWER-3	–	–	-
P.H. OF H2O TOWER-3	–	–	9.3
LEVEL NaOH TA/B	–	–	-
NaOH USED l/hr	–	–	-
TEMPERATURE °C	–	–	35
S.G. FRESH H2O	–	–	-
P.H. FRESH H2O	–	–	7.3
ROTAMETER l/hr			11(
DOSING PUMP %			30



Outlet of Overflow into the sea

FIG 2.12



Open Overflow reservoir with asphalt base



Open Overflow reservoir with hard rock

FIG 2.1

B) Open reservoir overflow (Fig 2.13)

This type of overflow is usually used beside the pumping station or inside the pumping station boundary. Two pumping stations have been using this type of overflow system.

The advantage of this type of system is that it does not need to be near the sea. It can be used as recharge for ground water if the pumping station is not located near residences. It does not need a reserve right way. There is no need for frequent maintenance.

The disadvantage of this system is that when it is located near the resident area it might create a nuisance and flies due to release of foul sewage into the open area where it might be left idle for a period of time. Also it needs a large area beside the pumping station. It needs to be cleaned after it has been used so that it will not be a place for flies to breed, because in the low areas in Bahrain water does not percolate very quickly.

Therefore an open overflow should be designed so that the sewage can be returned back to the pumping station after the failure has been amended or rectified in low lying areas. A fence should be built around the overflow reservoir to avoid any animals or humans entering the area.

C) Standby generators to cater to any overflow in pumping station.

This system is used where there are no other overflow facilities in the pumping station.

This type of overflow system is used in one place in Bahrain (Isa Town Pumping).

A large generator was installed in the pumping station on a standby basis. The advantage of this system is that it does not need a wide area or a reserved right of way.

The disadvantage of this system is that if there are any problems downstream in the sewerage system (such as broken pipes, blockage etc) such problems will lead to very big problems such as the flooding of low lying areas.

Also if a rare event such as a major penal failure occurred and the pumps were disqualified for a long period, flooding would occur.

Therefore, the author recommends some modifications to the existing systems and also proposes a new overflow system in section 3.4.3.3.

CHAPTER THREE

3.0 IMPROVEMENT TO THE EXISTING FOUL SEWERAGE SYSTEM

3.1 DESIGN PARAMETERS

3.1.1 AVERAGE DAILY FLOW (ADF)

It has been previously mentioned that there has been a slight increase in the average daily sewage flow from house connections from year 1974/75 to year 1984/85. (Table 1.1). This increase is about 3.5% which is smaller than the measured accuracy which is estimated to be approximately 10%. This means that the average daily sewage flow did not change in this ten year period.

Therefore 228 l/p/d plus infiltration (where applicable) can be established for use by all of the authorities responsible for the design of sewerage in Bahrain. Even if consideration is given for the offset of charging for water on a metered basis, this did not make a lot of difference, since the water consumption in Bahrain (table 3.1) is unlikely to change significantly as a result of water metering. Charging for water is more likely to result in a reduction of negligence (taps left running), wastage (due to body leaking in plumbing) and gardens.

3.1.2 PEAKING FACTORS AND PEAKING DAILY FLOW

It has been shown in (Fig 3.1) that the measured daily peaking factors are less than the assumed design peaking factors (Table 3.2 and 3.3). From the recorded data it is found that the safety factors range from 1.8 to 2.4. They depend upon the size of the area and thorough investigation and finally the size of the population. (Table 1.1).

Table 3.1 **Water Consumption After Metering**

With Metering	(l/p/d)	Without Metering (l/p/d)
Basic (cooking, washing)	175	175
Abuse (faulty fittings)	30	60
Gardening etc	100	127
Total measured at property:	260	317

Source: Hawksley, 1985

Table 3.2 **Summary of Flow Measurement Data**

Location	Population	Basic Flow + Some Abuse (ADF - I) l/p/d	Design Peaking Factor $\frac{(PF - I)}{(ADF - I)}$	Measured Peaking Factor $\frac{(PDF - I)}{(ADF - I)}$	Safety Factor
Stations 4 & 5 (Isa Town	12 450	234	3.1	1.7	1.82
Station A7C	2 500	240	4.0	1.01	1.99
Station A1C	840	189	4.7	2.18	2.16
Station A1E	550	238	5.1	2.19	2.33

Source: Hawksley, 1985

Table 3.3 Flow investigations undertaken at Isa town Pumping Station

Year	Population	Deaign Peaking Factor	Effective Design Peaking Factor	Measured Average Effect P.F.	Measured Average Minimum Flow Factor
		$\frac{PF - I}{(ADF - I)}$	$\frac{(PF)}{(ADF)}$	$\frac{(PDF)}{(ADF)}$	$\frac{(Min\ DF)}{(ADF)}$
1974	6953	3.4	3.0	1.73	0.48
1984	6780	3.4	3.0	1.67	0.43

where I is the allowance for Infiltration
PF = Peak Flow
and PDF = Peak Daily Flow

Source: Hawksley, 1985

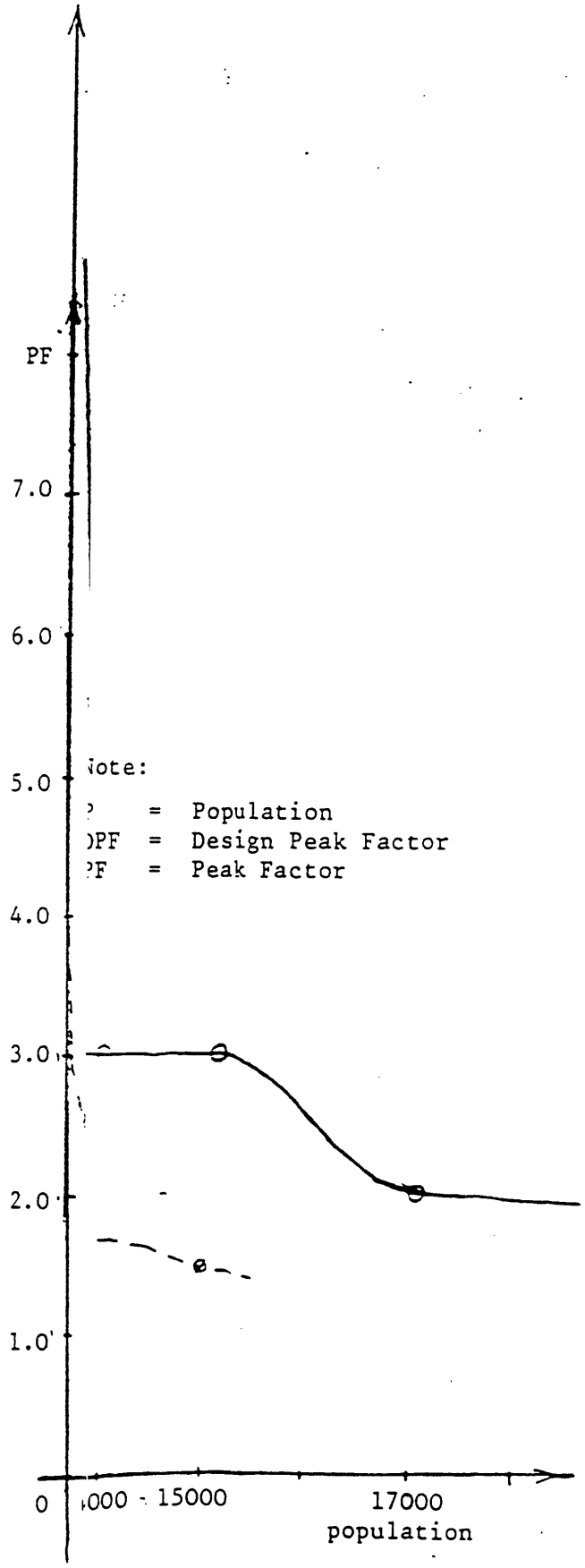

PF
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
Note:
= Population
DPF = Design Peak Factor
PF = Peak Factor
000 - 15000
17000
population

FIG 3.1

Since there were no significant changes in average daily factor flows in the 10 year period, it is preferable to retain the safety factors at their present level in order to guard against such uncertainties as pump performance, Table 3.1 abuse and rainfall etc.

There are some major items which could reduce the design peaking factors to daily peaking factors. They are as follows:

a) To ensure daily self cleaning velocities where only a few properties are connected to the system and there is a very flat sewer gradient.

b) To avoid unnecessarily long retention time and septicity build up in pumping stations and lift stations.

Any additional cost of adapting item (a) should be weighed very carefully against the potential benefits for operation and maintenance in the light of the evidence presently available.

Item (b) would appear to offer considerable advantages where duty pump(s) can be designed for daily peaks and standby pumps can provide the required factor of safety.

Obviously it is important to consider the initial and long term velocities and these should be checked according to the hydraulic design recommendation.

3.2 HYDRAULIC DESIGN

As has been mentioned in section 2.2.1 that Hydraulic research station (England) has been used for hydraulic design in Bahrain.

A more simplified method has been introduced by the author, which could be used in the design office. The idea is based on achieving design parameters such as:

a) Minimum velocities and gradients, which would be just sufficient to avoid silt deposition and the risk of blockages.

b) Avoiding the build-up of septicity in a highly corrosive concentration in the sewerage system.

Therefore a spreadsheet has been developed by the author for this purpose.

The formula which has been used is based on the Colebrook-White equation.

$$V = -2\sqrt{(2gDs)}\ \log\left[\frac{Ks}{3.7D} + \frac{2.5V}{D\sqrt{(2gDs)}}\right]$$

where

V = Velocity (m/s)

g = gravitational acceleration

s = Hydraulic gradient

v = Kinematic Viscosity of fluid

Ks = Roughness = 0.006 to 15mm (Hydraulic Research 1990)

The author developed this spreadsheet to assist in the designing of sewers. This allows the designer to check that minimum self cleaning velocities obtained at partial depths of flow at minimum sewer gradients (Figure 3.2).

Spreadsheet

"Foul sewer K_s 1.5mm" (Water Authorities Association)

Kinematic viscosity of fluid 1.143×10^{-6} m^2/s

Proportional Discharge

The partial filling of a pipe must be taken into account when designing sewers and the velocity and discharge volume will vary in relation to the depth of flow of the effluent. Using the parameter D (hydraulic diameter) and D (Velocity diameter), the Colebrook-White formula can be re-arranged as

$$Qp = -6.96\left[log10\left(\frac{7.42x10^{-7}}{DH\sqrt{(iDM)]}} + \frac{K}{3.71DM}\right) Dv^2\sqrt{(iDM)}\right]$$

Where

Qp = Discharge at partial flow (m^3/sec)

i = Pipe gradient (m/m)

K_2 = Roughness Value (m)

D_M = 4AW/PW

D = $\sqrt{4AW/\pi}$

and

where (Figure 3.3)

AW = Cross Sectional of flow

PW = Wetted perimeter(m)

R = Area/P

$AW = 3.14r^2(\phi/180)-(b/2)\ (r-h)$

where

$\phi = \text{Cos}\ [(r-h)/r]$

$b = 2\ [r^2-(r-h)^2]^{0.5}$

$Pw = 2(3.14)\ (r)\ (\phi/180)$"

(Bowker and Smith,1989)

Comparing these data from spreadsheet (Table B-1 appendix B) with Hydraulic Research (Table B-5 in appendix B). Using Velocity formula, it has been found that the flows and the velocities in the spreadsheet have very similar values to those Hydraulic Research table. Also comparing the data from the spreadsheet (Table B-3,4 and 6 appendix B) with the Bland design table (Table B-2 appendix B) using the partial flow formula. It has been found that the higher flow values with lower velocity values were obtained. This minor difference could be due to using different Kinematic Viscosity Values(m^2/s) than have been used in (Table B-2 appendix B), (Also velocity value changes according to temperature (Table 5 appendix B)

Therefore the spreadsheet could be satisfactorily used in the design office.

Here is an example of the spreadsheet.

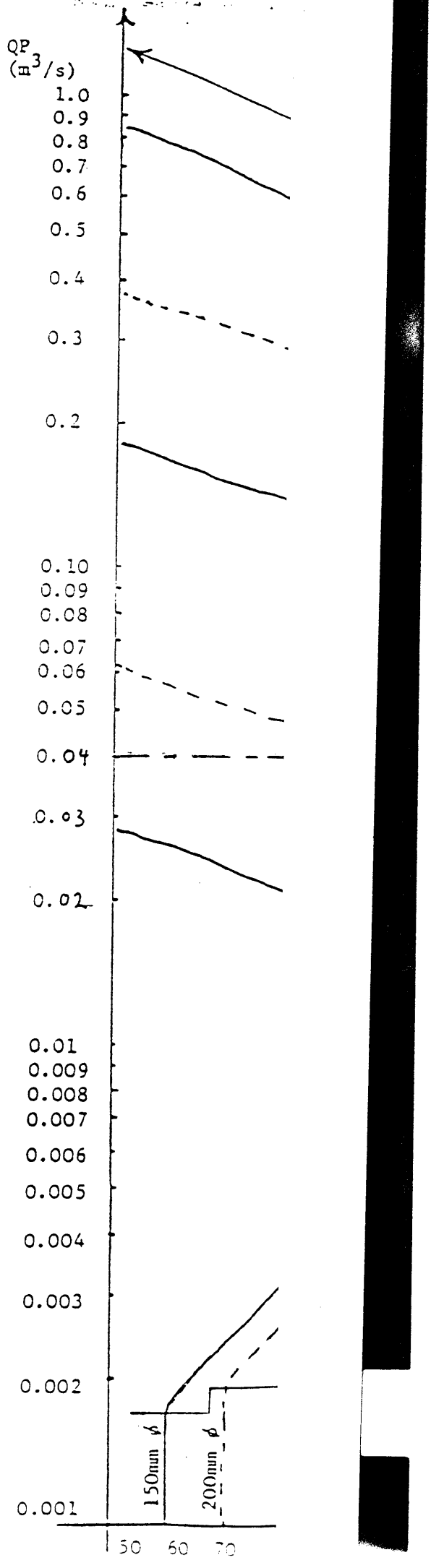
Usage of the graph in the Design Office

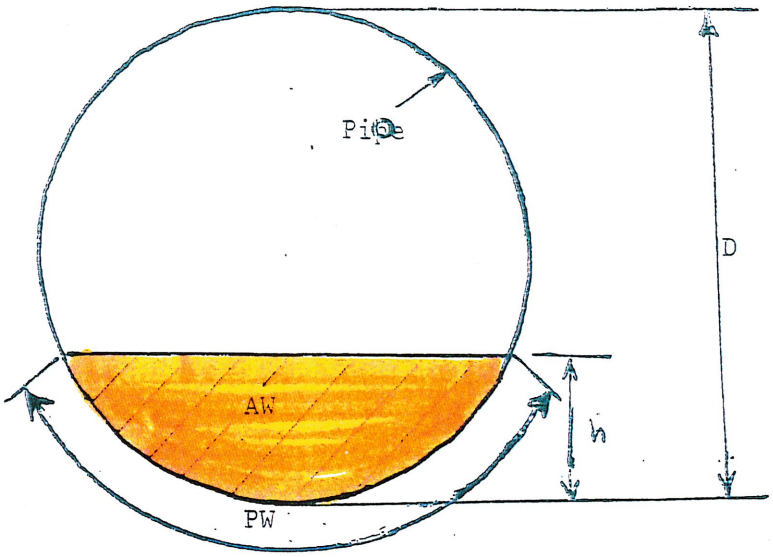
If the estimated flow is to be 40 l/s, then

1. For 200mm dia. is the maximum flow with 0.83% slope.
2. For 300mm dia, is the maximum flow with 0.25% slope.
3. For 400mm dia. is the maximum flow with 0.22% slope.
4. For 500mm dia. is the maximum flow with 0.20% slope.

This means that for a flow of 40 l/s, four different alternatives could be considered in designing a sewer line.

This could be an advantage to a design office when selecting a suitable pipe diameter with minimum slope that could suit their conditions.


QP
(m3/s)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.009
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
150mm φ
200mm φ
50
60
70



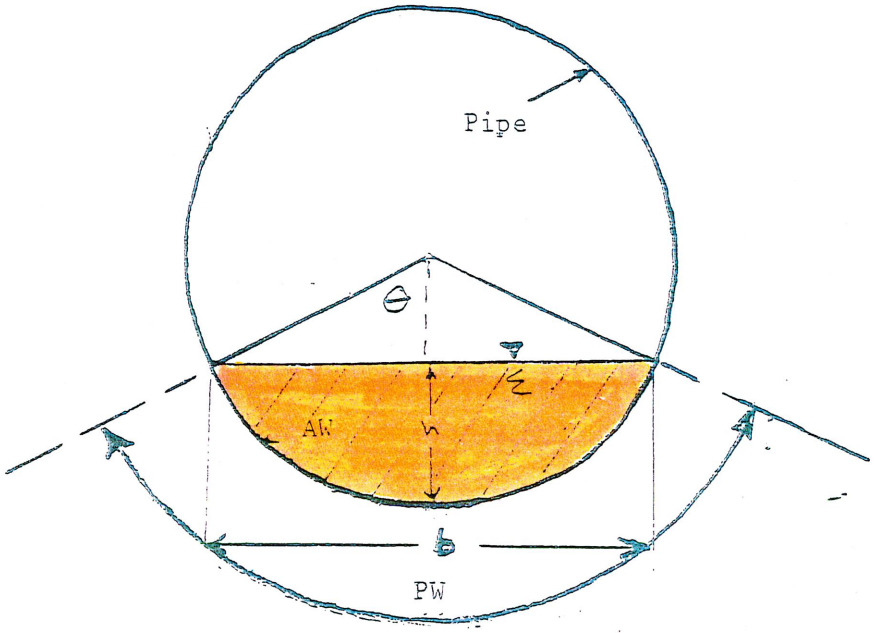


FIG 3-3

Any newly introduced sewerage system is liable to be subjected to various forms of wastage as a result of habits and the quantities of sand and silt entering the system.

It is necessary to consider both short term and long term conditions for hydraulic designs for main sewers and branch sewers. In the short term, the minimum allowable velocity ought to be achievable and in the long term, the velocity running full condition will permit a flatter gradient than if the short term condition is considered alone. Therefore, the result should be a compromise between the two conditions. This is usually dependent to some extent upon the judgement of the engineer who is designing the sewer.

Also it is important to know the ultimate and immediate contributing population, as well as the rate of population growth.

3.3 ACCESS CHAMBERS

3.3.1 HOUSE CONNECTION

As has been stated in Section 2.5 the access to the sewerage system from the house drain is provided through an inspection chamber which is located outside properties.

This is to enable the MWPW to carry out the maintenance work without entering the properties.

Inspection chambers are different from manholes. The chambers are designed to allow maintenance work to be carried out from the ground surface and also it is not necessary for man entry.

The existing inspection chambers have been used locally and they are found to be satisfactory. They do not require any changes. However an inspection chamber which is not located in the road could be constructed from PVC. An inspection point with one access could be used where the depth is shallow. The use of structural GRP might possibly be considered for an inspection chamber, but GRP is not necessary because these inspection chambers are more shallow than the manholes and there is less risk than in the main manholes and they are not designed for man entry.

In built-up areas where the majority of connections are made at the front of the property, a situation can arise where three sewers (a branch sewer in the road and a lateral at each side) should be constructed parallel to each other. This will be the most economical use of space as the corridor used by the lateral pipelines will be occupied by the inspection chambers.

Certainly lateral sewers crossing the street should be avoided wherever possible, because they increase the service congestion and cause additional obstructions for future pipelines as for cable construction along the street.

3.3.2 MANHOLES

The existing manhole designs are satisfactory and there are no obvious changes seen to be necessary. The size of the manholes and the concrete wall thickness are both satisfactory.

GRP liners are more reliable than PVC liners since PVC liners have jointing weaknesses. To prevent hydrogen sulphide and acid gaining ingress in to the concrete the following aspects should be considered:

To prevent hydrogen sulphide and acid gaining ingress into the concrete the following aspects should be considered.

a) GRP liner thickness should be specified such that they will withstand any damage during maintenance work.

b) Clay ware channels should be used rather than the in situ concrete channel.

c) Minimum thickness of 10mm epoxy should be applied. (Fig. 3.4).

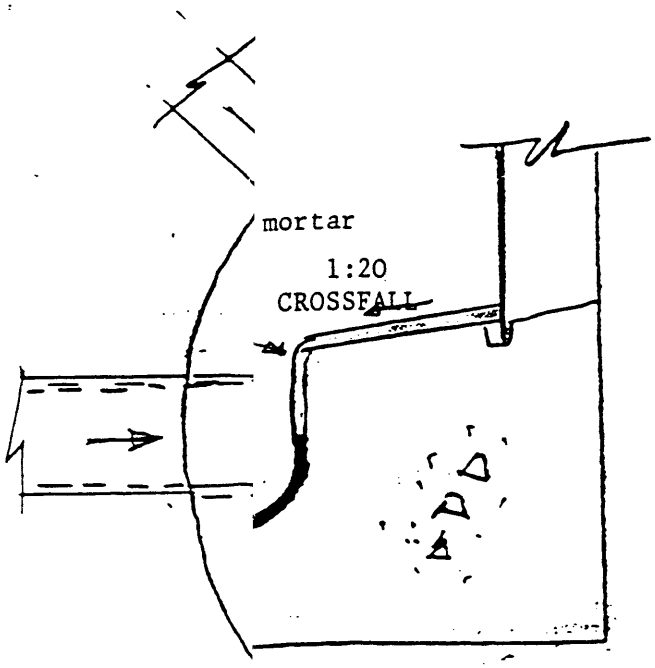
d) Sharp edges should be avoided.

e) Attention should be paid to the detailing of the joint between the benching and the incoming channels.

f) Coal tar coating should not be used on the benching because experience has shown that it is not satisfactory.

j) No ladders or iron steps should be installed in the manholes.

neering brick



-A

Figure **3.4**
Minimum thickness of epoxy mortar details

k) Sealing plates of GRP material should be provided below the manhole cover in order to prevent sand from entering into the manholes. This will also protect the cover from sulphide attack.

l) Engineering bricks should be used for raising and fixing the cover.

m) Special care should be taken to protect the mortar between the brick and this is illustrated in (Fig. 3.4).

3.4 PUMPING STATIONS

3.4.1 GENERAL DESIGN CONSIDERATIONS

The wet well/dry well pump stations have an advantage over the equivalent submersible stations. Hence the maintenance gangs can undertake maintenance work without having to lift the entire pump out of the well. Submersible pump stations are used for small flows and also where there is limited space. (Fig. 1.4 and 3.5).

The following points illustrate the types of the pumping stations used:

a) For district pumping stations, both lift stations and small pressure main stations are used as submersible pumping stations.

b) For main pumping stations, wet wells/dry wells are used for larger incoming flow. (Design capacity greater than 300-350l/s).

Table (3.4) illustrates district pumping station and main pumping station design details.

Table 3.4 District Pumping Station and Main Pumping Station

DISTRICT PUMPING STATIONS	MAIN PUMPING STATION
1. Single sump	Two sumps (wet well/dry well)
2. Circular sump	Rectangular sump well
3. Two pumps (one operating and the other on standby)	Four pumps: three pumps operating and one on standby
4. Submersible pumps	The motor is placed in the housing above the dry well and connected to the pump by a drive shaft.
5. Small flow (maximum flow 300 l/s)	Large flow (minimum flow 300 l/s)
6. The pump operates at a fixed speed	Pumps operate at a fixed speed and also at variable speeds.
7. Internal sump wall is lined with GRP liner.	Internal sump wall is lined with PVC.

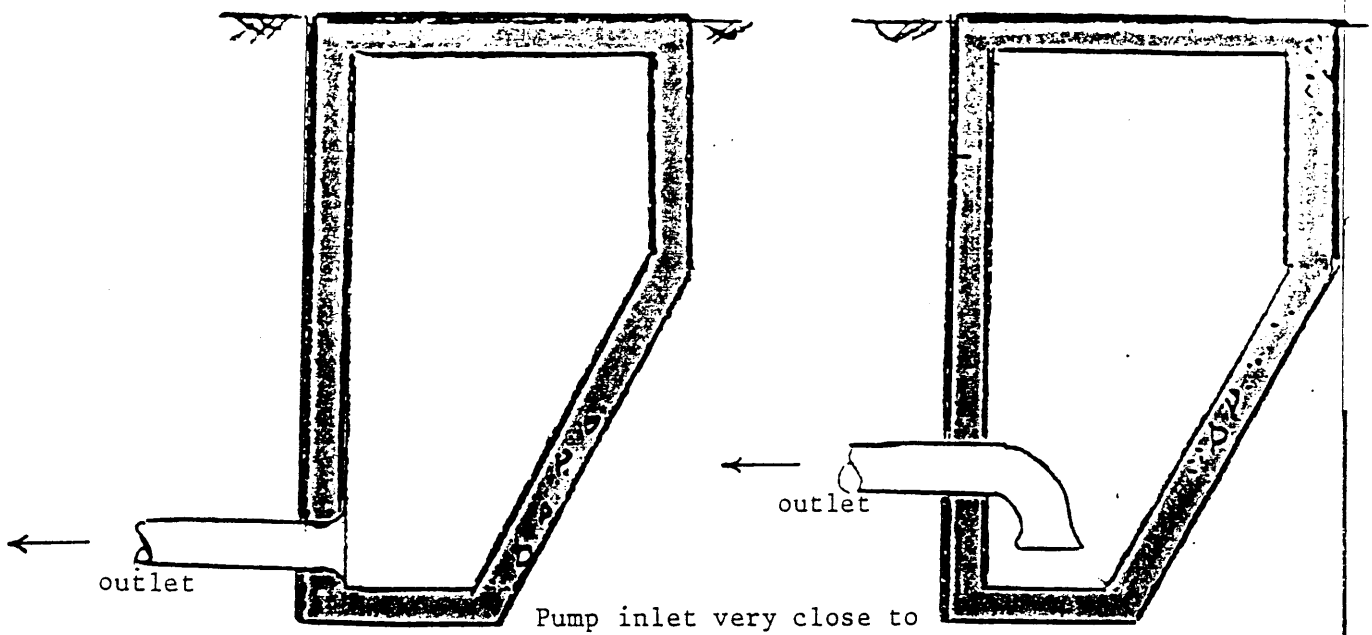


Figure 3.5

Removal of grit by pumping station

3.4.2 DISTRICT PUMPING STATIONS

The fixing of the submersible pump has been giving problems due to the corrosion of the bolts holding the stand, which the pump is fixed to. This caused an entire pump to move away from its position. (Fig. 2.5). This is often because the grouting and fixing of the bolts at installation was sometimes not a solid fixing. It has happened that the bolt and/or the nuts have worked loose. Such problems can be solved by using a stainless steel bolt system, which is both drilled and grouted with resin. Hence all the bolts and nuts, which are to be used in the sump should be of stainless steel.

Cast iron and ductile iron pipe work have been used in some pumping stations and they have proved to be inadequate against corrosion. (in high septicity levels). Also stainless steel pipe work has been used, but this has corroded during a similar period of time as that of cast iron and ductile iron pipe. GRP has recently been used by MWPW and MOH in some of the new pumping stations and it has been working satisfactorily.

Pipe work will also need to be changed from time to time. However, it should be remembered that the changing of GRP pipe work is much easier than is the changing of cast iron or stainless steel pipes.

There should be a standard length for pipe work in the sump, because it would make

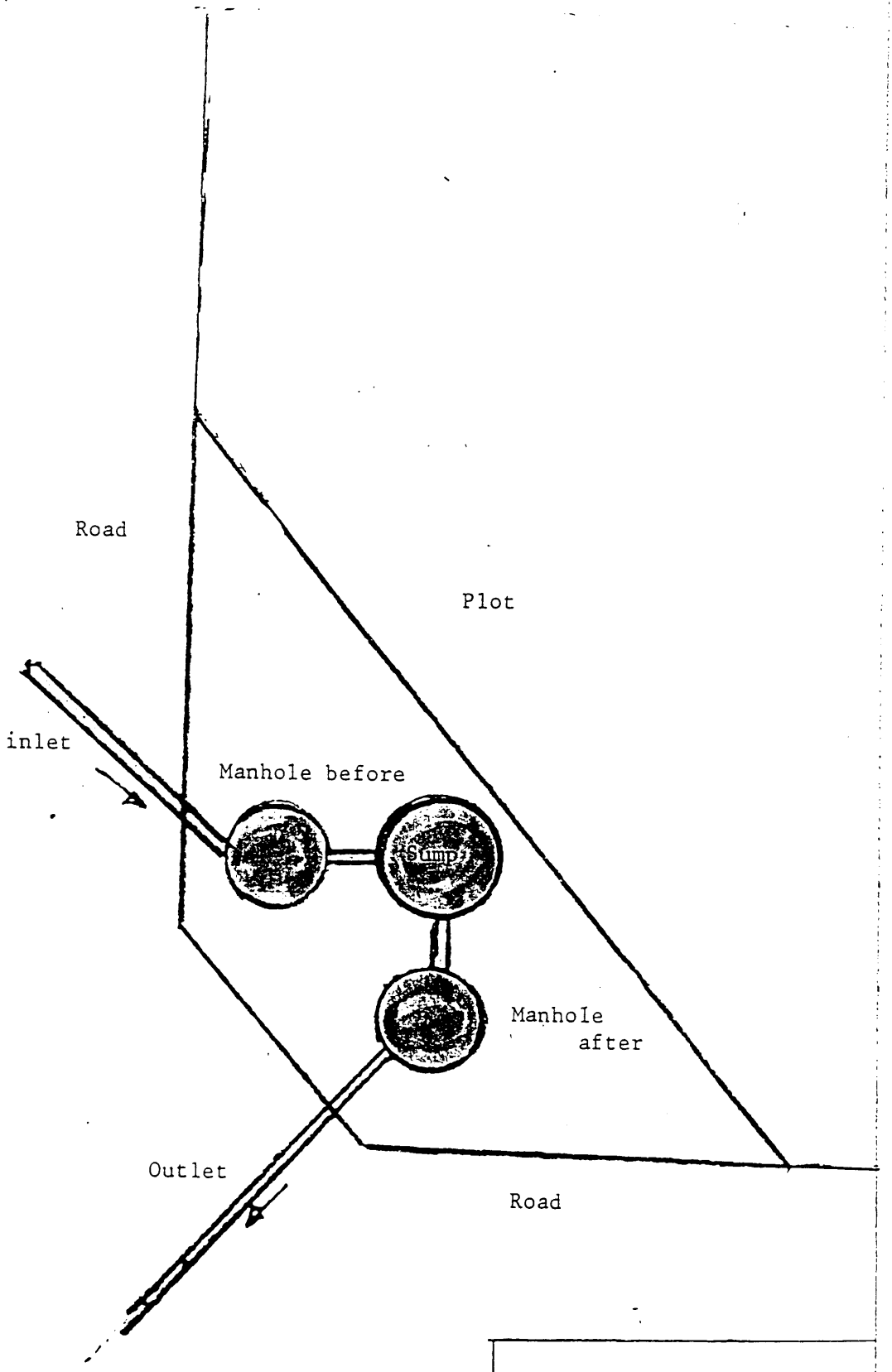


Figure **3.6**
District pumping station layout

the replacement of the pipe much easier.

To assist the program of maintenance, two extra manholes should be provided inside of the pump station, one before the sump and one after. (Fig. 3.6). In the first manhole all incoming sewers are to be collected, so there is only one inlet into the sump itself. The second manhole is provided to assist the bypass pumping in the case of maintenance or major failure and also to control the outlet position from the sump.

3.4.3 MAIN PUMPING STATIONS

All of the main pumping stations in Bahrain have been installed with single speed pumps, except for two (pumping station A1 and pumping station D1). (Fig. 1.3).

Pumping station A1 has variable speeds and the other has two speeds. The purpose of the multiple speed pump is to match more closely the incoming sewage flows and to pass them on to their destination with the minimum delay. However multiple speed pumps require more complex control circuiting and an additional switchgear. Therefore they will be much more complicated than the single speed pumps. Hence single speed pumps should be used in all future designs unless special circumstances dictate the need for multiple speed pumps.

To assist accurate future planning flow measurement equipment should be installed in the main pumping stations, which will enable the MWPW to proceed with the design

of new stations and the renovation of existing stations. Also it will introduce new designs, or confirm old ones, concerned with grit removal from the pumping stations or rising main.

"The measured flow for several study periods can be correlated with pump operating time, then the quantity of influent in the future can be estimated by reading the total hours of pump operation from the times attached to the electrical controls of the pumps". (Hammer, 1986).

It is preferable for the larger pumping stations with multiple duty pumps, to have more than one sump. This will provide a facility for the shutting down of one sump at a time while maintenance operations are performed (mainly degritting).

The more sumps there are, the more penstocks (flood gates) or other devices are necessary for isolating each sump. Taken to an extreme each pump could be provided with its own sump to draw from, but the numbers of penstocks that would be required would be excessive and this is not to be encouraged, because most penstocks in trunk pumping stations (where overflow facilities have been provided) have corroded to the point where they are unusable within a period of six years. Therefore pumping stations should not be provided with more than two sumps.

3.4.3.1 GRIT HANDLING

As has been mentioned in section 2.6.2.1, grit handling in Bahrain needs a more simple or an easier way of handling grits. The author recommends three different types of grit handling which should be used in Bahrain and these are as follows:

a) Pump operation and intake

In order to ensure the maximum possible pumping of grit through the sewerage system so that minimum silt or grit is trapped in the sump, this intake should be very close to the bottom of the sump where the grit will tend to settle first. (Fig 3.5). The detention time of the pump operation should be between 6-20 minutes, so that the accumulation of the solid will be much less. The pump velocity should take into account the cleaning requirements in the main, or providing a mixture in the pumping sump so that it will prevent silt or grit from settling. The sump flow should slope towards the pump intake and the pump intake should be located at the lowest part of the sump so that the grit will flow directly into the pump intake. Using this type of system to handle grit could cause serious effects to the pumping plant, although this system requires much less manpower to handle the grit problem. With this in mind, the author recommends that this system should only be used when there are no other alternatives (such as b or c in the following section). Such situations could be where there is lack of space or where the pump sump is located in the road and there is no chance of building other manholes beside it or there is only one sump in the pumping station.

b) Desilting Manhole

All new lift or rising mains stations in Bahrain have an extra manhole provided before the pump sump in the pumping station. Therefore this manhole can become the desilting manhole (Fig 3.6 and Fig 3.7).

This desilting manhole will trap all the grit and silt and so will prevent the accumulation of grit and the silt in the pump sump. The benefit of this system is that the pumps will be less affected by silt or grit. The desilting operation can be carried out without involving a pump sump.

The desilting operation will be much easier and will take a lesser period of time. It can be achieved by connecting the pipe from the suction tanker with the top of the existing pipe in the desilting manhole and this suctioning the grit and silt out of the manhole. There is usually no need for workmen to enter the manhole.

The disadvantage of this system is that it might increase septicity in the manhole, but since all the manholes in Bahrain are constructed with protection against septicity, there will be no problem. The author recommends the transfer of all manholes before pump sump to the degritting manhole.

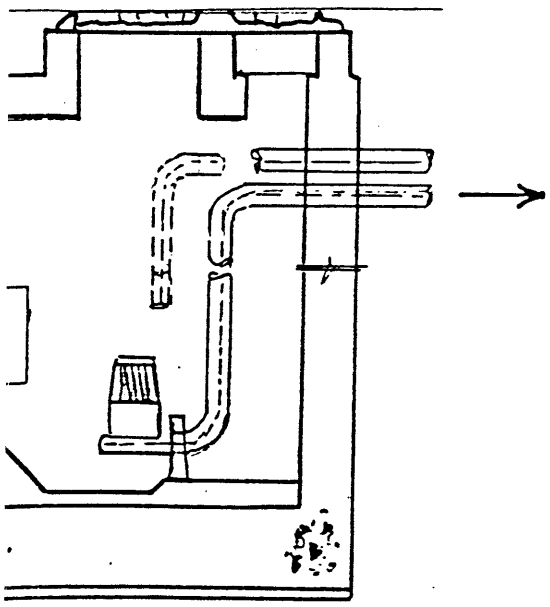
c) Degritting System

As has been mentioned previously an extra manhole is provided before pump sump

in the pumping station. This manhole can be transferred into a degritting system manhole which can clear the manhole from the grit and the silt in ten minutes (Styles 1990) (Fig 3.8).

The benefit of this system is that only one man can operate the system. The system is very effective and efficient. It can operate with or without (Styles 1990) a suction tanker or any other machinery. The disadvantage is that it needs clean water so that the jets will not be blocked. There should be a sufficient pressure in the degritting systems, so that a sufficient agitation in the jet is provided (which will help the settled grit to be separated). A reservoir should be installed a reservoir at a height or water pressure provided by a pump. The author recommends that the installation of this type of system could simplify the desilting process from the pumping stations.

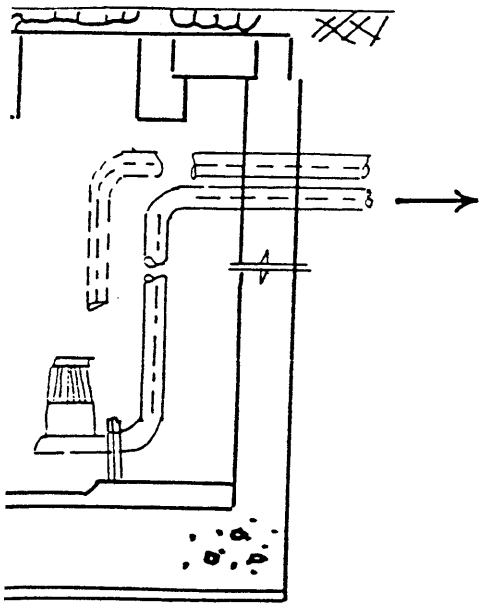
Accumulation of silt and grit in the sewage system in Bahrain is causing a lot of problems. This needs to be a frequent removal of silt from the system for it to operate normally. (Blockage of the sump affects the pump performance). Mainly the accumulation of silt is due to the environment itself (because there is a lot of sand everywhere). The existing sewerage system in Bahrain does not have a settlement well, where the silt can accumulate. Therefore, all the silt accumulates in the pumping station sump, because it is the lowest part of the system. Therefore the existing system should be modified with a more effective system so that it can operate in a better manner.



ꟾPING SUMP

DESILTING MANHOLE

FIG 3.7



NG SUMP

G SYSTEM

FIG 38

Hence the author has recommended three methods which could improve the desilting systems. The systems will reduce the manning power and the number of suction tankers required. Also they will give much easier methods and will hasten the desilting process. Extra labourers in the sump might not be necessary at all.

The author also recommends that better management could be achieved by providing an inventory of the time, data, quantity of silt and location of each station. This will not only give management a better idea of performance, but will improve that performance by demonstrating that it is being monitored.

3.4.3.2 ODOUR CONTROL

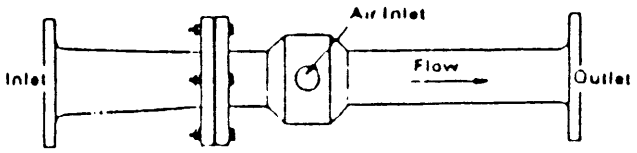
As has been mentioned in section (2.6.2.2) oxygen injection, gas scrubbing plants and chemical dosing are both used for odour control in Bahrain. But it has been seen that they are not satisfactory and unsuitable for use in Bahrain.

An air injection method has been proposed by the author, which could be used for odour control and a complete description of it follows.

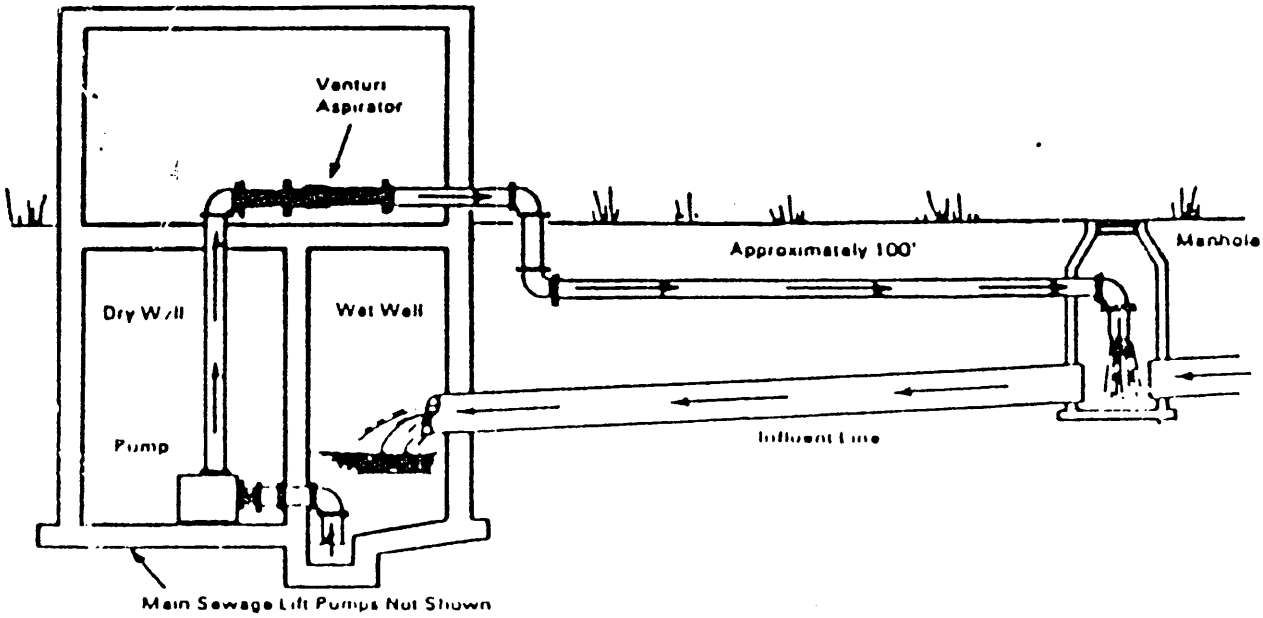
Air Injection

"The addition of sufficient air into sewage which contains oxygen prevents or significantly reduces further sulphide generation occurring and allows biochemical oxidation of the existing dissolved sulphides." (Bowker and Smith 1989)..

"The application of air injection into waste water needs the necessary pressure into the sewage. For small applications small equipment will be necessary which will consist of merely a pumped a venturi aspirator. (Fig 3.9). Larger applications will probably require multiple compressor and more sophisticated control systems with possible dissolution systems. If this is not done the separate housing for compressor and associated controls may be required (Fig 3.10). Usually air injection is installed down stream . Air injection depends upon the temperature of waste water characteristics (O_2 uptake rate), detention time in rising mains, hydrolysis profilt of the rising main and the degree of sulphide control desired." (Bowker & Smith 1989)



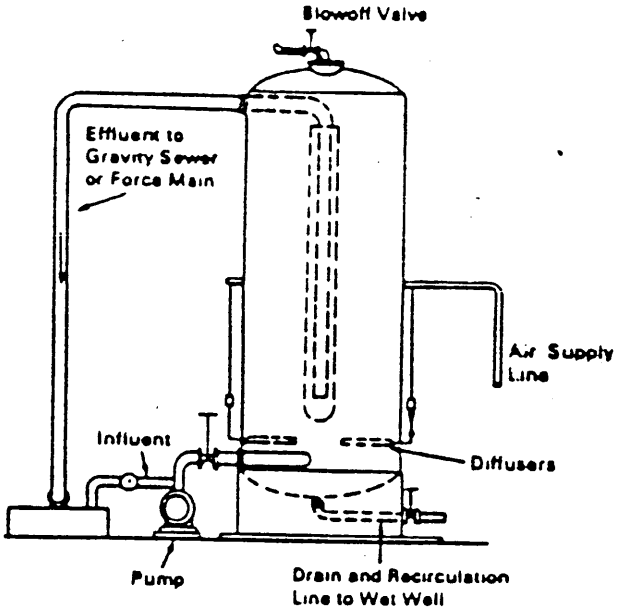
Typical Venturi aspirator.



Application of Venturi aspirator for sulfide control in wet well.

(Source: B&S 1989)

FIG 3.9



Pressure tank dissolution system

(Source: B & S 1989)

FIG 3-10

The disadvantage of an air injection is the creation of an air pocket which may have an effect on the sewer line. Hence careful control is needed to avoid an air pocket which may have an effect the system. Also air injection nozzling might become blocked.

The advantages of air injection are:

1) Air does not cost money.

2) No danger of explosion.

3) Self purification of the sewage in sewer lines.

4) No need for complicated equipment, only an air compressor.

5) Much less O & M requirement.

6) It can be used in pumping stations, lift stations, rising mains and small sewerage systems.

7) It controls both septicity and odour (Nuisance).

Therefore, this is a more reliable plant and requires less maintenance and operation skills than the other three methods, which have been mentioned before. Hence the author recommends that Air injection should be used for controlling odour and septicity problems in the sewerage system. Also, problems of blockages can be solved by frequent maintenance.

A complete design of air injection systems is listed below.

Design of air injection systems

"The first step in the design of air injection systems is to estimate the air requirements

to achieve the desired objective of sulphide control. This is based largely upon expected oxygen uptake rates in the sewer. Then the overall oxygen balance in a sewer can be described by" (Bowker & Smith 1989)

Where

$$\frac{doz}{dt} = Rf - Rr - Re$$

where

$\frac{doz}{dt}$ = rate of change of oxygen concentration as the stream moves down the sewer (mg/l-hr)

Rf = Rate of gain of oxygen by surface reaction (mg/l-hr)

Rr = Rate of loss of oxygen due to reaction in the reaction. (Oxygen uptake rate of waste water (mg/l-hr)

Re = Rate of loss of oxygen due to reaction on the slime layer (mg/lhr)

In rising main, the surface reaction rate, can be assumed to be zero.

In gravity sewers,

$$Rf = 0.96\ CA\ T\ (Su)^{3/8}\ D(dm)^{-1}$$

Where

CA = Turbulence factor

$$CA_2 1 + \left[\frac{0.17U^2}{gdm}\right]$$

Where

0.17	=	empirical coefficient
g	=	gravitational constant
T	=	temperature coefficient
S	=	slope of energy grade line m/m
U_2	=	Stream Velocity m/s
D_2	=	Do deficit, mg/L
dm	=	mean hydraulic depth (m)

Values for Rr, oxygen uptake in waste water should be experimentally derived from a sampling at several locations along the sewer reaches of interest. The sampling period should fully cover normal diurnal flow variations.

Where

Re	=	5.3 [O2][Su]1/2 R^{-1}
Re_2	=	5.3 $[O_2][Su]^{½}$ R^{-1}

Where

[O2]	=	Oxygen concentration, mg/L
S	=	Slope of the energy grade, m/m
U	=	Stream velocity m/s
R	=	Hydraulic radius of the stream, m. (Bowker & Smith 1989)

Several different approaches can be employed to estimate air requirements for sulphide

control in a particular sewer reach.

One approach involves the following procedures:

A) Determine the typical initial oxygen uptake rates, Rr in waste water from sampling programmes and develop the oxygen uptake curve as shown (Fig 3.11).

B) Calculate the waste water detention time in the sewer reach at minimum average peak flows.

C) Measure oxygen uptake at i-hour increments on samples collected during periods of maximum oxygen uptake.

D) Plot oxygen uptake rate with the time as shown in (Fig 3.12).

E) Calculate the area under each curve in (Fig 3.12) for cumulative increments of time to yield the oxygen demand in mg/L.

F) Plot ratios of oxygen demand to initial oxygen uptake rate as (Fig 3.13).

G) Multiply design oxygen uptake rate by above ratios at detention times corresponding to yield oxygen requirement in mg/l for design purposes use ratios from (Fig 3.13).

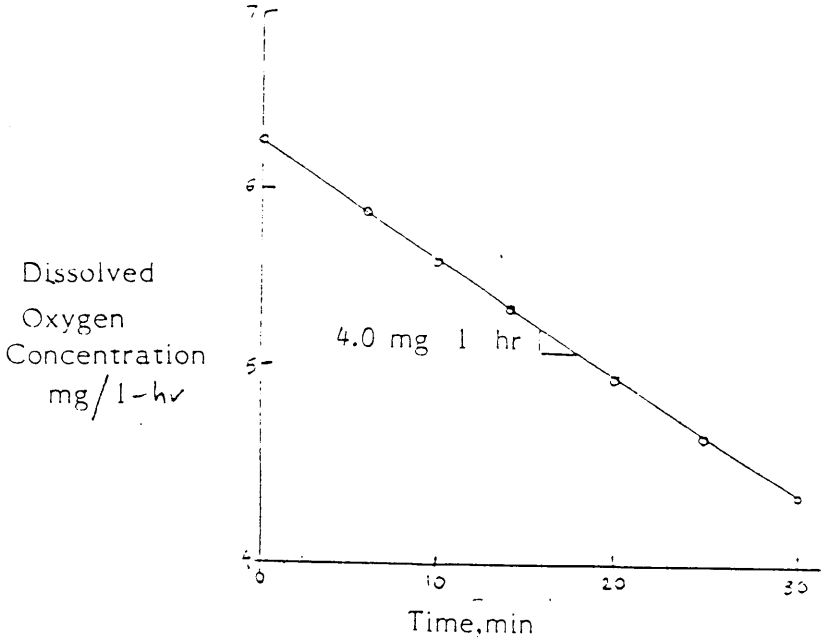
H) Calculate oxygen uptake rate by slime layer Re

$$Re = 5.3\,[O2][Su]^{1/2}\,R^{-1}$$

I) Multiply oxygen uptake rate in the slime layer by ratios obtained

(G) To yield the oxygen requirement by slime layer (mg/L) at various detention times (flow rates).

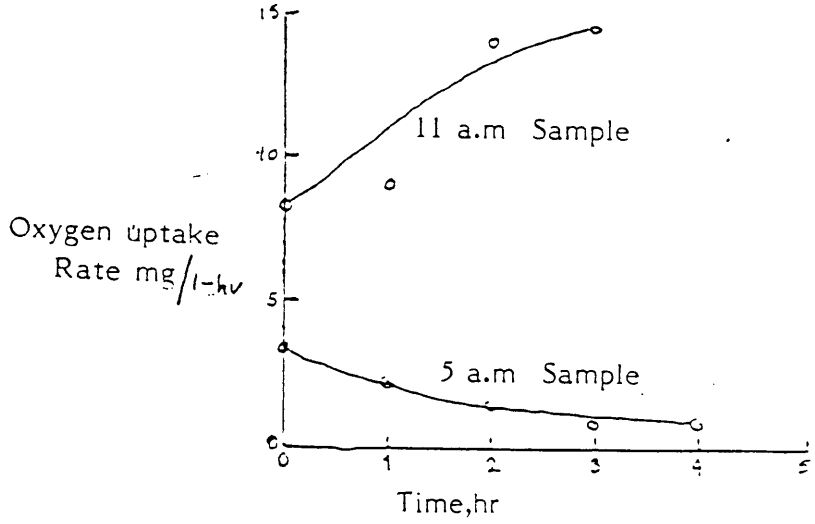
J) Add results from steps G and I to yield total oxygen required in mg/L at various rates.



Example of variation in Oxygen uptake rate with time

(Source, B&S 1989)

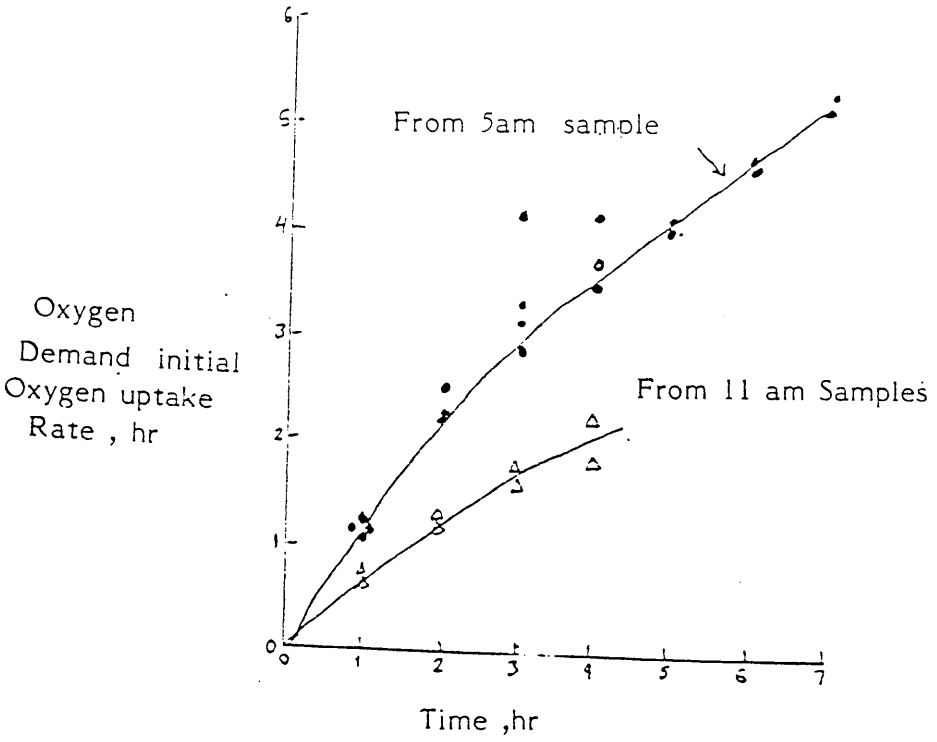
FIG3.11



Example variation in Oxygen uptake rate with time.

(Source, B&S 1989)

FIG 3.12



Example Ration of Cumulative demand to initial Oxygen uptake VS.Time

(Source, B&S 1989)

FIG 3.13

K) Determine mass flow rate of oxygen required (flow x concentration) Kg/d.

L) Calculate air requirements in m^3/hr for minimum, average and maximum flows.

For direct air injection in force mains an alternative approach is to use the following:

$$Qa = \frac{(6x10^{-7})VRr}{P_1 \log[CP_1 + P_2/P_1]} \times \frac{Ua+Uw}{Ua}$$

Where

Qa = Required air flow at ambient force main pressure m^3/min.

V = Volume of pressure main m^3

Rr = Rate of loss of oxygen due to reaction in the stream (oxygen uptake rate of waste water) mg/l-hr.

P1 = Ambient atmospheric pressure, atm.

P2 = Gauge pressure at injection point, atm.

Ua = Velocity of the air relative to atm, waste water m/s = 1.66 $d^{1/2}$.

where = pipe diameter (m)

Uw = Velocity of the waste water (m/s).

This equation generally predicts more than enough air for sulphide control, since it is based on maintaining an excess of oxygens in actual practice. Adequate sulphide control can often be achieved using one third or one half as much air as would be required to satisfy. The oxygen consuming capacity of the waste water. It should also be noted that pressure greater than 276 Kpa (40Psi) is due to the large impact on both oxygen and nitrogen dissolution.

3.4.3.3 OVER FLOW

As has been mentioned in section 2.6.2.3., an overflow system which is used in Bahrain needs some modification, also the author recommends a modification and another type of overflow system.

The suggested improvements to the system are as follows:

1) Pipe connection from pumping station to the sea is a good drainage system, but it has some disadvantages.

   A) Dumping of untreated waste water into the sea will have an effect on the aquatic life and hence will pollute the environment.

   B) Wastage of water by the dumping of waste water in to the sea. Waste water is usually used for Irrigation after it has been treated. (This is due to the lack of water in Bahrain).

2) As has been shown in the pumping station overflow reservoir (Fig 2.13), waste water is released due to failure into the reservoir. It has not been considered for land application or for recharging of the ground water.

Usually the overflow reservoir is used once in a year or once every two to three years and it is used for a very short period until the failure in power and pump is amended.

New land application of waste water on the ground will percolate directly through the ground and hence will give the benefit of recharging ground water. This type of overflow reservoir is an open space, therefore it can be considered also as a storm

basin. Using this type of overflow reservoir it will cost much less than those which have been built (Fig. 2.13) because the base of the proposed reservoir will not need an asphalt base or any other hard material.

It is recommended that in constructing any overflow reservoir for a pumping station, it should be built so that it can be used as an overflow for a pumping station and at the time as land application and as a storm basin, (Fig 3.14) (This type of land application is not applicable in low lying areas).

3) A standby generator is used as an overflow system in the pumping station. This type of overflow system has been used in one pumping station in Bahrain (Isa Town pumping station). This pumping station always gives problems when there is heavy rainfall or breakdown in the down stream of the sewer. Therefore BSOM always arranges a staff and tankers to be on standby basis. This step has increased operation and maintenance requirements for this pumping station. A standby generator system has been used in this pumping station because of the lack of space in the pumping station.

The author recommends that a close overflow reservoir could be a solution for this pumping station (Fig 3.14).

The capacity of the close overflow reservoir should be minimum, between one to two

days. (Daily flow in the pumping station 7500 m^3/d).

Hence too full capacity of close overflow reservoir will have a size of = 28m x 27m x 5m, which seems to be quite big, therefore four smaller reservoirs might be more sufficient = 20m x 19m x 5m. This is because constructing a smaller reservoir will need smaller structural cover and will reduce construction costs and simplify construction (a smaller reservoir is much the easiest to build). Also replacing the cover of a small reservoir will cost much less than the replacement of a big reservoir cover. A smaller reservoir could be built in different stages and in different places rather than all of it being built in one place. Also, there might not be a sufficient space available around the pumping station, therefore a small reservoir is much more suitable than a large one.

Therefore total capacity available will be for three days of full capacity (a one day flow can be stored in the system itself).

Also, the author recommends that where there are pumping stations discharging waste water into the sea, they should be provided with close overflow reservoirs.

This is because of:

A) Avoiding environmental pollution (the effects on aquatic life).

B) Lack of fresh water in Bahrain.

It is not economical to dump any water into the sea while that water has been

desalinated from the sea with high cost. Also this water will help in reducing the usage of fresh water for irrigation (because there was a study in Bahrain University that about 70% of the water consumption in Bahrain is being used for irrigation). Therefore all water could be re-used and it would be a benefit from using a close overflow reservoir.

CHAPTER FOUR

4.0 OPERATION AND MAINTENANCE

4.1 OPERATIONS AND MAINTENANCE OBJECTIVES

The major objective in a sewerage system operation is to enable the sewage to flow from its source to the point of disposal in an economical manner. At the point of disposal the sewage should be correctly discharged so that there are no harmful environmental effects posing a risk to public health.

The following headings summarise the main objectives.

a) Public Health

The operation of the sewerage system should be in such a way as to maintain flow at all times and to minimise any risk to public health from untreated or partially treated sewage discharging into the sea and contaminating the environment.

b) Septicity, Corrosion and Smell Nuisance

To minimise the effects of septicity or corrosion of the system and the smell nuisance during the operation of sewerage systems it is essential that the sewage flow should be constantly moving towards the point of treatment with no delay.

c) Running Costs

To minimise running costs the systems must operate in the most economic manner possible. High running costs are caused by wastage of resources. There should be

feedback and thorough analysis of information so as to ensure continuing improvement in the operation and maintenance of the system and optimum capital works design.

d) Maintaining Assets

Adequate planning is necessary to ensure preventative maintenance so that capital assets remain fully functioning for their operational life. Therefore it is essential to consider carefully in general terms assuming reasonable design both in manufacturing and construction standards together with the correct operation and maintenance procedures.

The following could be assessed as typical operational lives.

"A Sewer and manholes - 60 years

B Pumping station and treatment plants - 60 years

C Machinery and switchgear - 12 to 18 years". (Hawksley, 1985)

Much of Bahrain is low lying and flat particularly in the coastal areas which are also the most densely populated. (Fig. 1.2).

This means that very few sewerage systems can rely on gravity alone for the transport of sewage from its source to the point of disposal.

Pumping stations are essential features in most of the sewage systems in Bahrain.

They lift the sewage so that it can gravitate onwards to the next station and further to the point of the disposal. (Fig. 1.4).

This means that failure of any one station will interrupt the sewage flow and will prevent the normal discharge of all the sewage upstream of the system. A slow response to rectify such a failure could cause the system to fill up and the sewage to overflow onto the ground.

Blockage in sewers could also cause similar conditions. Septicity is in great measure due to the inefficient transfer of sewage. This could be the result of improper setting of the pumping station controls and a consequence of over long storage periods of the sewage in the sewers often entering the pumping stations. Partial blockage can also cause sewage to back up in a sewer and this creates septic conditions. Therefore most of the sewerage systems in Bahrain are designed to cope with the effects of septicity, but all systems are susceptible to some extent.

Inadequate operation and maintenance can lead to an increase in the costs of running a sewerage system. In addition there are failures in replacing worn or defective parts in pumps. This leads to extended pump running times and an increase in the use of energy. Taking into account the above inadequate operation and maintenance factors it can be seen that there could be total failure of a major capital asset. This might result in there being no means of sewage or sewage disposal until major and expensive

providing works having been carried out, so causing risk to public health which the original system had been intended to protect.

Operation and maintenance can be separated into two categories:

a) Routine Operation and Preventative Maintenance.

b) Emergency Operation and Maintenance.

The difference between them is usually dependent upon policy and the funding available. Routine works tend to be much more expensive in the short term, but they enable the capital asset to have a longer overall life. Emergency work tends to be cheaper in the short term but results in the earlier replacement of capital assets.

4.2 SEWER

4.2.1 RODENT CONTROL

Rodents living and breeding within a sewerage system can pose a serious risk to public health and cause structural damage to the system. It is essential that the rat population is controlled by a programme of systematic poisoning carried out in conjunction with the Public Health Directorate. Approved baits should be used.

A little copper sulphate could be used in sewers for killing rats without any side effects to vegetation if the sewage water is used after treatment for irrigation purposes. It is essential that all service connections to house drainage are plugged so as to prevent the rodents entering the domestic sewerage system or leaving an access for

rats to the dry area surface.

4.2.2 SEWER INSPECTION

Annual inspection of all manholes and chambers is ideal, but this is not possible at present because of the rapid expansion of the sewerage system of Bahrain. It is recommended therefore that all the manholes on the trunk sewers together with those manholes with known problems are inspected annually. This does not mean that the other manholes are never inspected, but that they will be inspected at longer time intervals. However, these inspections will be absolutely determined by experience and they will be influenced particularly by reference to the need for corrosion control.

During the inspection of a sewer it is important to record and report on the condition, giving details of corrosion, evidence regarding rat populations, signs of surcharge and any damage. A successful inspection program operates on a planned schedule carried out inside the pipelines, either virtually for man entry sewers or by CCTV survey in non man entry sewers.

4.2.3 SEWER CLEARING

Much of Bahrain is low lying and flat, particularly in the coastal areas which are also the most densely populated. Improper design, or poor construction will result in serious and expensive sewer problems.

Frequent cleaning records should be kept on the maintenance performed, especially in regard to a sewer of low flows or with an inadequate gradient.

Other sewers will require no cleaning for many years. A sewer pipe is cleaned either by jetting or rodding, or by the addition of chemicals. Manholes and inspection chambers should only require a little cleaning, except where damage has occurred to the sealing plates below the covers, or where they have been removed. If such cleaning is required it should be carried out at the same time as in the inspection areas.

4.2.4 SEWERAGE SYSTEM RENOVATION

All repair work to manholes and inspection chambers should be carried out at the same time as routine maintenance. It is important that the integrity of the corrosion protection systems is maintained so as to prevent early deterioration of the concrete wall the benching, the brick work and the covers.

Special attention should be given to the type of materials which are used for repairing the plain concrete benching, protection to the underside of suspended slabs and the brickwork so as to avoid major rebuilding works.

It is necessary to arrange for immediate repair particularly where there is damage to the concrete benchings in manholes, rather than delay repair because leaving repairs

to deteriorate greatly will cost much more in the long term. Also those older manholes might be checked annually after being repaired. An inventory of the conditions of all manholes which have been repaired and those needing repair should be repaired, following inspection by the maintenance staff, so that staged renovation work can be put in hand.

4.2.5 CLEARANCE OF BLOCKAGE

Flooding of sewage is regarded as emergency work and therefore it should be given the highest priority. It is extremely important to have a maintenance staff on standby on a 24 hour basis. Also accurate records are to be kept both of the location and the cause of each blockage so as to provide feedback of information for both maintenance and design purposes.

Blockage of a sewer is usually caused by mis-use by the public. Contractors also cause problems by mis-connection and by damaging the existing system during construction. Also poor sewerage design can cause blockages.

Blockage by the public can be minimised by instruction through educational programs. Instruction can be achieved through the mediums of television and radio and in schools also by informing the residents both through writing or verbally. All houses having connections to the main sewerage system should be instructed regarding correct management. Also any house service connection which is connected to the main

system should be trapped or plugged so that the system cannot be used for the disposal of refuse or building materials, fuel, oil and surface water.

Poor sewer gradients or poor branch connections can result in the blockage of the main sewer. It is most important that such cases, and the type of development sewered are accurately recorded in order to provide feedback for the designers. Usually maintenance staff are assumed to be responsible for the clearance of blockages which are outside the properties. It is preferable that when a blockage has been reported that first an inspector should be sent to the location of the blockage, to check the situation there before committing plant and labour. Such a procedure would help to save time and a better spread of the work load for emergency crews.

Blockage due to construction activities must be restricted by legislation, which will enforce the guilty party to pay for any damage or blockage caused to the sewerage system. It is vitally important to set up an effective system of enforcement which should be backed by law.

4.3 PUMPING STATION

4.3.1 STATION FAILURE

It will be necessary to establish an emergency scheme to avoid both flooding and the overflow of sewage, if there is a mains power failure or the breakdown of major equipment at a pumping station.

Recently the MWPW have purchased sufficient mobile pumping equipment to provide an over-pump facility, but only in the small pumping stations.

However it is extremely difficult to provide this facility in the major pressure pumping stations, so additional precautions should be taken to prevent the failures of pumps or power, and particularly in the stations without overflow backup.

Therefore the following aspects should be considered and the necessary actions be taken:

a) **Personnel** - a regularly updated list should be kept of all personnel, including supervisory staff, drivers, mechanics etc who are available, together with their contact numbers and addresses for call out. This list would be available at all times.

b) **Plant and equipment** - a status report of the essential plant and equipment should be made available to supervisory staff and should be updated continuously, stating location and availability.

c) **Co-ordination** - in the case of a major emergency it may be necessary to pool the resources of the Department of Sewerage Operation and Maintenance (DSOM) and those of other responsible bodies, such as the Public Health Engineering Department (PHE), Water Supply Directorate and even possibly

those of the contractors.

Prior liaison with these parties should take place to ensure that procedures are as those for an extended power failure, a major sewer collapse or major flooding.

d) **Control** - in the case of partial failure of the system, when either a single or a series of pumping stations fail, operational staff should be briefed upon the correct course of action. In particular they should be aware of which upstream stations should be switched and at which point to overflow into the sea, or into a drain and which stations are the most vulnerable with regard to the factors of potential flooding or high rates of flow.

4.3.3 CONTROL SETTINGS

There have been instances where pumping stations have been set for future flow conditions. This has resulted in an overly long retention of sewage in the sumps with consequent septicity and corrosion.

It is essential that operations personnel are aware of these consequences arising from such actions and that they are provided with sufficient flow information to enable them to set the level control correctly, thus minimising the time that sewage is retained in the pumping stations. Regular adjustment of the level controls in a pumping station

so as to achieve this objective should be thought of as a continuous process of fine tuning according to the changing flow rates. In connection with this problem, the installation of additional flow meters in certain major stations would greatly assist the operations personnel.

4.3.4 MONITORING

Monitoring of pumping stations should be carried out by the operational staff and the information should be relayed on to the maintenance staff.

Monitoring should be the responsibility of the maintenance staff. It should be carried out by capable technicians who have the ability to immediately report any problems or changes during their work period in order that any necessary maintenance work can be put in hand immediately. Some pumps might run for an excessive number of hours, due to worn, or partly blocked impellers and a report might not be recorded. This will cause an increase in running costs, and highlight the necessity for frequent reports regarding the condition of pumps which should be provided by the personnel monitoring the stations.

This means that daily visits should be made to all of the main pumping stations in order to monitor flows, any breakdowns or other problems. The cost of such attendance is very high, but the effect of failing to act quickly in a case of emergency, such as a fire in the control panels, could be serious e.g. flooding and pollution

combined with very high over pumping and replacements costs, especially in the main pumping station with no overflow backup.

Instrumentation, Control and Automation (I.C.A) which includes telemetry, is an important tool in the hands of anyone operating an efficient system of sewerage and sewage treatment, particularly where labour is not cheap. The capital costs of ICA are not normally excessive but modern systems are complex and there is the cost of the salaries of the trained personnel who are required to maintain such a facility.

Therefore it is essential that the equipment is suitable for use in Bahrain and that it will be able to perform with the utmost reliability in the aggressive Bahrain environment.

In any comparison made between ICA and the full-time manning of pumping stations, full allowance must be made for time running costs of ICA systems and the saving in labour costs.

Regular observation of the sewerage system together with careful maintenance will greatly improve the life of a system. The ICA system can monitor the whole sewerage system without requiring personnel at the station. Information is transmitted by telemetry back to a central data logging facility.

This would enable central "control" to be advised of any condition at any major pumping station automatically without relying on a station attendant. Because the telemetry could be a two-way control it would also be able to call up any station at any time to verify the current situation. This could be extended to the transmission of control commands to the stations for instance to change the duties of pump etc.

A study should be considered for the ICA system and also how it could best be used as an economic alternative to manned attendance. This is in consideration of the growing number of pumping stations and the continuous expansion of the sewerage systems in Bahrain.

CHAPTER FIVE

5.0 INSTITUTIONAL DEVELOPMENT

EXISTING INSTITUTION STRUCTURE

Sewerage work in Bahrain is done by two different organisations but the operation and the maintenance of both works are done under one department, BSOM (MWPW).

In all existing settlements or old areas where no sewerage system exists, the sewer work is done by the MWPW, which involves planning design construction operations and maintenance.

All new developments (new housing built by Government) and new town sewerage work is done by the MOH. The MOH does all the work from the planning, design and construction but not the operation and maintenance of the system.

Those two organisations are working separately with different standards and different constructors. Usually the MWPW gives their main big project to consultants, while the MOH design their project indoors. There are regular meetings between the MOH and the MWPW at the management level to discuss and agree on different points.

The author see that this type of cooperation will not solve the problems and will not give sufficient feedback data to the engineers.

For example: If there is a project going on, and the MOH engineers wanted to discuss

some point of the design with BSOM or they would not comment (because the MOH does not do O & M). The MOH engineer has to write a letter through their director to MWPW. This type of letter (due to bureaucracy) takes a very long time to be answered and there is also a similar situation if the MWPW wants to write back to the MOH. Hence the project will be completed without any comments from BSOM (MWPW) (the people who are going to be in charge of the operation and maintenance of the project in the future).

In order to improve this situation, the author recommends that there should be direct cooperation between the MOH engineers and the MWPW engineers without any complexity. This could be done by taking a copy of the letter which is going to be sent to them through their director. Also the MWPW should send their feedback on design to the MOH and if there are problems these should be discussed directly with the MOH engineers so that a better solution can be reached. Therefore more coordination at lower levels will reduce the problem and better systems could be achieved.

5.1 LIAISON AND STANDARDISATION

Standard details and design parameters should be adopted at the beginning of any project. If standard details and design parameters are included later it becomes very difficult to persuade different developers and promoting bodies to conform to common standards and details. These standard specifications for sewerage and drainage work will aid the smooth progress of the project with a minimum number of staff. However

there is also a need to standardise planning, design procedure, construction and handover.

Project Stages

The construction of any sewerage project should be done in the following stages:

"A **Planning** - (Preliminary details)

1. Outline drawings (Base map at suitable scale).
2. Population (Existing and future served)
3. Outline design details.
4. Outline programme of work.

B **Detailed Design**- (Detailed information)

1. Detailed drawings, including sewer plans at 1:500 scale.
2. Detailed design calculations for sewers, pumping stations.
3. Detailed specifications.
4. Detailed programme of works, including proposed programme of handover.

C **Construction and Handover** (Information and procedures)

1. Notification of the commencement of works to enable on-site inspection.
2. Handover procedure.
3. Provision of record drawings for civil, mechanical and electrical works in a standard format. Sewer records to be suitable for digitising."

(Hawksley, 1985)

5.2 SEWER CONNECTIONS

Additional staff will be needed to supervise connections to the sewerage system. Also consideration should be given to upgrade the existing system for processing the large numbers of applications for sewer connections. However in the absence of the policy for charging applicants for such connections, sufficient Government funds must be made available to meet demand, as if failure occurs in the supervision of sewer connections the results will be extensive damage and abuse of sewerage systems. This will include ponding of sewage due to poor standard of construction connections, giving rise to public health hazards through rodent generation and odour emissions.

5.3 ADMINISTRATION

Previously operation and maintenance or sewerage system were done by public health engineering. This department is changed to a new department which is now called DSOM. The new department is consist of two engineers and the others staff are all technicians. This means that lack of engineers will be causing difficulties in processing and analyzing the large amount of data.

Therefore the author recommends increasing the number of engineers and also the provision of an effective data base. These two items will achieve better coordination between the design, contract supervisor operation and maintenance sections.

The two items are as follows:

A) An increase in the number of engineers would allow more analysis of data and feedback of the design and contract supervision. This will provide the method and rate of frequency of recording and more control of the system. Also it will reduce the operation and maintenance requirement.

B) An effective data base would help in analyzing the larger amount of data and will establish what records provide the method and the rate of frequency of recording. Hence the engineers would have a better system of analysis leading to feedback to both design and construction sections of what has been designed and constructed. This would also reduce design time, improve efficiency of operation and maintenance and would also provide a better means of costing and budgeting for preventative maintenance and remedial works.

5.4 STAFF TRAINING

A Staff Training Programme is necessary for the success of any institutional development. It is necessary to establish a Training Programme which can cope with the development of the existing sewerage system.

Training Programmes should include suitable academic courses and practical work, which demonstrate the routine safety precautions and the action to be taken in an emergency situation. A number of staff should be appointed as safety officers after completing their training programme, so that they can train new engineers and

supervisory staff.

A training programme should include at least the following subjects:

a) Hazards involved in working in sewers.

b) General precautions

c) Method of testing atmosphere

d) Maintenance and testing of detection safety and rescue equipment

e) Principles and use of the equipment

f) Practical use of the equipment under working conditions

h) First Aid

i) Principles and importance of hygiene

j) Rescue techniques (Inst. of Civ. Eng. 1969)

k) Sewage works operation

l) CCTV Survey

m) High pressure jetting of sewers

n) Corrosion control

In the absence of such a training programme, the result could be inefficient and unsafe methods of working with consequent risk to the public, the workforce and early deterioration of assets.

CHAPTER SIX

6.0 CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSION

As has been seen from this report, the Bahrain Islands are low lying areas with very little rainfall and high temperature (28°C-40°C). Due to the flatness of the ground a flat sewer system is being constructed with a large number of pumping stations, which need many suction tankers and a large number of labourers. Due to the lack of labourers and machinery in respect of the number of pumping stations. This causes delays to the desilting process from the sump and increases the amount of silt in the sump before it can be removed (lack of rainfall caused the presence of a high amount of grit in the environment). Therefore, accumulation of silt or grit in the sewer line will prevent movement of sewage in the system.

In order to minimise the operation and maintenance requirements of the sewer network, the deposits can be removed at various points and by applying various methods. Also, a well managed system of removal is required. Because of delay of sewage in the sewer line with the presence of high temperature, this means that the septicity problem will be high. Different types of equipment have been used to solve the septicity problem but none of them are very satisfactory.

In order to improve the sewerage system in Bahrain a number of recommendations have been made by the author.

6.2 RECOMMENDATIONS

- In designing any sewerage system for Bahrain, consideration should be taken into account so that the sewerage should be kept moving continuously to the treatment works (Section 1.3).

- Each town should have its own treatment works so as to reduce the grit and septicity problems and also utilise a smaller land area for treatment plants (Section 1.5).

- There were no significant changes in average daily flow in a 10 year period; therefore, it is preferable to retain the safety factors at their present levels in order to guard against such uncertainties as pump performance, abuse, rainfall and surcharging, etc (Section 3.12).

- A spreadsheet has been developed to assist in the designing of sewers. This allows the designer to check that minimum self cleaning velocities are obtained at partial depths of flow in a minimum sewer gradient (Section 3.2).

- Initial and long term velocities should be considered when designing any sewer line (Section 3.2).

- Inspection chambers which are not located in the road, could be constructed

from PVC material. Also, an inspection point could be constructed if the depth is shallow (Section 3.3.1).

- To prevent hydrogen sulphide and acid gaining ingress into the concrete in manholes, a number of aspects should be considered and are as follows:

  Specified GRP liner thickness and clay wave channels are to be used, then application of epoxy mortar should not be less than 10mm thickness, sharp edges should be avoided, attention should be paid to the detailing of joints between and in the incoming channels, coal tar coatings are not to be applied on benching, no ladder or iron steps are to be installed in manholes and sealing plates of GRP material. They should be provided below the manhole covers (Section 3.3.2).

- To assist accurate future planning, flow measurement equipment should be installed in the main pumping stations, which will enable the MWPW and MOH to proceed with the design of new stations and the renovation of existing stations. Also it will introduce new designs, or confirm old ones, concerned with grit removal from the pumping stations or the rising main (Section 3.4.3).

- Three different types of grit handling should be used in Bahrain so as to simplify the desilting process from the pumping stations, as follows:

A) Pump operation and intake.

B) Desilting of Manholes.

C) Desilting Systems. (Section 3.4.3.1)

- Better management could be achieved by providing an inventory of time, data, quantity of silt and location of each station. This not only gives management a better idea of performance, but it improves that performance by demonstrating that it is being monitored. (Section 3.4.3.1).

- Air injection should be used instead of oxygen injection or scrubbing plants in order to control odour (septicity) problems in the sewerage system in Bahrain. (Section 3.4.3.2).

- In constructing any open overflow reservoir for a pumping station, it should be built so that it can be used as an overflow for the pumping station and at the same time for land application purposes and finally a storm basin. (Section 3.4.3.3).

- A close overflow reservoir should be used for a pumping station which has no overflow facilities or for a pumping station which is discharging its overflow into the sea. (Section 3.4.3.3).

- Routine maintenance to be carried out at the same time as desilting and cleaning. (Section 4.2.4).

- Instrumentation, control and Automation (ICA) studies are to be carried out. (Section 4.3.4).

- For better institutional development there should be direct cooperation between the MOH and MWPW engineers. (Section 5.0).

- Standard details and design parameters should be adopted at the beginning of any project. (Section 5.1).

- Ensure adequate supervision of service connections to the main sewer. (Poor branch connections can result in the blockage of the main sewer). (Section 5.2).

- Establish an improved system of recording and analyzing by computer which would reduce design time, improve efficiency of operation and maintenance and provide a better means of costing and budgeting for preventative maintenance and remedial work. (Section 5.3).

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Appendix A

1. Sewage Rehabilitation Manual Definition

2. Effect of hydrogen sulphide on sewerage systems.

Table A.1 Sewerage Renovation

Maintenance	The systematic inspection and cleaning of a sewer including minor repairs but not involving reconstruction of the main structural fabric or alteration of the original dimensions. Maintenance includes joints repair, renewal of inspection covers, step irons, etc., and sealing of cracks when the structural fabric is not impaired.
Renovation	The operation of effectively improving the conditions of an existing sewer by in-situ techniques such as will provide for a substantially increased life. Renovation may or may not improve the structure strength of the pipeline.
Renewal	The reconstruction of the whole structure of a sewer as an entity to the same dimensions as the original pipeline but not necessary in the same position.
Replacement	The construction of new sewers either in the same location as the original or in a new location so that the fundamental purpose of the original sewer will be incorporated in the functions of the new conduit. replacement can include a proportion of improvement or development work.
Man-entry sewers	A sewer of 1000mm diameter or greater is considered to be sufficiently large for internal maintenance and renovation to be carried out. Sewer from 600mm to 1000 diameter may be entered by the restricted space allows only simple tasks or inspection to be carried out.
Non-Man-Entry Sewer	A sewer of less than 600mm diameter is considered as non-man entry.
Structural Renovation	A method of sewer renovation which does not require the existing sewer to assist it in supporting the ground loading. In general this applies to pipes which should be laid as sewers in their own right.
Non-Structural Renovation	Methods which do not stand by themselves but augment the structural strength of the existing sewer are considered as non-structural. Generally the combination of the original sewer plus a non-structure renovation method plus pressure grouting forms a whole which is structurally strong".

Sewer Renovation, 1978

After investigation, MWPW had suggested figures for controlling:

"Hydrogen Sulphide:

1. Up to 5mg/l concentration in non-turbulent sewage can be accepted.

2. From 5-10mg/l H_2S concentration is the borderline.

3. Above 10 mg/l the concentration is hazardous to the systems and to humans".

Source: Hawksley, 1985

Appendix B

1. Hydraulic and roughness coefficient design

2. Spreadsheet details

APPENDIX B - 1

"In 1939 Colebrook studied the frictional resistance of commercial pipes, and the equation he derived, in conjunction with White, provided an accurate basis for hydraulic design which has had experimental confirmation over very wide conditions of flow, the general form of the equation is:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left[\frac{K_s}{3.7D} + \frac{2.51}{Re\sqrt{\lambda}} \right]$$

where

λ = friction coefficient $2g\ DS/V^2$

K_2 = a linear measure of effective roughness

D_2 = Pipe Diameter

Re_2 = Reynolds Number, VD/ν

In engineering terms, the expression for pipe flow be written:-

$$V = -2\sqrt{(2g\ DS)} \log \left[\frac{K_s}{3.7D} + \frac{2.51\nu}{D\sqrt{(2g\ DS)}} \right]$$

where

V = Velocity (m/s)

g = gravitational acceleration

S = hydraulic gradient: headloss per unit length a - Water surface shape in uniform flow in open channel

ν = kinetic viscosity of fluid

For open-channel flow, this may be expressed in terms of the hydraulic mean depth:

$$V = -\sqrt{(32gRS)}\ \log\left[\frac{k_s}{14.8R} + \frac{1.255\nu}{R\sqrt{32gRS}}\right]$$

where

R = hydraulic mean depth (area/wetted perimeter)

The application of this rather complex formula to practical problems in the hydraulic design of channels and pipes was simplified with the derivation of a set of charts at the then Hydraulics Research Station, which are published in Ref. 1. Ref.1 also includes worked examples of the use of the charts. However, there were many requests from engineers engaged in the design of pipes for a tabulation of the discharges and velocities in pipes over a range of diameters, hydraulic gradients and roughnesses appropriate to such work: the present publication gives such tables covering the following conditions:

Diameters	50mm to 2100mm
Gradients	0.0-001 to 0.1 (ie 1 in 10,000 to 1 in 10)
Roughnesses	k_s values from 0.006 to 15mm

INTERPOLATION

The tables indicate discharges and velocities for nominal metric pipe sizes. If the actual size of the pipe is within 5 per cent of the nearest nominal size discharges and velocities may be corrected according to the following expressions:

where

Q_a, Q_n, V_a, D_a and D_n are the actual and nominal; discharges, velocities and diameters respectively.

The exponent x depends on the Reynolds number Re, and the roughness ratio D/K_s as follows:-

Re D/k_s	10^5	5×10^5	10^6	5×10^6	10^7
≤50	2.72	2.72	2.72	2.72	2.72
100	2.72	2.72	2.71	2.70	2.66
500	2.72	2.71	2.70	2.64	2.64
1000	2.72	2.70	2.64	2.62	2.62
≥5000	2.72	2.66	2.64	2.61	2.60

If the actual size is not within 5 per cent of the nearest nominal size reference should be made to Charts for the hydraulic design of channels and pipes, Ref.1.

The linear measure of surface roughness

The pipes used by Nikuradse in his experiments were roughened by means of a uniformly-graded sand glued to the surface, and the diameter of those grains has

provided a standard of comparison again which other surfaces may be judged, their roughness thus being given as the size of sand which, as a uniform coating, would give the same resistance under rough turbulent flow. Such k_s values are available for most of the surfaces likely to be used in drainage works, and are listed in Table 38, which is based on recommendations made by Colebrook[2], Rouse[3], King[4], Lamont[5] and Perkins and Gardiner[6].

The Colebrook-White equation is most appropriate where the roughness consists of separate protuberances of random height and spacing, but some classes of pipe, for instance corrugated metal culverts, not having this form of roughness, do not follow the same resistance function in the transition zone between smooth-turbulence and rough-turbulence, and are therefore not included. The surfaces listed are classified as good, normal and poor examples of their respective categories, thus leaving to the engineer's judgement the actual value to be used in any particular scheme. The range of roughness covered by good and poor, takes account not only of the quality of the jointing, but also the variation in surface roughness to be found in pipes that are nominally of the same material. This list should not be taken as absolving the engineer of the responsibility for checking the actual surface roughness achieved in particular projects by precise hydraulic tests whenever possible. Where such direct evidence is available from comparable projects it should clearly take precedence over the general roughness values quoted here.

Surfaces of composite roughness

If the wetted perimeter of a channel or pipe is composed of surfaces of dissimilar roughness, for example where sliming has taken place only below the usual water level or where the invert is of concrete and the walls of brick, there are two ways of computing discharges using the tables in this report.

The first method is based on the concept of an equivalent grain roughness for the whole perimeter and utilises the expression

$$k_s = p_1 \, k_{s1} + p_2 \, k_{s2}$$

where p_1, p_2 denote the proportions of the total perimeter occupied by surfaces 1 and 2, and k_{s1}, k_{s2} denote the equivalent sand grain roughness of surfaces 1 and 2.

This method may be used in situations where the difference in roughness values is not excessive and where the two surfaces occupy similar proportions of the total perimeter. It will also give approximate answers outside these ranges. These ranges can be defined (somewhat arbitrarily) as:

$$0.05 < k_{s1}/k_{s2} < 20 \text{ and } 0.33 < p_1/p_2 < 3.0$$

The advantage of the above method is that it provides a direct solution to the problem of composite roughness and the tables can be used without the need to resort to successive approximate techniques.

The second method is based on the concept of an equivalent friction factor for the whole perimeter and utilises the expression

$$\lambda_s = p_1 \lambda_1 + p_2 \lambda_2$$

where the surface denote surfaces 1 and 2 as before. The method uses an approximation to the Colebrook and White equation suggested in Barr[7].

The method given below is not restricted to the limited ranges of conditions of the first method but, even so, errors will occur where there are large differences in the grain roughness of the two surfaces, say $k_{s1}/k_{s2} < 0.01$ or $k_{s1}/k_{s2} > 100$. The method requires a successive approximation approach as follows:

(i) Determine an approximate value for k_s using:

$$k_s = p_1 k_{s1} + p_2 k_{s2}$$

(ii) Use the appropriate table to determine the flow, Q

(iii) Calculate Re

(iv) Calculate $C_1 = \dfrac{k_{s1}}{14.8R} \dfrac{5.1286}{Re^{0.89}}$

(v) Calculate $C_2 = \dfrac{K_{s2}}{14.8R} \dfrac{5.1286}{Re^{0.89}}$

(vi) Determine C from: $\log C = \dfrac{\log C_1}{(p1 + (\log C_1/\log C_2)^2 p_2)^{1/2}}$

(vii) Calculate $k_s = 14.8R\left(C - \dfrac{5.1286}{Re^{0.89}}\right)$

(viii) Use the appropriate table to determine the flow,Q

(ix) Repeat (iii) to (viii) until Q changes by less than the required tolerance.

Other sources of resistance

When designing a pipe-line on the basis of its full-bore capacity, allowance must be made for head losses which will occur at bends, manholes or other appurtenances involving changes in cross-section. Similar allowances should also be made under conditions of free-surface flow. The customary practice is to take these into account as an equivalent additional length of pipe, and, where such losses are a small part of the whole resistance, this is a satisfactory procedure. Hydraulic handbooks, manuals and other technical publications give head-loss coefficients in the expression $h_1 = V^2/2g$. Head losses at straight through open-channel manholes are generally small, the head loss coefficients being of the following approximate magnitudes:

	Part-full	**Full-bore**
Open-channel manhole	< 0.1	0.05-0.25
Open-channel manhole with bend	- 0.3	- 1.5
Open-channel manhole with pipe bend beyond manhole	- 0.3	- 0.3

If a manhole incorporates a junction, losses are much higher and depend on the relative magnitudes of the flows in the branches and the geometry of the junction.

In addition, there is some loss of head at the entry to a pipe, which will depend upon the sharpness of the arris, and furthermore, the kinetic energy of flow ($V^2/2g$) is generally not recoverable at the exit; these factors may well prove important if

the pipe or conduit is relatively short. The head loss with a sharp-edged re-entrant inlet is approximately $V^2/2g$, with a flush headwall the loss coefficient drops to about 0.4, whilst a rounding as little as a sixth or seventh of the pipe diameter will almost eliminate the entrance loss.

In short culverts and conduits at hydraulically steep slopes, separation may occur at the inlet if it is square-edged; the inlet then acts as a controlling section and precludes full-bore operation. The friction-loss approach to pipe-line design is not generally applicable to short culverts and conduits.

Part-full pipes

Sometimes, having designed a gravity pipeline for a certain discharge flowing full, the depth of flow for some lesser discharge is required. Alternatively, the designer may wish to know the velocities and discharges at particular depths of flow within the pipe. These problems are readily solves using the coefficient for part-full pipes listed at the base of each column in Tables 1 to 33 and the data provided in Tables 35 and 36.

The method is to determine the pipe-full discharge or velocity using the appropriate table and to read off the part-full coefficient at the base of the column. Table 35 then gives proportional velocities in terms of proportional depths and Table 36 gives proportional discharges in terms of proportional depths, for circular and egg-shaped sections. Table 37 gives the geometrical functions relating to part-full circular and egg-shaped sections and Table 38 gives recommended roughness

values of a range of pipes and channels".

Examples

1. A circular section concrete water supply pipe of 500mm diameter is to be laid to a slope of 0.006. Assuming that the hydraulic gradient is equal to the slope of the pipe, determine (a) the full-bore discharge, and (b) the velocity of flow when the depth is 225mm.

Table 38 indicates an appropriate roughness value k_s of 0.15mm. Hence the flow data given in Table 16 is appropriate.

(a) Reading from Table 16 the full-bore discharge is 0.379 m^3/s.

(b) Table 16 gives a full-bore velocity of 1.931m/s and a coefficient for part-full pipes, 0, of 250. At a depth of 225mm the proportional depth is 225/500 = 0.45. Table 35(a) gives a proportional velocity of 0.956 (interpolating between adjacent columns). Hence the velocity of flow when the depth is 225mm is 0.956 x 1.931 = 1.846m/s.

Source: Wallingford, 1990

Table B-1

k	R	h	Aw	Qp	Vel
0.0200	0.0015	0.075	0.1125	0.0163078	0.0202	1.240099
0.0143	0.0015	0.075	0.1125	0.0163078	0.0171	1.04768
0.0167	0.0015	0.075	0.1125	0.0163078	0.0185	1.132668
0.0125	0.0015	0.075	0.1125	0.0163078	0.0160	0.97914
0.0100	0.0015	0.075	0.1125	0.0163078	0.0143	0.875146
0.0080	0.0015	0.075	0.1125	0.0163078	0.0128	0.782134
0.0070	0.0015	0.075	0.1125	0.0163078	0.0119	0.731241
0.0063	0.0015	0.075	0.1125	0.0163078	0.0113	0.693416
0.0055	0.0015	0.075	0.1125	0.0163078	0.0106	0.647513
0.0050	0.0015	0.075	0.1125	0.0163078	0.0101	0.61711
0.0200	0.0015	0.100	0.1500	0.0289916	0.0436	1.502868
0.0167	0.0015	0.100	0.1500	0.0289916	0.0398	1.37278
0.0142	0.0015	0.100	0.1500	0.0289916	0.0367	1.265399
0.0125	0.0015	0.100	0.1500	0.0289916	0.0344	1.186872
0.0100	0.0015	0.100	0.1500	0.0289916	0.0308	1.060944
0.0080	0.0015	0.100	0.1500	0.0289916	0.0275	0.948313
0.0070	0.0015	0.100	0.1500	0.0289916	0.0257	0.886685
0.0060	0.0015	0.100	0.1500	0.0289916	0.0238	0.820475
0.0050	0.0015	0.100	0.1500	0.0289916	0.0217	0.748478
0.0040	0.0015	0.100	0.1500	0.0289916	0.0194	0.668844
0.0035	0.0015	0.100	0.1500	0.0289916	0.0181	0.625272
0.0200	0.0015	0.150	0.2250	0.0652312	0.1280	1.962813
0.0167	0.0015	0.150	0.2250	0.0652312	0.1170	1.793068
0.0143	0.0015	0.150	0.2250	0.0652312	0.1082	1.658782
0.0125	0.0015	0.150	0.2250	0.0652312	0.1011	1.550484
0.0100	0.0015	0.150	0.2250	0.0652312	0.0904	1.386165
0.0080	0.0015	0.150	0.2250	0.0652312	0.0808	1.239195
0.0070	0.0015	0.150	0.2250	0.0652312	0.0756	1.158778
0.0060	0.0015	0.150	0.2250	0.0652312	0.0700	1.07238
0.0050	0.0015	0.150	0.2250	0.0652312	0.0638	0.97843
0.0040	0.0015	0.150	0.2250	0.0652312	0.0570	0.874514
0.0030	0.0015	0.150	0.2250	0.0652312	0.0494	0.756568
0.0020	0.0015	0.150	0.2250	0.0652312	0.0402	0.616672
0.0200	0.0015	0.200	0.3000	0.1159665	0.2744	2.36639
0.0142	0.0015	0.200	0.3000	0.1159665	0.2311	1.993008
0.0167	0.0015	0.200	0.3000	0.1159665	0.2507	2.161848
0.0090	0.0015	0.200	0.3000	0.1159665	0.1839	1.585446
0.0080	0.0015	0.200	0.3000	0.1159665	0.1733	1.494432
0.0070	0.0015	0.200	0.3000	0.1159665	0.1621	1.397528
0.0060	0.0015	0.200	0.3000	0.1159665	0.1500	1.293418
0.0040	0.0015	0.200	0.3000	0.1159665	0.1223	1.054985
0.0030	0.0015	0.200	0.3000	0.1159665	0.1059	0.912855
0.0020	0.0015	0.200	0.3000	0.1159665	0.0863	0.744272
0.0015	0.0015	0.200	0.3000	0.1159665	0.0747	0.643784
0.0013	0.0015	0.200	0.3000	0.1159665	0.0695	0.598933
0.0125	0.0015	0.250	0.3750	0.1811977	0.3912	2.158785
0.0100	0.0015	0.250	0.3750	0.1811977	0.3498	1.930242
0.0070	0.0015	0.250	0.3750	0.1811977	0.2924	1.613978
0.0050	0.0015	0.250	0.3750	0.1811977	0.2470	1.363138
0.0040	0.0015	0.250	0.3750	0.1811977	0.2208	1.218601
0.0030	0.0015	0.250	0.3750	0.1811977	0.1911	1.054547
0.0020	0.0015	0.250	0.3750	0.1811977	0.1558	0.859958
0.0018	0.0015	0.250	0.3750	0.1811977	0.1478	0.815528
0.0017	0.0015	0.250	0.3750	0.1811977	0.1436	0.792387
0.0012	0.0015	0.250	0.3750	0.1811977	0.1205	0.664813
0.0011	0.0015	0.250	0.3750	0.1811977	0.1153	0.636265
0.0010	0.0015	0.250	0.3750	0.1811977	0.1099	0.606388

"$V_2 - 2\sqrt{(2g\ D$

where

V = Veloc
g = gravi
S = hydra
v = Kiner
K_s = Rough
(Hydraulics R

"Aw = 3.14

where

Q = $\cos^{-1}[$

b = $2[r^2-(r$

(Bowker and S

QpΣ VAw

PIPE FLOWING AT A PROPORTIONAL DEPTH OF 0.75

k_s = 1.5m/s to 1.0m/s
k_s = 0.6m/s less than or equal to 1.5m/s
k_s = 0.3m/s

450mm to 1000mm Nominal Bore

Table B-2

Nominal Bore mm		100		150		800		900		1000	
Hydraulic Gradient	1 in	velocity m/s	discharge l/s	velocity m/s	discharge l/s	velocity m/s	discharge l/s	velocity m/s	discharge l/s	velocity m/s	discharge l/s
.0010	1000.0	0.23	1.47	0.31	4.63	0.93	374.47	1.00	510.75	1.18	747.57
.0011	909.1	0.24	1.55	0.32	4.76	0.97	392.97	1.16	595.08	1.24	784.54
.0012	833.3	0.26	1.62	0.34	4.88	1.13	456.50	1.22	621.90	1.30	819.86
.0013	769.2	0.27	1.68	0.35	5.00	1.18	475.40	1.27	647.61	1.35	853.74
.0014	714.3	0.28	1.75	0.37	5.47	1.22	493.58	1.31	672.36	1.40	886.34
.0015	666.7	0.29	1.81	0.38	5.59	1.26	511.12	1.36	696.24	1.45	917.79
.0016	625.0	0.30	1.87	0.39	5.71	1.31	528.09	1.41	719.34	1.61	1014.83
.0017	588.2	0.31	1.93	0.40	5.83	1.35	544.54	1.45	741.72	1.66	1046.63
.0018	555.6	0.31	1.99	0.42	5.94	1.39	560.51	1.49	763.46	1.71	1077.50
.0019	526.3	0.32	2.04	0.43	6.05	1.42	576.04	1.64	840.61	1.75	1107.53
.0020	500.0	0.33	2.10	0.44	6.16	1.46	591.17	1.69	862.83	b.80	1136.79
.0022	454.5	0.35	2.20	0.46	6.36	1.65	665.31	1.77	905.68	1.89	1193.18
.0024	416.7	0.36	2.30	0.48	6.56	1.72	695.42	1.85	946.62	1.97	1247.08
.0026	384.6	0.38	2.40	0.50	7.10.47	1.79	724.30	1.93	985.89	2.06	1298.77
.0028	357.1	0.39	2.49	0.52	7.30.09	1.86	752.09	2.00	1023.69	2.13	1348.51
.0030	333.3	0.41	2.58	0.54	7.49.01	1.93	778.91	2.07	1060.16	2.21	1396.51
.0032	312.5	0.42	2.66	0.56	7.67.32	1.99	804.86	2.14	1095.43	2.28	1442.94
.0034	294.1	0.43	2.75	0.57	8.85.06	2.05	830.00	2.21	1129.62	2.36	1487.94
.0036	277.8	0.45	2.83	0.59	8.02.29	2.11	854.42	2.27	1162.82	2.42	1531.64
.0038	263.2	0.46	2.91	0.61	8.19.05	2.17	878.16	2.34	1195.11	2.49	1574.14
.0040	250.0	0.47	2.98	0.62	8.35.37	2.23	901.30	2.40	1226.56	2.56	1615.54
.0042	238.1	0.48	3.06	0.64	9.51.29	2.28	923.86	2.46	1257.24	2.62	1655.91
.0044	227.3	0.50	3.13	0.65	9.66.84	2.34	945.89	2.52	1287.20	2.68	1695.34
.0046	217.4	0.51	3.20	0.67	9.82.04	2.39	967.42	2.57	1316.48	2.74	1733.88
.0048	208.3	0.52	3.27	0.68	9.96.91	2.44	988.50	2.63	1345.14	2.80	1771.60
.0050	200.0	0.53	3.34	0.70	9.11.48	2.50	1009.14	2.68	1373.20	2.86	1808.53
.0055	181.8	0.55	3.51	0.73	10.46.67	2.62	1059.00	2.82	1441.00	3.00	1897.77
.0060	166.7	0.58	3.66	0.76	10.80.29	2.74	1106.65	2.94	1505.79	3.14	1983.04
.0065	153.8	0.60	3.81	0.80	11.12.54	2.85	1152.35	3.06	1567.93	3.27	2064.82
.0070	142.9	0.63	3.96	0.83	11.43.58	2.96	1196.33	3.18	1627.72	3.39	2143.52
.0075	133.3	0.65	4.10	0.86	12.73.53	3.06	1238.76	3.29	1685.42	3.51	2219.46
.0080	125.0	0.67	4.24	0.88	12.02.49	3.17	1279.81	3.40	1741.22	3.63	2292.91
.0085	117.6	0.69	4.37	0.91	12.30.57	3.26	1319.59	3.51	1795.31	3.74	2364.09
.0090	111.1	0.71	4.50	0.94	13.57.83	3.36	1358.22	3.61	1847.83	3.85	2433.22
.0095	105.3	0.73	4.62	0.96	13.84.34	3.45	1395.79	3.71	1898.91	3.96	2500.45
.0100	100.0	0.75	4.74	0.99	14.10.17	3.54	1432.38	3.81	1948.67	4.06	2565.93
.0110	90.9	0.79	4.97	1.19	16.59.95	3.72	1502.92	4.00	2044.58	4.26	2692.16
.0120	83.3	0.82	5.20	1.24	17.07.52	3.88	1570.33	4.17	2136.23	4.45	2812.78
.0130	76.9	0.86	5.41	1.29	18.53.15	4.04	1634.98	4.35	2224.13	4.64	2928.47
.0140	71.4	0.89	5.62	1.34	19.97.05	4.20	1697.19	4.51	2308.71	4.81	3039.79
.0150	66.7	0.92	5.82	1.39	19.39.42	4.35	1757.21	4.67	2390.33	4.98	3147.20
.0160	62.5	0.95	6.01	1.44	20.80.39	4.49	1815.27	4.83	2469.27	5.15	3251.09
.0170	58.8	0.98	6.19	1.48	21.20.11	4.63	1871.54	4.97	2545.77	5.31	3351.78
.0180	55.6	1.17	7.38	1.67	23.58.67	4.76	1926.18	5.12	2620.06	5.46	3449.55
.0190	52.6	1.20	7.59	1.71	24.96.17	4.90	1979.33	5.26	2692.32	5.61	3544.65
.0200	50.0	1.23	7.79	1.76	25.32.71	5.02	2031.09	5.40	2762.69	5.76	3637.27
.0220	45.5	1.29	8.17	1.85	26.70.41	5.27	2130.87	5.66	2898.36	6.04	3815.81
.0240	41.7	1.35	8.54	1.93	27.34.95	5.51	2226.21	5.92	3027.98	6.31	3986.41
.0260	38.5	1.41	8.89	2.01	28.97.05	5.73	2317.65	6.16	3152.31	6.57	4150.04
.0280	35.7	1.46	9.23	2.09	29.56.97	5.95	2405.64	6.39	3271.94	6.82	4307.49
.0300	33.3	1.66	10.50	2.16	30.14.93	6.16	2490.54	6.62	3387.38	7.06	4459.41
.0320	31.2	1.72	10.85	2.23	31.71.10	6.36	2572.66	6.84	3499.02	7.29	4606.34
.0340	29.4	1.77	11.19	2.30	32.25.64	6.56	2652.25	7.05	3607.23	7.52	4748.76
.0360	27.8	1.82	11.52	2.37	33.78.69	6.75	2729.53	7.25	3712.31	7.74	4887.04
.0380	26.3	1.87	11.84	2.44	34.30.36	6.94	2804.69	7.45	3814.50	7.95	5021.53
.0400	25.0	1.92	12.16	2.50	35.80.75	7.12	2877.90	7.65	3914.03	8.16	5152.53
.0420	23.8	1.97	12.46	2.56	36.29.96	7.29	2949.30	7.84	4011.11	8.36	5280.29
.0440	22.7	2.02	12.76	2.62	37.78.06	7.47	3019.02	8.02	4105.90	8.56	5405.05
.0460	21.7	2.07	13.05	2.68	38.25.12	7.63	3087.18	8.20	4198.57	8.75	5527.00
.0480	20.8	2.11	13.33	2.74	38.71.22	7.80	3153.87	8.38	4289.24	8.94	5646.33
.0500	20.0	2.15	13.61	2.80	39.82.58	7.96	3219.18	8.55	4378.04	9.12	5763.20
.0550	18.2	2.26	14.29	2.94	41.88.98	8.35	3376.96	8.97	4592.56	9.57	6045.52
.0600	16.7	2.36	14.93	3.07	43.91.03	8.72	3527.72	9.37	4797.53	10.00	6315.29
.0650	15.4	2.46	15.55	3.20	45.89.23	9.08	3672.32	9.76	4994.13	10.41	6574.02
.0700	14.3	2.56	16.14	3.32	47.83.98	9.43	3811.46	10.13	5183.30	10.80	6822.99
.0750	13.3	2.65	16.72	3.43	48.75.63	9.76	3945.71	10.49	5365.83	11.18	7063.21
.0800	12.5	2.73	17.27	3.55	50.64.45	10.08	4075.56	10.83	5542.37	11.55	7295.55
.0850	11.8	2.82	17.81	3.66	52.50.69	10.39	4201.41	11.16	5713.48	11.90	7520.74
.0900	11.1	2.90	18.33	3.77	53.34.58	10.69	4323.61	11.49	5879.62	12.25	7739.39
.0950	10.5	2.98	18.84	3.87	55.15	10.99	4442.46	11.80	6041.21	12.59	7952.06

Table B-3

i	k	R	h	Aw	Pw	Qp	Vel	u	o	x
0.0200	0.0015	0.075	0.01	0.00218	0.123	0.0016	0.7419	0.0024	0.037417	0.0
0.0167	0.0015	0.075	0.012	0.0025	0.135	0.0017	0.6983	0.0026	0.040694	0.0
0.0143	0.0015	0.075	0.017	0.00331	0.162	0.0023	0.6919	0.0028	0.04755	0.0
0.0125	0.0015	0.075	0.023	0.0043	0.190	0.0030	0.6944	0.0028	0.054046	0.1
0.0100	0.0015	0.075	0.035	0.00637	0.237	0.0044	0.6964	0.0025	0.063443	0.1
0.0080	0.0015	0.075	0.05	0.0091	0.290	0.0063	0.6932	0.0018	0.070711	0.1
0.0070	0.0015	0.075	0.062	0.01137	0.329	0.0079	0.6918	0.001	0.073865	0.1
0.0063	0.0015	0.075	0.075	0.01387	0.370	0.0096	0.693	0	0.075	
0.0055	0.0015	0.075	0.1	0.01864	0.450	0.0129	0.692	-0.002	0.070711	0.1
0.0050	0.0015	0.075	0.125	0.02311	0.542	0.0155	0.6726	-0.003	0.055902	0.1
0.0142	0.0015	0.1	0.01	0.00316	0.142	0.0023	0.7322	0.0039	0.043589	0.0
0.0125	0.0015	0.1	0.011	0.00338	0.149	0.0023	0.6951	0.0041	0.045596	0.0
0.0100	0.0015	0.1	0.022	0.00573	0.212	0.0040	0.6999	0.0049	0.062578	0.1
0.0080	0.0015	0.1	0.035	0.00861	0.271	0.0060	0.6988	0.0049	0.075993	0
0.0070	0.0015	0.1	0.043	0.01046	0.303	0.0072	0.6912	0.0047	0.082164	0.1
0.0060	0.0015	0.1	0.056	0.01356	0.350	0.0094	0.6909	0.004	0.0898	0.1
0.0050	0.0015	0.1	0.078	0.01903	0.424	0.0133	0.6965	0.0021	0.09755	0.1
0.0040	0.0015	0.1	0.118	0.02927	0.550	0.0204	0.6968	-0.002	0.098367	0.1
0.0035	0.0015	0.1	0.15	0.03721	0.658	0.0252	0.6785	-0.004	0.086603	0.1
0.0080	0.0015	0.15	0.01	0.00543	0.173	0.0038	0.6933	0.0075	0.053852	0.1
0.0070	0.0015	0.15	0.018	0.00808	0.233	0.0056	0.6929	0.0094	0.071246	0.1
0.0060	0.0015	0.15	0.03	0.01193	0.303	0.0083	0.6987	0.0108	0.09	
0.0050	0.0015	0.15	0.045	0.01685	0.375	0.0117	0.6969	0.0112	0.107121	0.1
0.0040	0.0015	0.15	0.068	0.02476	0.467	0.0172	0.6946	0.0103	0.125603	0.1
0.0030	0.0015	0.15	0.11	0.04017	0.613	0.0278	0.692	0.0058	0.144568	0.1
0.0025	0.0015	0.15	0.155	0.05742	0.756	0.0399	0.6957	-7E-04	0.149917	0.1
0.0060	0.0015	0.2	0.01	0.00808	0.199	0.0058	0.7122	0.0119	0.06245	0.1
0.0050	0.0015	0.2	0.02	0.01263	0.283	0.0088	0.693	0.0157	0.087178	0.1
0.0040	0.0015	0.2	0.04	0.02121	0.404	0.0146	0.6904	0.0192	0.12	
0.0030	0.0015	0.2	0.075	0.03673	0.562	0.0254	0.6902	0.0195	0.156125	0.1
0.0020	0.0015	0.2	0.155	0.07562	0.844	0.0524	0.6925	0.0088	0.194872	0.1
0.0019	0.0015	0.2	0.17	0.08326	0.892	0.0577	0.693	0.0059	0.197737	0.1
0.0015	0.0015	0.2	0.29	0.14403	1.280	0.1001	0.6947	-0.016	0.178606	0.1

Table B-3

0.0040	0.0015	0.25	0.018	0.01585	0.300	0.0110	0.694	0.0216	0.093145	0.1863	(
0.0030	0.0015	0.25	0.05	0.03314	0.505	0.0229	0.6924	0.03	0.15	0.3	(
0.0020	0.0015	0.25	0.12	0.07271	0.804	0.0507	0.6968	0.0278	0.213542	0.4271	(
0.0018	0.0015	0.25	0.145	0.08778	0.893	0.0612	0.6976	0.0238	0.226881	0.4538	(
0.0017	0.0015	0.25	0.16	0.09701	0.944	0.0677	0.6975	0.021	0.233238	0.4665	(
0.0012	0.0015	0.25	0.3	0.18614	1.391	0.1292	0.6942	-0.012	0.244949	0.4899	
0.0011	0.0015	0.25	0.39	0.24146	1.700	0.1667	0.6905	-0.029	0.207123	0.4142	(

Table B-4

i	k	R	h	Aw	Pw	Qp	Vel
0.0200	0.0015	0.075	0.1125	0.02093	0.493	0.0282	1.3473
0.0143	0.0015	0.075	0.1125	0.02093	0.493	0.0238	1.1382
0.0167	0.0015	0.075	0.1125	0.02093	0.493	0.0258	1.2306
0.0125	0.0015	0.075	0.1125	0.02093	0.493	0.0223	1.0637
0.0100	0.0015	0.075	0.1125	0.02093	0.493	0.0199	0.9507
0.0080	0.0015	0.075	0.1125	0.02093	0.493	0.0178	0.8496
0.0070	0.0015	0.075	0.1125	0.02093	0.493	0.0166	0.7943
0.0063	0.0015	0.075	0.1125	0.02093	0.493	0.0158	0.7532
0.0055	0.0015	0.075	0.1125	0.02093	0.493	0.0147	0.7033
0.0050	0.0015	0.075	0.1125	0.02093	0.493	0.0140	0.6703
0.0200	0.0015	0.1	0.15	0.03721	0.658	0.0607	1.6313
0.0167	0.0015	0.1	0.15	0.03721	0.658	0.0554	1.49
0.0142	0.0015	0.1	0.15	0.03721	0.658	0.0511	1.3735
0.0125	0.0015	0.1	0.15	0.03721	0.658	0.0479	1.2882
0.0100	0.0015	0.1	0.15	0.03721	0.658	0.0428	1.1515
0.0080	0.0015	0.1	0.15	0.03721	0.658	0.0383	1.0292
0.0070	0.0015	0.1	0.15	0.03721	0.658	0.0358	0.9623
0.0060	0.0015	0.1	0.15	0.03721	0.658	0.0331	0.8904
0.0050	0.0015	0.1	0.15	0.03721	0.658	0.0302	0.8122
0.0040	0.0015	0.1	0.15	0.03721	0.658	0.0270	0.7258
0.0035	0.0015	0.1	0.15	0.03721	0.658	0.0252	0.6785
0.0200	0.0015	0.15	0.225	0.08373	0.986	0.1782	2.128
0.0167	0.0015	0.15	0.225	0.08373	0.986	0.1628	1.944
0.0143	0.0015	0.15	0.225	0.08373	0.986	0.1506	1.7983
0.0125	0.0015	0.15	0.225	0.08373	0.986	0.1407	1.6809
0.0100	0.0015	0.15	0.225	0.08373	0.986	0.1258	1.5027
0.0080	0.0015	0.15	0.225	0.08373	0.986	0.1125	1.3434
0.0070	0.0015	0.15	0.225	0.08373	0.986	0.1052	1.2562
0.0060	0.0015	0.15	0.225	0.08373	0.986	0.0973	1.1625
0.0050	0.0015	0.15	0.225	0.08373	0.986	0.0888	1.0606
0.0040	0.0015	0.15	0.225	0.08373	0.986	0.0794	0.9479
0.0030	0.0015	0.15	0.225	0.08373	0.986	0.0687	0.82
0.0020	0.0015	0.15	0.225	0.08373	0.986	0.0560	0.6683
0.0090	0.0015	0.2	0.3	0.14885	1.315	0.2556	1.7175
0.0080	0.0015	0.2	0.3	0.14885	1.315	0.2410	1.6188
0.0070	0.0015	0.2	0.3	0.14885	1.315	0.2253	1.5139
0.0060	0.0015	0.2	0.3	0.14885	1.315	0.2085	1.401
0.0050	0.0015	0.2	0.3	0.14885	1.315	0.1903	1.2784
0.0040	0.0015	0.2	0.3	0.14885	1.315	0.1701	1.1427
0.0030	0.0015	0.2	0.3	0.14885	1.315	0.1472	0.9887
0.0020	0.0015	0.2	0.3	0.14885	1.315	0.1200	0.806
0.0019	0.0015	0.2	0.3	0.14885	1.315	0.1169	0.7855
0.0015	0.0015	0.2	0.3	0.14885	1.315	0.1038	0.6972
0.0014	0.0015	0.2	0.3	0.14885	1.315	0.1002	0.6733
0.0013	0.0015	0.2	0.3	0.14885	1.315	0.0965	0.6486
0.0125	0.0015	0.25	0.375	0.23258	1.644	0.5436	2.3374
0.0100	0.0015	0.25	0.375	0.23258	1.644	0.4861	2.0899
0.0070	0.0015	0.25	0.375	0.23258	1.644	0.4064	1.7474
0.0050	0.0015	0.25	0.375	0.23258	1.644	0.3432	1.4758
0.0040	0.0015	0.25	0.375	0.23258	1.644	0.3068	1.3192
0.0030	0.0015	0.25	0.375	0.23258	1.644	0.2655	1.1416

NABEEL.XLS

Table B-4

0.0020	0.0015	0.25	0.375	0.23258	1.644	0.2165	0.9309
0.0018	0.0015	0.25	0.375	0.23258	1.644	0.2053	0.8828
0.0017	0.0015	0.25	0.375	0.23258	1.644	0.1995	0.8577
0.0012	0.0015	0.25	0.375	0.23258	1.644	0.1673	0.7195
0.0011	0.0015	0.25	0.375	0.23258	1.644	0.1602	0.6886
0.0010	0.0015	0.25	0.375	0.23258	1.644	0.1526	0.6563

Table B-5

Gradient	Pipe diameters in mm: 50	75	80	100	125	150	175	200	225	250	275	300
0.00075	0.108	0.145	0.152	0.178	0.208	0.236	0.262	0.286	0.310	0.333	0.354	0.375
1/ 1333	0.212	0.641	0.764	1.397	2.550	4.163	6.295	8.999	12.327	16.329	21.051	26.539
0.00080	0.112	0.150	0.157	0.184	0.215	0.244	0.270	0.296	0.320	0.344	0.366	0.388
1/ 1250	0.219	0.663	0.790	1.444	2.636	4.303	6.505	9.299	12.739	16.873	21.752	27.422
0.00085	0.115	0.155	0.162	0.190	0.222	0.251	0.279	0.305	0.330	0.354	0.378	0.400
1/ 1176	0.226	0.684	0.815	1.490	2.719	4.438	6.710	9.591	13.137	17.401	22.432	28.278
0.00090	0.119	0.159	0.167	0.195	0.228	0.259	0.287	0.314	0.340	0.365	0.389	0.412
1/ 1111	0.233	0.704	0.839	1.534	2.800	4.570	6.908	9.874	13.524	17.913	23.092	29.109
0.00095	0.122	0.164	0.172	0.201	0.235	0.266	0.295	0.323	0.350	0.375	0.400	0.423
1/ 1053	0.240	0.724	0.863	1.578	2.879	4.698	7.101	10.149	13.901	18.412	23.734	29.918
0.00100	0.125	0.168	0.176	0.206	0.241	0.273	0.303	0.332	0.359	0.385	0.410	0.434
1/ 1000	0.246	0.744	0.886	1.620	2.955	4.822	7.289	10.417	14.268	18.897	24.359	30.705
0.00110	0.132	0.177	0.185	0.217	0.253	0.286	0.318	0.348	0.377	0.404	0.430	0.456
1/ 909	0.259	0.781	0.930	1.701	3.102	5.062	7.651	10.934	14.975	19.333	25.564	32.223
0.00120	0.138	0.185	0.194	0.226	0.264	0.299	0.332	0.364	0.394	0.422	0.450	0.476
1/ 833	0.271	0.817	0.973	1.778	3.243	5.292	7.997	11.428	15.651	20.727	26.715	33.674
0.00130	0.144	0.193	0.202	0.236	0.275	0.312	0.346	0.379	0.410	0.440	0.468	0.496
1/ 769	0.282	0.851	1.014	1.853	3.379	5.512	8.329	11.902	16.299	21.584	27.820	35.065
0.00140	0.149	0.200	0.209	0.245	0.286	0.324	0.360	0.393	0.426	0.457	0.486	0.515
1/ 714	0.293	0.884	1.053	1.924	3.509	5.723	8.648	12.358	16.923	22.410	28.883	36.404
0.00150	0.155	0.207	0.217	0.254	0.296	0.335	0.372	0.407	0.441	0.473	0.504	0.533
1/ 667	0.304	0.916	1.091	1.993	3.634	5.928	8.957	12.798	17.525	23.206	29.908	37.696
0.00160	0.160	0.214	0.224	0.262	0.306	0.347	0.385	0.421	0.455	0.488	0.520	0.551
1/ 625	0.314	0.947	1.127	2.060	3.755	6.125	9.255	13.223	18.107	23.976	30.900	38.946
0.00170	0.165	0.221	0.231	0.271	0.316	0.357	0.397	0.434	0.470	0.504	0.536	0.568
1/ 588	0.324	0.977	1.163	2.125	3.873	6.317	9.544	13.636	18.671	24.723	31.862	40.157
0.00180	0.170	0.228	0.238	0.279	0.325	0.368	0.408	0.447	0.483	0.518	0.552	0.585
1/ 556	0.334	1.006	1.198	2.187	3.987	6.503	9.824	14.036	19.219	25.447	32.795	41.333
0.00190	0.175	0.234	0.245	0.286	0.334	0.378	0.420	0.459	0.497	0.533	0.567	0.601
1/ 526	0.343	1.034	1.231	2.249	4.099	6.684	10.097	14.426	19.752	26.152	33.703	42.476
0.00200	0.180	0.240	0.251	0.294	0.343	0.388	0.431	0.471	0.510	0.547	0.582	0.617
1/ 500	0.353	1.061	1.264	2.308	4.207	6.860	10.363	14.805	20.271	26.839	34.588	43.590
0.00220	0.189	0.252	0.264	0.308	0.360	0.407	0.452	0.495	0.535	0.574	0.611	0.647
1/ 455	0.370	1.114	1.327	2.423	4.415	7.200	10.876	15.537	21.271	28.163	36.293	45.738
0.00240	0.197	0.264	0.276	0.322	0.376	0.426	0.473	0.517	0.559	0.599	0.638	0.676
1/ 417	0.387	1.165	1.387	2.533	4.615	7.524	11.365	16.235	22.227	29.428	37.922	47.790
0.00260	0.205	0.275	0.287	0.336	0.392	0.443	0.492	0.538	0.582	0.624	0.665	0.704
1/ 385	0.403	1.213	1.445	2.638	4.806	7.836	11.835	16.906	23.144	30.641	39.484	49.758
0.00280	0.213	0.285	0.298	0.349	0.407	0.460	0.511	0.559	0.604	0.648	0.690	0.731
1/ 357	0.419	1.260	1.500	2.739	4.990	8.135	12.287	17.551	24.026	31.808	40.988	51.652
0.00300	0.221	0.295	0.309	0.361	0.421	0.477	0.529	0.578	0.626	0.671	0.715	0.757
1/ 333	0.434	1.305	1.554	2.837	5.168	8.424	12.723	18.173	24.877	32.935	42.438	53.479
0.00320	0.229	0.305	0.319	0.373	0.435	0.493	0.546	0.598	0.646	0.693	0.738	0.782
1/ 313	0.449	1.349	1.606	2.931	5.339	8.704	13.145	18.775	25.701	34.024	43.841	55.246
0.00340	0.236	0.315	0.329	0.385	0.449	0.508	0.563	0.616	0.666	0.715	0.761	0.806
1/ 294	0.463	1.391	1.656	3.023	5.506	8.975	13.554	19.358	26.499	35.080	45.201	56.959
0.00360	0.243	0.324	0.339	0.396	0.462	0.523	0.580	0.634	0.686	0.736	0.783	0.829
1/ 278	0.477	1.432	1.705	3.112	5.668	9.238	13.951	19.925	27.274	36.105	46.522	58.623
0.00380	0.250	0.333	0.349	0.407	0.475	0.537	0.596	0.652	0.705	0.756	0.805	0.852
1/ 263	0.490	1.472	1.753	3.198	5.825	9.494	14.337	20.476	28.028	37.102	47.806	60.240
Coefficient for part-full pipes:	14	20	20	25	35	40	45	50	60	70	70	80

ks = 1·500mm S < 0·004

Source: Hydraulic Research, 1990

8 continued

ks = 1·500mm
S = 0·004 to 0·1
ie hydraulic gradient = 1 in 250 to 1 in 10

Water (or sewage) at 15° C
full bore conditions.
velocities in m/s
discharges in l/s

Table B-5

Gradient	Pipe diameters in mm: 50	75	80	100	125	150	175	200	225	250	275	300
0.00400	0.256	0.342	0.358	0.418	0.487	0.551	0.612	0.669	0.723	0.776	0.826	0.875
1/ 250	0.503	1.511	1.799	3.282	5.978	9.743	14.713	21.013	28.762	38.074	49.057	61.816
0.00420	0.263	0.351	0.367	0.428	0.499	0.565	0.627	0.686	0.741	0.795	0.846	0.896
1/ 238	0.516	1.549	1.844	3.365	6.127	9.986	15.080	21.536	29.478	39.021	50.277	63.353
0.00440	0.269	0.359	0.376	0.439	0.511	0.579	0.642	0.702	0.759	0.814	0.867	0.917
1/ 227	0.528	1.586	1.888	3.445	6.273	10.224	15.438	22.047	30.177	39.946	51.468	64.854
0.00460	0.275	0.367	0.384	0.449	0.523	0.592	0.656	0.718	0 776	0.832	0.886	0.938
1/ 217	0.540	1.622	1.931	3.523	6.416	10.456	15.788	22.547	30.860	40.850	52.633	66.320
0.00480	0.281	0.375	0.393	0.458	0.534	0.605	0.671	0.733	0.793	0.850	0.905	0.959
1/ 208	0.552	1.658	1.973	3.600	6.555	10.683	16.130	23.035	31.529	41.735	53.773	67.756
0.00500	0.287	0.383	0.401	0.468	0.545	0.617	0.685	0.748	0.809	0.868	0.924	0.978
1/ 200	0.564	1.692	2.014	3.675	6.692	10.905	16.466	23.514	32.184	42.602	54.889	69.162
0.00550	0.301	0.402	0.421	0.491	0.572	0.648	0.718	0.785	0.849	0.911	0.970	1.026
1/ 182	0.592	1.776	2.114	3.857	7.022	11.443	17.276	24.671	33.766	44.695	57.585	72.558
0.00600	0.315	0.420	0.440	0.513	0.598	0.677	0.750	0.820	0.887	0.951	1.013	1.072
1/ 167	0.618	1.856	2.209	4.030	7.337	11.956	18.051	25.776	35.278	46.695	60.161	75.802
0.00650	0.328	0.438	0.458	0.534	0.623	0.704	0.781	0.854	0.924	0.990	1.054	1.116
1/ 154	0.644	1.933	2.301	4.197	7.640	12.448	18.794	26.836	36.728	48.614	62.632	78.915
0.00700	0.341	0.454	0.475	0.555	0.646	0.731	0.811	0.887	0.959	1.028	1.095	1.159
1/ 143	0.669	2.007	2.389	4.357	7.931	12.922	19.508	27.856	38.123	50.460	65.009	81.910
0.00750	0.353	0.470	0.492	0.574	0.669	0.757	0.840	0.918	0.993	1.064	1.133	1.200
1/ 133	0.693	2.078	2.474	4.511	8.212	13.379	20.198	28.840	39.470	52.241	67.303	84.799
0.00800	0.365	0.486	0.508	0.593	0.691	0.782	0.867	0.948	1.025	1.099	1.170	1.239
1/ 125	0.716	2.147	2.556	4.661	8.484	13.822	20.865	29.792	40.772	53.964	69.522	87.594
0.00850	0.376	0.501	0.524	0.612	0.713	0.806	0.894	0.978	1.057	1.133	1.207	1.278
1/ 118	0.738	2.214	2.635	4.806	8.747	14.250	21.512	30.715	42.034	55.634	71.673	90.303
0.00900	0.387	0.516	0.540	0.630	0.734	0.830	0.920	1.006	1.088	1.166	1.242	1.315
1/ 111	0.760	2.279	2.712	4.946	9.002	14.666	22.139	31.611	43.259	57.255	73.761	92.933
0.00950	0.398	0.530	0.555	0.647	0.754	0.853	0.946	1.034	1.118	1.199	1.276	1.351
1/ 105	0.781	2.342	2.788	5.083	9.251	15.071	22.750	32.482	44.451	58.832	75.792	95.491
0.01000	0.408	0.544	0.569	0.664	0.774	0.875	0.971	1.061	1.147	1.230	1.309	1.386
1/ 100	0.802	2.404	2.861	5.216	9.493	15.465	23.345	33.331	45.612	60.368	77.770	97.983
0.01100	0.429	0.571	0.597	0.697	0.812	0.918	1.018	1.113	1.203	1.290	1.374	1.454
1/ 91	0.841	2.522	3.002	5.473	9.960	16.225	24.491	34.967	47.850	63.329	81.583	102.786
0.01200	0.448	0.597	0.624	0.728	0.848	0.959	1.064	1.163	1.257	1.348	1.435	1.519
1/ 83	0.879	2.636	3.137	5.718	10.406	16.951	25.586	36.530	49.988	66.158	85.226	107.375
0.01300	0.466	0.621	0.650	0.758	0.883	0.999	1.107	1.210	1.309	1.403	1.494	1.581
1/ 77	0.916	2.744	3.266	5.954	10.834	17.648	26.637	38.029	52.039	68.871	88.721	111.776
0.01400	0.484	0.645	0.674	0.787	0.916	1.037	1.149	1.256	1.358	1.456	1.550	1.641
1/ 71	0.951	2.849	3.390	6.180	11.246	18.318	27.648	39.472	54.012	71.482	92.083	116.012
0.01500	0.501	0.668	0.698	0.815	0.949	1.073	1.190	1.301	1.406	1.508	1.605	1.699
1/ 67	0.984	2.950	3.510	6.399	11.643	18.964	28.623	40.864	55.916	74.001	95.328	120.099
0.01600	0.518	0.690	0.721	0.842	0.980	1.109	1.229	1.344	1.453	1.557	1.658	1.755
1/ 62	1.017	3.047	3.626	6.610	12.027	19.590	29.567	42.210	57.758	76.437	98.466	124.051
0.01700	0.534	0.711	0.744	0.868	1.010	1.143	1.267	1.385	1.498	1.605	1.709	1.809
1/ 59	1.049	3.142	3.739	6.815	12.400	20.196	30.481	43.515	59.543	78.799	101.507	127.882
0.01800	0.550	0.732	0.766	0.893	1.040	1.176	1.304	1.425	1.541	1.652	1.759	1.862
1/ 56	1.079	3.234	3.848	7.014	12.761	20.784	31.369	44.782	61.276	81.092	104.460	131.602
0.01900	0.565	0.752	0.787	0.918	1.069	1.209	1.340	1.465	1.584	1.697	1.807	1.913
1/ 53	1.109	3.323	3.954	7.208	13.113	21.357	32.232	46.014	62.961	83.322	107.332	135.220

Coefficient for part-full pipes:

18	25	30	35	45	50	60	70	80	90	100	110

ks = 1·500mm S < 0·1

Source: Hydraulic Research, 1990

Table B-6

i	k	R	h	Aw	Pw	Qp	Vel	u	o	x	v
0.0200	0.0015	0.075	0.101	0.01883	0.453	0.0250	1.328	-0.002	0.070349	0.1407	(
0.0143	0.0015	0.075	0.101	0.01883	0.453	0.0211	1.1219	-0.002	0.070349	0.1407	(
0.0167	0.0015	0.075	0.101	0.01883	0.453	0.0228	1.213	-0.002	0.070349	0.1407	(
0.0125	0.0015	0.075	0.101	0.01883	0.453	0.0197	1.0485	-0.002	0.070349	0.1407	(
0.0100	0.0015	0.075	0.101	0.01883	0.453	0.0176	0.9371	-0.002	0.070349	0.1407	(
0.0080	0.0015	0.075	0.101	0.01883	0.453	0.0158	0.8374	-0.002	0.070349	0.1407	(
0.0070	0.0015	0.075	0.101	0.01883	0.453	0.0147	0.7829	-0.002	0.070349	0.1407	(
0.0063	0.0015	0.075	0.101	0.01883	0.453	0.0140	0.7424	-0.002	0.070349	0.1407	(
0.0055	0.0015	0.075	0.101	0.01883	0.453	0.0131	0.6932	-0.002	0.070349	0.1407	(
0.0050	0.0015	0.075	0.101	0.01883	0.453	0.0124	0.6607	-0.002	0.070349	0.1407	(
0.0200	0.0015	0.1	0.1346	0.03345	0.604	0.0538	1.6081	-0.003	0.093823	0.1876	(
0.0167	0.0015	0.1	0.1346	0.03345	0.604	0.0491	1.4689	-0.003	0.093823	0.1876	
0.0142	0.0015	0.1	0.1346	0.03345	0.604	0.0453	1.3539	-0.003	0.093823	0.1876	(
0.0125	0.0015	0.1	0.1346	0.03345	0.604	0.0425	1.2699	-0.003	0.093823	0.1876	(
0.0100	0.0015	0.1	0.1346	0.03345	0.604	0.0380	1.1351	-0.003	0.093823	0.1876	(
0.0080	0.0015	0.1	0.1346	0.03345	0.604	0.0339	1.0146	-0.003	0.093823	0.1876	(
0.0070	0.0015	0.1	0.1346	0.03345	0.604	0.0317	0.9486	-0.003	0.093823	0.1876	
0.0060	0.0015	0.1	0.1346	0.03345	0.604	0.0294	0.8777	-0.003	0.093823	0.1876	(
0.0050	0.0015	0.1	0.1346	0.03345	0.604	0.0268	0.8007	-0.003	0.093823	0.1876	(
0.0040	0.0015	0.1	0.1346	0.03345	0.604	0.0239	0.7154	-0.003	0.093823	0.1876	(
0.0035	0.0015	0.1	0.1346	0.03345	0.604	0.0224	0.6688	-0.003	0.093823	0.1876	(
0.0200	0.0015	0.15	0.202	0.07531	0.907	0.1580	2.0984	-0.007	0.140698	0.2814	(
0.0167	0.0015	0.15	0.202	0.07531	0.907	0.1444	1.9169	-0.007	0.140698	0.2814	(
0.0143	0.0015	0.15	0.202	0.07531	0.907	0.1335	1.7733	-0.007	0.140698	0.2814	(
0.0125	0.0015	0.15	0.202	0.07531	0.907	0.1248	1.6575	-0.007	0.140698	0.2814	(
0.0100	0.0015	0.15	0.202	0.07531	0.907	0.1116	1.4818	-0.007	0.140698	0.2814	(
0.0080	0.0015	0.15	0.202	0.07531	0.907	0.0998	1.3246	-0.007	0.140698	0.2814	
0.0070	0.0015	0.15	0.202	0.07531	0.907	0.0933	1.2386	-0.007	0.140698	0.2814	(
0.0060	0.0015	0.15	0.202	0.07531	0.907	0.0863	1.1463	-0.007	0.140698	0.2814	(
0.0050	0.0015	0.15	0.202	0.07531	0.907	0.0788	1.0458	-0.007	0.140698	0.2814	(
0.0040	0.0015	0.15	0.202	0.07531	0.907	0.0704	0.9347	-0.007	0.140698	0.2814	(
0.0030	0.0015	0.15	0.202	0.07531	0.907	0.0609	0.8086	-0.007	0.140698	0.2814	(
0.0020	0.0015	0.15	0.202	0.07531	0.907	0.0496	0.659	-0.007	0.140698	0.2814	(
0.0090	0.0015	0.2	0.2693	0.13387	1.209	0.2267	1.6938	-0.013	0.18761	0.3752	(
0.0080	0.0015	0.2	0.2693	0.13387	1.209	0.2137	1.5965	-0.013	0.18761	0.3752	(
0.0070	0.0015	0.2	0.2693	0.13387	1.209	0.1999	1.4929	-0.013	0.18761	0.3752	(
0.0060	0.0015	0.2	0.2693	0.13387	1.209	0.1850	1.3817	-0.013	0.18761	0.3752	(
0.0050	0.0015	0.2	0.2693	0.13387	1.209	0.1688	1.2607	-0.013	0.18761	0.3752	(
0.0040	0.0015	0.2	0.2693	0.13387	1.209	0.1509	1.1269	-0.013	0.18761	0.3752	(
0.0030	0.0015	0.2	0.2693	0.13387	1.209	0.1305	0.975	-0.013	0.18761	0.3752	(
0.0020	0.0015	0.2	0.2693	0.13387	1.209	0.1064	0.7949	-0.013	0.18761	0.3752	(
0.0019	0.0015	0.2	0.2693	0.13387	1.209	0.1037	0.7746	-0.013	0.18761	0.3752	
0.0015	0.0015	0.2	0.2693	0.13387	1.209	0.0920	0.6875	-0.013	0.18761	0.3752	(
0.0014	0.0015	0.2	0.2693	0.13387	1.209	0.0889	0.664	-0.013	0.18761	0.3752	
0.0013	0.0015	0.2	0.2693	0.13387	1.209	0.0856	0.6396	-0.013	0.18761	0.3752	(
0.0125	0.0015	0.25	0.3367	0.20922	1.511	0.4823	2.3054	-0.02	0.234485	0.469	(
0.0100	0.0015	0.25	0.3367	0.20922	1.511	0.4313	2.0613	-0.02	0.234485	0.469	(

Table B-6

0.0070	0.0015	0.25	0.3367	0.20922	1.511	0.3606	1.7235	-0.02	0.234485	0.469	0
0.0050	0.0015	0.25	0.3367	0.20922	1.511	0.3045	1.4556	-0.02	0.234485	0.469	0
0.0040	0.0015	0.25	0.3367	0.20922	1.511	0.2722	1.3012	-0.02	0.234485	0.469	0
0.0030	0.0015	0.25	0.3367	0.20922	1.511	0.2356	1.1259	-0.02	0.234485	0.469	0
0.0020	0.0015	0.25	0.3367	0.20922	1.511	0.1921	0.9181	-0.02	0.234485	0.469	
0.0018	0.0015	0.25	0.3367	0.20922	1.511	0.1822	0.8706	-0.02	0.234485	0.469	0
0.0017	0.0015	0.25	0.3367	0.20922	1.511	0.1770	0.8459	-0.02	0.234485	0.469	0
0.0012	0.0015	0.25	0.3367	0.20922	1.511	0.1485	0.7097	-0.02	0.234485	0.469	0
0.0011	0.0015	0.25	0.3367	0.20922	1.511	0.1421	0.6792	-0.02	0.234485	0.469	0
0.0010	0.0015	0.25	0.3367	0.20922	1.511	0.1354	0.6473	-0.02	0.234485	0.469	0

Table : Velocity correction for variation in temperature

$V = V_{15} + at_{15}$ (in m/s) where $t_{15} = t - 15$ (in degrees centigrade)

RS (mm)	DS (mm)	Roughness values, k, in mm							
		0.006	**0.015**	**0.03**	**0.06**	**0.15**	**0.30**	**0.60**	**1.5**
500	2000	0.0327	0.0151	0.0080	0.0041	0.0017	0.0008	0.0004	0.0002
200	800	0.0290	0.0142	0.0077	0.0040	0.0017	0.0008	0.0004	0.0002
100	400	0.0257	0.0134	0.0075	0.0040	0.0016	0.0008	0.0004	0.0002
50	200	0.0222	0.0124	0.0071	0.0039	0.0016	0.0008	0.0004	0.0002
20	80	0.0174	0.0107	0.0065	0.0037	0.0016	0.0008	0.0004	0.0002
10	40	0.0140	0.0093	0.0060	0.0035	0.0016	0.0008	0.0004	0.0002
5	20	0.0110	0.0079	0.0054	0.0033	0.0015	0.0008	0.0004	0.0002
2	8	0.0077	0.0060	0.0044	0.0029	0.0014	0.0008	0.0004	0.0002
1	4	0.0057	0.0048	0.0037	0.0026	0.0013	0.0008	0.0004	0.0002
0.5	2	0.0042	0.0037	0.0030	0.0022	0.0012	0.0007	0.0004	0.0002
0.2	0.8	0.0028	0.0025	0.0022	0.0017	0.0011	0.0007	0.0004	0.0002
0.1	0.4	0.0020	0.0019	0.0017	0.0014	0.0009	0.0006	0.0004	0.0002
0.05	0.2	0.0014	0.0014	0.0013	0.0011	0.0008	0.0005	0.0003	0.0002
0.02	0.08	0.0009	0.0009	0.0008	0.0008	0.0006	0.0004	0.0003	0.0002
0.01	0.04	0.0007	0.0006	0.0006	0.0006	0.0005	0.0004	0.0003	0.0002

The values of 'a' tabulated above show the velocity adjustment in m/s for a 1°C temperature difference

Table Proportional velocities in pipes running part-full

(a) Circular sections

Proportional depth

Coefficient for part-full pipes = $\left[\frac{k}{D} + \frac{1}{3600DS^{1/3}}\right]^{-1}$ for water at 15°C (k and D in metres)

	5	10	20	50	100	200	500	1000	2000
0.02	0.000	0.043	0.073	0.100	0.116	0.128	0.140	0.148	0.154
0.04	0.073	0.121	0.154	0.184	0.200	0.213	0.226	0.235	0.242
0.06	0.143	0.191	0.224	0.254	0.270	0.283	0.297	0.305	0.312
0.08	0.207	0.254	0.286	0.316	0.332	0.345	0.358	0.366	0.373
0.10	0.267	0.313	0.344	0.372	0.387	0.400	0.412	0.420	0.427
0.12	0.324	0.367	0.396	0.423	0.438	0.450	0.462	0.469	0.476
0.14	0.377	0.418	0.446	0.471	0.485	0.496	0.508	0.515	0.521
0.16	0.428	0.466	0.492	0.516	0.529	0.539	0.550	0.557	0.562
0.18	0.475	0.511	0.536	0.558	0.570	0.580	0.590	0.596	0.601
0.20	0.521	0.554	0.577	0.598	0.609	0.618	0.627	0.633	0.638
0.22	0.564	0.595	0.616	0.635	0.645	0.654	0.662	0.668	0.672
0.24	0.606	0.634	0.653	0.670	0.680	0.688	0.696	0.700	0.704
0.26	0.645	0.671	0.688	0.704	0.713	0.720	0.727	0.731	0.735
0.28	0.683	0.706	0.722	0.736	0.744	0.750	0.757	0.761	0.764
0.30	0.719	0.740	0.754	0.767	0.774	0.779	0.785	0.789	0.792
0.35	0.802	0.817	0.827	0.837	0.842	0.846	0.850	0.852	0.854
0.40	0.877	0.886	0.892	0.898	0.902	0.904	0.907	0.908	0.910
0.45	0.942	0.947	0.950	0.953	0.954	0.955	0.957	0.957	0.958
0.50	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.55	1.050	1.046	1.043	1.041	1.039	1.038	1.037	1.037	1.036
0.60	1.092	1.084	1.080	1.075	1.073	1.071	1.069	1.067	1.066
0.65	1.126	1.116	1.109	1.103	1.099	1.097	1.094	1.092	1.091
0.70	1.152	1.140	1.131	1.124	1.120	1.117	1.113	1.111	1.109
0.75	1.169	1.156	1.146	1.138	1.133	1.130	1.126	1.124	1.122
0.80	1.177	1.163	1.153	1.145	1.140	1.136	1.132	1.129	1.127
0.85	1.174	1.160	1.151	1.142	1.137	1.134	1.130	1.127	1.125
0.90	1.158	1.145	1.136	1.129	1.124	1.121	1.117	1.115	1.113
0.95	1.120	1.111	1.104	1.098	1.095	1.092	1.090	1.088	1.087

Note: Values of the coefficient for part-full pipes for use with this table are given at the foot of each column of the tables for full-bore conditions.

Table . Proportional discharges in pipes running part-full

(a) Circular sections

Coefficient for part-full pipes = $\left[\frac{k_s}{D} + \frac{1}{3600DS^{1/3}}\right]^{-1}$ for water at 15°C (k_s and D in metres)

Proportional depth	5	10	20	50	100	200	500	1000	2000
0.02	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001
0.04	0.001	0.002	0.002	0.002	0.003	0.003	0.003	0.003	0.003
0.06	0.003	0.005	0.005	0.006	0.007	0.007	0.007	0.007	0.008
0.08	0.008	0.010	0.011	0.012	0.012	0.013	0.013	0.014	0.014
0.10	0.014	0.016	0.018	0.019	0.020	0.021	0.021	0.022	0.022
0.12	0.022	0.025	0.027	0.029	0.030	0.031	0.031	0.032	0.032
0.14	0.032	0.036	0.038	0.040	0.041	0.042	0.043	0.044	0.044
0.16	0.044	0.048	0.051	0.053	0.055	0.056	0.057	0.057	0.058
0.18	0.058	0.063	0.066	0.068	0.070	0.071	0.072	0.073	0.074
0.20	0.074	0.079	0.082	0.085	0.087	0.088	0.089	0.090	0.091
0.22	0.092	0.097	0.100	0.104	0.105	0.107	0.108	0.109	0.110
0.24	0.112	0.117	0.121	0.124	0.126	0.127	0.128	0.129	0.130
0.26	0.133	0.139	0.142	0.145	0.147	0.149	0.150	0.151	0.152
0.28	0.157	0.162	0.165	0.169	0.171	0.172	0.173	0.174	0.175
0.30	0.181	0.187	0.190	0.193	0.195	0.197	0.198	0.199	0.200
0.35	0.250	0.255	0.258	0.261	0.263	0.264	0.265	0.266	0.267
0.40	0.327	0.331	0.333	0.336	0.337	0.338	0.339	0.339	0.340
0.45	0.411	0.413	0.415	0.416	0.416	0.417	0.417	0.418	0.418
0.50	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
0.55	0.592	0.589	0.588	0.587	0.586	0.585	0.585	0.584	0.584
0.60	0.684	0.679	0.676	0.673	0.672	0.671	0.669	0.669	0.668
0.65	0.775	0.768	0.763	0.759	0.756	0.755	0.753	0.751	0.750
0.70	0.861	0.852	0.846	0.840	0.837	0.835	0.832	0.831	0.829
0.75	0.941	0.930	0.922	0.916	0.912	0.909	0.906	0.904	0.902
0.80	1.010	0.997	0.989	0.982	0.977	0.974	0.971	0.969	0.967
0.85	1.064	1.051	1.043	1.035	1.030	1.027	1.023	1.021	1.019
0.90	1.097	1.085	1.077	1.070	1.066	1.063	1.059	1.057	1.055
0.95	1.099	1.090	1.084	1.078	1.075	1.072	1.069	1.068	1.066

Note: Values of the coefficient for part-full pipes for use with this table are given at the foot of each column of the tables for full-bore conditions.

Table Geometric

(a) Circular sections

Proportional depth	tted Surimeter wid ÷ al diameter	Sectional area ÷ total area	Hydraulic mean depth ÷ D/4	Mean depth ÷ D
	21	0.878	1.217	0.897
0.02	0.238	0.897	1.215	0.961
0.04	0.356	0.915	1.210	1.035
0.06	0.475	0.932	1.203	1.126
0.08	0.595	0.948	1.192	1.241
0.10	0.6			
	17	0.963	1.177	1.393
0.12	0.742	0.976	1.158	1.613
0.14	0.872	0.987	1.132	1.977
0.16	0.910	0.995	1.094	2.792
0.18	0.			
0.20	0.			
	face width			
0.22	0.			
0.24	0.			
0.26	0.			
0.28	0			
0.30	0			
0.32	0			
0.34	0			
0.36	0			
0.38	0			
0.40	0			
0.42	0			
0.44	0			
0.46	0			
0.48	0			
0.50				
0.52				
0.54				
0.56				
0.58				
0.60				
0.62				
0.64				
0.66				
0.68				
0.70				
0.72				
0.74				
0.76				
0.78				
0.80				

) x √ (proportional mean depth)

0 x √ (proportional mean depth) x proportional section area

Table Recommended

Classification (assumed clear wise stated)	Suitable values of k, (mm) Good	Normal	Poor
Smooth materials			
Drawn non-ferrous pipes of n per lead etc, and non-metall glass, perspex etc	—	3.0	6.0
Asbestos-cement	—	3.0	6.0
Metal	—	1.5	3.0
Spun bitumen or concrete li	—	0.6	1.5
Wrought iron			
Rusty wrought iron			
Uncoated steel			
Coated steel	—	1.5	3.0
Galvanised iron, coated cast	—	0.6	1.5
Uncoated cast iron	—	0.3	0.6
Tare relined pipes	—	0.15	0.3
Old tuberculated water mai			
degrees of attack:			
Slight	0.15	0.3	0.6
Moderate	0.06	0.15	0.30
Appreciable	0.03	0.06	0.15
Severe			
(Good: Up to 20 years us years use, Poor: 80 to 10	60	150	300
Wood	—	300	600
Wood stave pipes, planed p			
Concrete			
Precast concrete pipes with	15	60	150
Spun precast concrete pipe			
Monolithic construction ag	150	300	600
Monolithic construction ag			
Clayware			
Glazed or unglazed pipe:			
With sleeve joints			
With spigot and socket j — dia < 150mm			
With spigot and socket j — dia > 150mm			
Pitch fibre (lower value rel			
Glass fibre			
uPVC			
With chemically cemented			
With spigot and socket joi metre intervals			
Brickwork			
Glazed			
Well pointed			
Old, in need of pointing			

es considerably during any year. The nor-
eeded for approximately half of the time.
l, generally on a continuous basis, for one
uld be interpolated for velocities between