IMPACTS OF STORMWATER AND GROUNDWATER INGRESS ON MUNICIPAL SANITATION SERVICES

Report to the
WATER RESEARCH COMMISSION

by

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on behalf of the

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EXECUTIVE SUMMARY

South Africa has experienced rapid growth in water service provision, particularly in the last decades. In the past, standards have been sacrificed and many older and poor quality sewers are showing signs of leaking. Insufficient attention is paid to maintenance and rehabilitation resulting in overloading of sewers and Waste Water Treatment Works.

Sewer blockages and collapses take place due to penetration of roots, structural movement, opening of joints, corrosion, sedimentation and inadequate construction. Besides stoppages and collapses, stormwater inflows and ongoing groundwater infiltration (or so-called extraneous flows) can reduce the originally designated capacity of a sewer collection system and negatively affect operation of the entire waterborne sanitation system including the wastewater treatment component (WWTP). Increases in extraneous flows reduce the effectiveness of the biological process leading to higher pollution loads leaving the WWTP and disposing partially treated urban wastewater into adjacent river ecosystems.

Urban wastewater quantity and quality management issues and problems are now equally important for either centralized or decentralized disposal of urban wastewater return flows. Both quantity and quality of such water is crucial to the well-bring of other water users located downstream of the sources discharging wastewater and stormwater.

The urban water cycle is adversely affected by extraneous flows. If there is exfiltration, there can be groundwater pollution which also affects the catchment water balance. Water consumption is higher if plumbing leaks and more water is cycled on a macro scale. This increases the overall water supply cost as well as causing pollution. The linkage of water flow between water supply input point and treated effluent output point enables observation of the whole urban water cycle. The evaluation of return flows and consumptive use is a critical component in water resources development studies at various wastewater catchment levels. Stormwater inflows and groundwater infiltration into sewers have costly implications. Although in the past it has been the practice to allow spare sewer capacity, this can increase sewer costs by 10%-30%, and wastewater treatment by some 10%. The alternatives available to mitigate extraneous flows are:

- Allow spare capacity in sewers and wastewater treatment plant (WWTP).
- Lay pipes to high specifications to minimize infiltration/exfiltration
- Refurbish existing deteriorated sewers
- Manage the feeder area to minimize extraneous flows
- Stormwater management (e.g. retention or detention holding dams at WWTP)

A questionnaire distributed nationally produced alarming data. Little attention is paid to inflows and infiltration and it is customary to allow for up to 50% of sewer capacity and also to design WWTP to cope with this flow. This project identified low awareness about I/I/E problems and remedial/rehabilitation techniques by most South African WSAs/WSPs. Due to the magnitude and complexities inherent to municipal waterborne sewer systems, WSAs/ WSPs cannot make an educated decision on developing a new or upgrading/rehabilitating existing systems without a mixture of field and modelled data.

One of the major conclusions in this project is that most WSAs/WSPs in South Africa resort to reactive maintenance, where problems are dealt with on a corrective basis as they arise. Consequently, municipal wastewater system maintenance budgets are commonly low and are based on the previous year's financial expenditure on clogging and collapses. Stoppages and clogging of sewers in South Africa per unit length of sewer are about ten times higher than the international average. In the meantime, the deterioration of municipal waterborne sewers continues to the point of failure and beyond.

In order to gauge general awareness about inflow/infiltration (I/I) problems in municipal sanitation systems in South Africa, a nation-wide e-mail survey was conducted under this research project. The survey generated new valuable information and verified several parameters for the development/enhancement of urban separated sewer systems.

It was established that most municipal sewer systems in South Africa have been in existence for 30 to 50 years and the aging process is taking its toll so that issues related to rehabilitation or replacement are becoming more important to the WSAa/WSPs. The type of materials used in the construction of sewer systems have also changed from clay and concrete to uPVC and AC piping, generating different problems.

The sample survey indicated that the most common causes of stormwater inflows and groundwater infiltration in the South African context are as follows:

- Inadequate design of certain system components,
- Illegal house down-pipe connections to the municipal sewers (all surveyed municipalities operate separate instead of combined sewers),
- Open gullies serve primarily as sullage disposal (this is typically in most formal and informal townships),
- Unsealed manholes primarily due to theft of the manhole covers,
- Faulty pipe joints due to improper construction or deterioration,
- Roots penetrating joints
- Unwise man-made stormwater channelisation (e.g. road crossings and culverts) and unattended overgrown vegetation in natural channels, and
- High groundwater table.

Next to the common causes generated by this survey, other factors were identified which can contribute locally to inflow/infiltration to sewers:

- Undulating topography may lead to easy flooding due to marginal changes in stormwater flows,
- Re-considered flood lines,
- Swimming pools can be a contributing factor if additional stormwater or backwash water is linked directly to the sewers,
- Ground movement due to removed mine dumps destroying continuity of sewers, and
- Thunderstorms of short duration and higher intensities in various locations.

From the limited but representative sample of the nation-wide survey, it is concluded that the typical average sewer blockage rate is 3,3 blockages/km sewer pipe/p.a. This figure is more than double the average commonly quoted in the limited literature in South Africa of 1,2 blockages/km pipe/year and far supercedes international averages. This aspect will influence the calculation of costs if trenchless technology is adopted in sewer maintenance programmes. The most common materials used by the municipalities are identified as uPVC and AC piping, in pipe diameters ranging from 100 to 1000 mm. A surprising aspect of municipal sewer systems is the large number of pumping stations built in some of the existing systems which give rise to a number of problems.

An area in the Ekurhuleni Metropolitan Municipality (namely Boksburg urban area) was assessed as a case study. Next to residential and recreation grounds, it abuts an industrial area and sewers are built through waterlogged ground. It was found that stormwater inflows amounted to up to 40% of sewer capacity and groundwater infiltration amounted to 15% of capacity. Leaking household and faulty plumbing contributed a comparable amount. Field investigation methodologies and inflow/infiltration monitoring results compiled by others in the urban areas of Gauteng province and elsewhere in South Africa were evaluated and relevant findings were combined into this report. Typical values of key sewer flow components are as follows:

- Residential wastewater outflows range between 0,01 and 1,20 ℓ/min/household
- Water leakages into municipal waterborne sewers range between 0,06 and 0,20 ℓ/min/household
- Groundwater infiltration into municipal sewers ranges between 0,01 and 0,50 ℓ/min/m-dia/m-pipe for all types of sewer materials

From technical reports available on the subject of inflow/infiltration in Boksburg and Benoni, the following were identified:

- Stormwater and surface inflows account for dramatic peak flows (up to 3 times the AADWF) experienced, particularly in the Boksburg Outfall. The source of the inflows can be attributed predominantly to household stormwater being directed into the sewer system through gulleys, and to a lesser extent, due to missing or damaged manhole covers.
- Ground water infiltration produces a steady base flow in the sewers, which increases treatment costs and reduces the operating capacity in downstream sewers. It appears that only in severe cases where the extent of groundwater infiltration may cause structural collapse or substantial reduction in the capacity that pipe replacement or repair makes financial and practical sense.
- Both stormwater inflow and groundwater infiltration are continuing to increase due to reactive maintenance instead of planned preventative maintenance and planned rehabilitation programmes.

The key conclusion this project identified was low awareness about I/I problems and remedial/rehabilitation techniques by most South African WSAs/WSPs. Due to the magnitude and complexities inherent to municipal waterborne sewer systems, only a few WSAs/WSPs can make an educated decision on developing a new or upgrading/rehabilitating an existing system (or its key components). They lack mainly field and modeled data, particularly on inflow/infiltration/ exfiltration events and their consequences.

Guidelines concerning construction and rehabilitation of sewers were prepared as part of the contract. Methods of evaluating the problem and remedying the situation were listed. It is recommended to consider that groundwater infiltration exceeding 0,10 ℓ/min/m-dia/m-pipe is excessive for all sewer pipe materials. Another major observation from this project is that the maintenance strategy of most WSA/WSPs in South Africa is essentially reactive maintenance, where problems are dealt with on a corrective basis as they arise. Consequently, municipal wastewater system maintenance budgets are commonly low and are based on the previous year's financial expenditure mainly from clogging and collapsing sewers. It has been established from a survey that stoppages and clogging of sewers in South Africa per are about ten times higher than the international average, averaging to 3,3 blockages/km/sewer pipe/per annum.

It was also established by this project that the costs associated with maintaining or expanding existing and/or developing new urban wastewater infrastructure appear to be large, but well invested if allocated on a regular basis. Because there is not yet enough pressure applied from the wastewater services end-users to municipal managers about the economics of alternative solutions, conventional methods prevail and benefits are not highlighted in the cost analysis. Water infiltration in sewer pipelines is common and should be included in the peak design flow. A norm of 15% of the dry weather flow allowance for extraneous flows is a generally acceptable standard. Flows exceeding this norm will result in pipe capacity problems and an unnecessary increase in sewer discharge volumes and treatment costs. A reduction in infiltration/inflow rates will not only save on sewerage treatment costs, but may defer capital expenditure for the upsizing of collection sewer pipelines and wastewater treatment plant. The decision to solve or ignore an infiltration problem should therefore be based on a benefit-cost analysis.

Due to the magnitude and complexity of the attention required to research, design, construction and management of wastewater sanitation systems, all relevant stakeholders must share responsibility for development and management of these systems. The application of new local and international technologies must be promoted by the WSAs and WSPs through adopting the Guidelines in capacity building programmes.

To sustain a reliable municipal wastewater infrastructure and required service to customers, new and improved solutions to existing and emerging problems will have to be researched. Spheres for further research relate to flow monitoring, assessment of structural integrity, operation and maintenance programmes, and new methods in rehabilitation of sewers.

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Glossary

Aerobic:	Condition in which dissolved oxygen is present
Anaerobic:	Condition in which dissolved oxygen is not present
Attentuation:	The reduction in peak flow or concentration and increase in minimum flow and concentration of the diurnal variation in wastewater flow as it passes through the sewerage system
Bedload:	That part of the sediment load that travels by rolling or sliding along the sewer invert or deposited bed, or by saltation
Biochemical oxygen demand:	The amount of dissolved oxygen consumed by microbiological action when a sample is incubated in the dark at 20^oC
Black water:	Wastewater consisting of human excreta, urine and the associated sludge
Blockage:	A deposit in a sewer or drain resulting in restriction of flow
Catchment:	An area served by a single drainage system
Chemical oxygen demand:	The measure of oxygen required to oxidize all organic material in a water sample with a strong chemical, usually potassium dichromate
Combined sewer	Sewer conveying both wastewater and surface water
Diurnal variation:	The variation in flow rate or in the concentration (or mass flow) of a substance over a period of 24 hours
Domestic wastewater:	Wastewater discharged from kitchens, washing machines, lavatories. bathrooms and similar facilities
Drain:	A pipeline, usually underground, designed to carry wastewater and/or surface water from a source to a sewer; a pipeline carrying land drainage flows or surface water from a highway
Effluent:	Liquid discharged from a given process
Exfiltration:	The escape of wastewater from the sewerage system into the surrounding soil via cracks or malfunctioning pipe joints
Foul sewage:	Waterborne waste of domestic or industrial origin excluding rainwater and surface water
Grey water:	Wastewater from kitchen and bath effluent
Gross solids:	Large faecal and organic matter and other wastewater debris
Infiltration (to sewer):	The ingress of groundwater into a drain or sewer system through defects in pipes, joints or manholes
Inflow:	Stormwater surface runoff that enters a sewer through deficient manholes, etc.
Invert:	The bottom of the inside of a pipe or conduit
Lateral:	A private drain carrying drainage flows from a property to a public sewer
Manhole:	A chamber with a removable cover constructed on a drain or sewer to permit entry by personnel

Partially separate System Separate system in which some surface water is admitted to the sewers that convey foul water

Pollutant: Any substance conveyed in solution, suspension or as a discrete solid and discharged to a water course, thus adversely affecting its quality

Pressure sewerage System A system that operates under positive pressure to pump drainage flows from a property or group of properties into a public sewer; the system may consist of one or more pumps, storage chambers, pipework and non-return valves

Receiving water: Watercourse, river, estuary or coastal water into which the outfall from a combined sewer overflow or wastewater treatment works discharges

Runoff: Water from precipitation which flows off a surface to reach a drain, sewer or receiving water

Sediment: Material transported in a liquid that settles or tends to settle

Separate system: A drain or sewer system, normally of two pipelines, one carrying wastewater and the other surface water

Sewage: Wastewater

Sewer: Pipeline or other conduit, normally underground, designed to convey wastewater, stormwater or other unwanted liquids

Sewer flooding: The unintentional escape of sewage from a sewerage system; the inability of drainage flows to enter a sewerage system because of surcharging

Sewerage system: System of sewers and ancillary works that conveys wastewater to a treatment works or other disposal point

Soffit: The top of the inside of a pipe or conduit

Specific gravity: The mass of a substance divided by the mass of the same volume of water

Surcharge: The condition in which wastewater and/or surface water is held under pressure within a gravity drain or sewer system, but does not escape to the surface to cause flooding

Suspended solids: Solids transported in suspension in the wastewater flow and prevented from settling by the effects of flow turbulence

Wastewater: Water discharged as a result of cleansing, culinary or industrial processes to a drain or sewer system

Abbreviations

ADWF: Average dry weather flow (ℓ/s)
$BOD_{(5)}$: Biochemical oxygen demand (five day)
COD: Chemical oxygen demand
CSO: Combined sewer overflow
CCTV: Closed-circuit television
DO: Dissolved oxygen
DWF: Dry weather flow
DWL: Dry weather load
EBOD: Effective biological oxygen demand
EDU: Equivalent discharge unit (100 ℓ/day)
ERWAT: East Rand Water Care Company
FFT: Flow to full treatment
FOG: Fat, oils and greases
GIS: Geographic information system
HRT: Hydraulic retention time
IDP: Integrated development plan
MDPF: Maximum daily peak flow
MH: Manhole
MPN: Most probably number
NWA: National Water Act (Act 36 of 1998)
OMR: Operation, maintenance and repair
pH: Hydrogen-ion capacity
PDWF: Peak dry weather flow (ℓ/s)
PWWF: Peak wet weather flow (ℓ/s)
PS: Pumping station
PSS: Pressure sewerage system
SS: Suspended solids
TOC: Total organic carbon
TSS: Total suspended solids
UDF: Urban development framework
UPM: Urban pollution management
WSA: Water Services Act (Act 108 of 1997)
WSAM: Water Situation Assessment Model (DWAF)
WSDP: Water services development plan
WWTW: Wastewater treatment works

CHAPTER 1. INTRODUCTION TO WATERBORNE SANITATION PROBLEMS

1.1 Status of waterborne sanitation in South Africa

1.1.1 Basic sanitation facility

Since 1994, water supply and sanitation services have been determined according to the general requirements guided by the equitable delivery of services and redistribution of available resources approach under the philosophy of integrated economic development. In satisfying such requirements, the basic sanitation facility is at present regarded as a Ventilated Improved Pit (VIP) toilet. However, the desired sanitation system for all urban dwellers is a flush toilet connected to a fully waterborne sanitation system.

1.1.2 Status of sanitation development and key stakeholders

The World Summit held in Johannesburg, South Africa, in 2002 concluded and recommended to halve the number of people worldwide who have at present no access to safe drinking water and basic sanitation by the year 2015. Table 1.1 below illustrates the most probable status of sanitation services in South Africa in 2003.

Table 1.1. Type and extent of sanitation in South Africa

Type of sanitation facility	(%)	Number of households (million)
None	13,6	1,56
Chemical toilets	1,9	0,22
Bucket collection system	4,1	0,47
Pit latrines without ventilation	22,8	2,62
Ventilated Improved Pit (VIP)	5,7	0,66
Flush toilet (septic tank)	2,8	0,32
Flush toilet (waterborne sewer)	49,1	5,65
Total for South Africa	**100,0**	**11,50**

Sources: Sanitation – DWAF (2003), Households – STATSSA (2004)

More than eighty percent of households in South Africa have some form of sanitation facility installed and on average 20 000 basic sanitation facilities (i.e. VIPs) are built annually around South Africa according to the Strategic Framework for Water Services (SFWS) programme. This is a programme on the strategic challenges and opportunities in water services delivery which has been agreed between the DWAF (i.e. State), SA Association of Water Utilities (SAAWU), SA Local Government Association (SALGA) and unions representing water industry sector employees. The sanitation targets agreed are as follows:

- *All schools to have adequate and safe water supply and sanitation services by 2005.*
- *All bucket toilets are to be eradicated by 2006.*
- *All clinics to have adequate and safe water supply and sanitation services by 2007.*
- *All people in SA to have access to a basic sanitation facility by 2010.*

At present some 6 million households in South Africa are connected to waterborne sewers which are experiencing specific problems in their operation and maintenance.

1.2 Background on provision of urban waterborne sanitation services

1.2.1 Conditions in provision of urban waterborne sanitation services

The national aim in the implementation of water services (i.e. water supply and sanitation) will inevitably generate extensive demand from all communities who wish to be provided with not only potable water but also adequate sanitation (preferably waterborne sewerage). The new water legislation and more stringent environmental requirements for better protection of river ecosystems will indirectly support more extensive development of urban sanitation services and upgrading of existing infrastructure.

Since promulgation of the Water Services Act (Act 108 of 1997) and National Water Act (Act 36 of 1998), numerous new water services criteria concerning design and construction of sanitation infrastructure have to be verified taking into consideration socio-economic changes within South African society.

The underlying obligation in providing basic sanitation requirements is that every water services institution (WSI) in South Africa must take measures to prevent any objectionable substance from entering any water ecosystem. Such a substance can be domestic wastewater, industrial effluent, petroleum products, chemicals and leachates from solid waste dumps.

Section 21 of the NWA deals with water use licenses from the State, enforced by the DWAF, which may be required for discharging effluent into a watercourse. The authorisation would specify the types and maximum levels of contaminants that the effluent is allowed to contain. If accepting that a discharge would pose a risk to the treatment process or lead to a breach of the permit, the WSI should only agree to accept the effluent once the harmful substances have been removed or reduced. All permit holders, but primarily, industries have to comply with the following:

- *Pre-treating their effluent such that it complies with the permit conditions.*
- *Separating effluent discharges and treating the harmful component of the discharges separately.*
- *Collecting harmful matters that are then removed by appropriate waste disposal contractors*.
- *The quantity and concentration of the effluent must be considered together to arrive at the total contaminant load.*

It must be noted that permit holders are not allowed to dilute effluent in order to comply with the set concentration limits.

1.2.2 Urbanisation trends and urban waterborne sanitation

Significant changes in land use, particularly in urban areas, highlighted several of issues present in numerous municipal areas such as, for example, the fluctuation in local groundwater tables and increased flood lines of urban regulated streams. Changes in water legislation and environmental protection laws, concerns over increasing pollution of groundwater resources, and, particularly, the need for water services authorities and providers to optimise the allocation of capital between new developments and upgrading existing infrastructure and processes, are leading to increased awareness about the impacts of stormwater inflow and groundwater infiltration in waterborne sanitation. The effects of urban developments on stormwater quality and quantity as well as groundwater infiltration into the sewer facilities increased the urgency for a strategic approach to these problems.

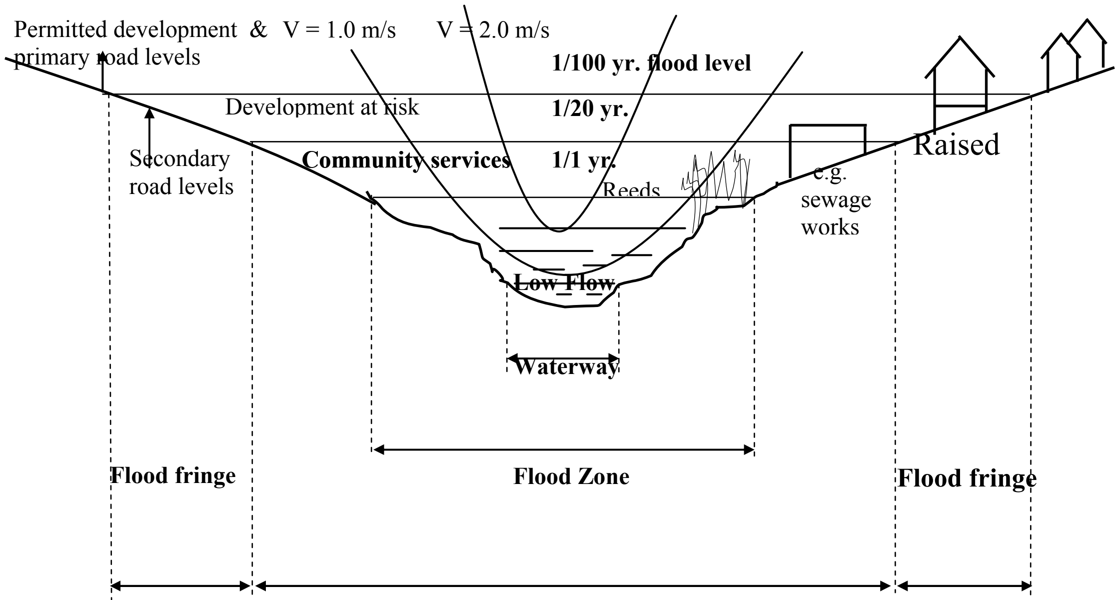


Figure 1.1. Impacts of urban development on river channel zones (after Stephenson and Furumele, 2001)

The development of small urban areas generate a large concentration of impervious areas such as roofs and roads. This results in a change of the local hydrological cycle causing a decrease in ground infiltration and groundwater recharge and the pattern of surface and river runoff also being altered. The changed conditions accelerate high peak flows, large runoff volumes including transport of pollutants and sediment load from urban areas. The existence of urban areas influences the state of the ecological systems along the river courses.

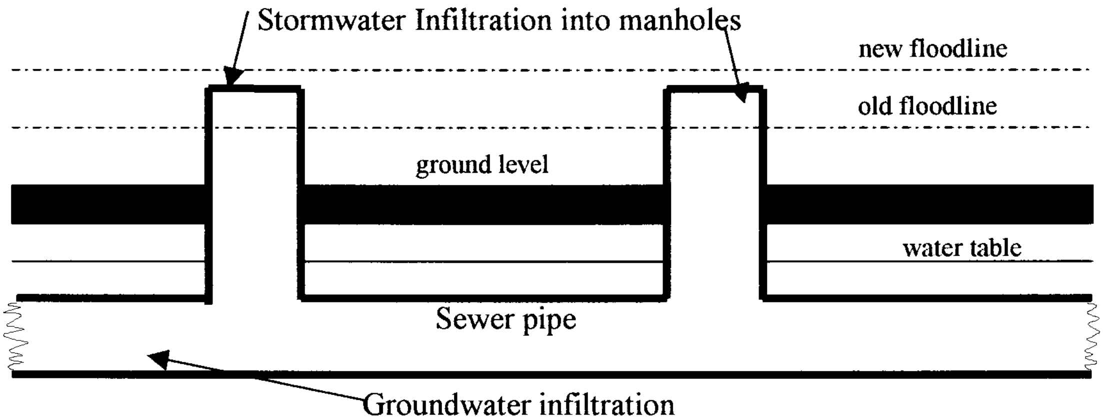


Figure 1.2. Effects of floodline changes on existing waterborne infrastructure

The traditional philosophy on conveyance and disposal of urban stormwater is changing and it is now generally accepted that stormwater should be attenuated locally within urban areas. This is very important where stormwater is separated from wastewater. In a more advanced way, the stormwater infrastructure, particularly in newly developed urban areas, stores surface flow so that it can be treated and possibly reused. This approach to the management of stormwater will influence the whole approach to the development of stormwater facilities and wastewater management. In the near future, stormwater might constitute a “new” water source if treated directly for reuse or together with “grey water” in supplementing the domestic water use primarily in urban areas.

1.2.3 Problem of extraneous flows in urban waterborne sanitation

Extraneous flows can be defined as an excessive inflow/infiltration of water (or I/I events) into the existing waterborne sewerage system due to uncontrolled surface inflow and/or groundwater infiltration on account of infrastructural deficiencies (e.g. missing manhole covers, damaged pipes due to poor trench bedding, etc.) or incorrect management practices of urban stormwater.

In the South African context, I/I events are seasonal, depending on the precipitation intensity, patterns of land use and other parameters of a drainage catchment. Excessive inflow/infiltration may cause sewer surcharges, local flooding and unnecessary pumping of wastewater at critical locations within the collection network and treatment plants.

At the wastewater treatment plants, hydraulic overload may adversely affect both the physical and biological treatment processes. Wet weather periods require usually overflow bypassing for temporary storage if inflow/infiltration volumes are excessive.

1.2.4 Problem of exfiltration from waterborne sanitation systems

Exfiltration can occur when the elevation of the sewer liquid level is above the groundwater table. The positive head created by such circumstances can cause the raw sewage to exfiltrate through open joints into the surrounding ground with a strong possibility of polluting the groundwater. Exfiltration can also cause a concentrated flow in the sewer trenches and the raw sewage can find it’s way into ground and surface water sources (i.e. boreholes, streams, etc.) introducing a serious environmental and health hazard. Leaking waterborne sewers have long been suspected of being a source of groundwater contamination mainly in urban areas. Presently, the WSA/WSPs responsible for urban waterborne sanitation systems do not possess sufficient awareness and have any suitable evaluation methods for effective determination of exfiltration from waterborne sewers.

1.3 Compliance with water services legislation and regulations

1.3.1 National legislation related to the provision of urban water services

Municipal water supply and sanitation services (i.e. water services) are considered mutually inclusive and are being developed as far as possible according to a balanced development strategy since the introduction of the Water Services Act (Act 108 0f 1997).

Historically, the water law principles approved by the SA Government in 1996, together with the SA Constitution and Agenda 21, have led to the inception of the National Water Policy being adopted as government policy in 1997. The promulgation of the National Water Act (Act 36 of 1998) culminated the process formulating the National Water Resources Strategy (NWRS) which is now guiding the development and management of the remaining water resources in South Africa. Several steps have already been taken in implementing the NWRS, as for example; water use licensing, free water up to 6 000 litres per household per month, the formation of nineteen Water Management Areas and Catchment Management Agencies, etc.

Both WSA (1997) and NWA (1998) are important to the NWRS, but there are other pieces of contemporary legislation relevant to the development and management of municipal sanitation systems as shown in Table 1.2 below:

Table 1.2. SA legislation related to urban water services provision

Sphere of legislation	Act	Abbreviated reference
Atmospheric Pollution Prevention Act, 1965	Act No 45 of 1965	APPA (1965)
Conservation of Agricultural Resources Act, 1983	Act No 43 or 1983	CARA (1983)
Environmental Conservation Act, 1989	Act 73 of 1989	ECA (1989)
Health Act, 1977	Act No 63 of 1977	HA (1977)
Local Government Transition Act, 1993	Act N9 209 of 1993	LGTA (1993)
Mineral Act and its Regulations, 1991	Act No 50 of 1991	MAAR (1991)
Municipal Structures Act, 1998	Act No 117 of 1998	MSTA (1998)
Municipal Systems Act, 2000	Act No 32 of 2000	MSA (2000)
National Environmental Management Act, 1998	Act No 107 of 1998	NEMA (1998)
National Water Amendment Act, 1999	Act No 45 of 1999	NWAA (1999)
Water Research Act, 1971	Act No 34 of 1971	WRA (1971)

Sources: DWAF and Government Gazettes (various)

1.3.2 Evolving standards and national codes of practice

The design standards used in the development of municipal water services evolved historically on account of technological, socio-economic and most importantly political changes. The standards listed in Table 1.3 below should be considered when evaluating the performance of a municipal sanitation system.

Table 1.3. Design standards used in the development of municipal sanitation services

Standards known as	Official title of the document	Published by (year)
Blue Book	"Guidelines for the Provision of Township Services in Residential Townships"	CSIR
Green Book	"Towards Guidelines for Services and Amenities in Developing Communities"	CSIR
Brown Book	"Proposed Development Guidelines for Housing Projects"	Used by the Cape Provincial Administration
British Standards	"Manual of British Water Engineering" adapted in South Africa as "Water Supply and Sanitation in Developing Communities"	UK (1983)
Red Book	"Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development" abbreviated as PESART	CSIR (1994)
New Red Book	"Guidelines for Human Settlements Planning and Design" abbreviated as GHSPD	CSIR (2000)

It should be noted that the document entitled "Guidelines for Human Settlements Planning and Design" (or the so-called New Red Book) was prepared by the CSIR under the patronage of the SA Department of Housing as a living document to be updated from time to time. CSIR published updated versions of Chapters 9 and 10 in August 2003. Chapter 10 deals with design guidelines for waterborne sanitation systems, but the issues of inflow/infiltration/exfiltration are only briefly attended. Actual installations of waterborne sewer infrastructure in urban areas are subjected to the SABS code of practice as listed in Table 1.4

Table 1.4. National codes of practice for the SA water industry

SABS code of practice	Description	Abbreviated reference
SABS 090	Code of practice for community protection against fire	SABS 090 (19720
SABS 0120	Code of practice for use with standardized specification for civil engineering construction and contract documents	SABS 0120 (1981)
SABS 0252	Water supply and drainage for buildings. Parts 1 and 2	SABS 02520 (1994)
SABS 0306	The management of potable water in distribution systems	SABS 0306 (1999)
SABS 0400	Code of practice for the application of the National Building Regulations	SABS 0400 (1990)
SABS 1200	National Standardised Specification for Engineering Construction	SABS 1200 (1996)

Source: South African Bureau of Standards (www.standsa.co.za)

1.3.3 Revised municipal by-laws for urban water services development

Water Services Authorities/Providers are obliged to conduct a decision process in the development and particularly the maintenance of waterborne sanitation systems according to the legislation and regulations summarized in paragraphs 1.3.1 and 1.3.2. However, the actual development of a project at various municipal locations is subjected to by-laws developed according to the specific local conditions.

Typical municipal by-laws deal with the general conditions attending the administrative aspects of a project development as well as design guidelines for the whole water services cycle (i.e. water supply and sanitation). Municipal water supply and sanitation services should be seen as one system when determining an optimal course of action in a project development emphasising the whole cycle of municipal water services. Two representative sets of by-laws were overviewed for design criteria in water services and aspects of extraneous flows in waterborne sanitation systems.

Table 1.5. Foremost recently revised municipal by-laws

By law origin	Description	Remarks
City of Tshwane Metropolitan Municipality, Service Delivery Department	General principles and guidelines for the design and construction of water and sanitation systems in the city of Tshwane	Revised August 2003
Johannesburg Water (Pty) Ltd. – Investment Delivery Division	Guidelines and Standards for the Design and Maintenance of Water and Sanitation Services	Revised in 2003

Source: As stated above

1.3.4 Standard approach to condition and performance assessment of sanitation infrastructure

The condition and performance of water services infrastructure assets are key factors in the WSA/WSP delivery obligations. Ongoing assessment of asset conditions and therefore planned maintenance, rehabilitation or replacement, are critical to efficient and sustainable operation of water services subsystems over their designated life-span. Several components of a water services system have to operate on a 24 hour 365 days a year basis. Water/wastewater pipelines operate potentially in aggressive conditions and their performance can deteriorate fast. Decision-making in evaluating the performance and condition of an existing component of a system must commonly address the following issues:

- *Is the component (or module, subsystem, system) operating to its designated (designed) capacity, demands, reliability and/or users needs?*
- *How serious is the problem of aging and deterioration?*

- *What are the reasons for the problem?*
- *Are they the real problems?*
- *What are the ramifications to deal with an identified problem?*
- *What is the probability of failure and its consequences?*
- *How quickly can the problem be rectified?*
- *What is the estimated remaining functional life of the component (or module, subsystem or whole system)?*

To conduct an educated and skillful assessment of the condition and performance of various components of a water services system, good knowledge and essential training in structural and hydraulic properties of various materials are required. The assessor must also be well acquainted with the following definitions and ways of assessing water related assets:

- *Definitions of the aggressive environments,*
- *Methods of material properties assessment used in water engineering,*
- *Definitions of the current level of safety and serviceability,*
- *Methods in estimating future rates of material deterioration,*
- *Definitions related to the minimum acceptable levels of service*

A useful check-list in the first round of assessing conditions and performance is illustrated in Table 1.6 below:

Table 1.6. Key water services asset assessment objectives

Type of assessment	**Assessment objectives and means**
Conduit function	Critical at main interceptors / sewer outfall
Pipe network condition	Visual inspection, internal inspection by CCTV
Water / wastewater quality	Sampling of toxicity levels, surveys of hydrogen sulphide for gases
Level of service	Customer complaints, breaks and blockages, stormwater overflows, flooding of properties
Operational performance	Topographical and flow surveys, infiltration, storm inflow, exfiltration
Specific assessment	Status of rising mains and pumping stations, etc.

1.4 Urban wastewater reclamation and disposal problems

The reality of an increasing emphasis on both wastewater quantity and quality cannot be ignored anymore due to deteriorating quality of the water in many rivers in South Africa as they may be receiving large volumes of untreated or partially treated urban wastewater and stormwater.

Urban wastewater quantity and quality management issues and problems are now equally important for either centralized or decentralized urban wastewater return flows disposal. The largest proportion of urban rehabilitated wastewater in South Africa is used indirectly by the water users depending on surface water from rivers where the natural water has already been blended with hopefully treated urban wastewater. Both quantity and quality of such water is crucial to the wellbeing of other water users located downstream of the sources discharging wastewater and stormwater.

WSAs/WSPs providing wastewater treatment and disposal services primarily from centralized WWTPs are facing a struggle in adjusting their present treatment and disposal processes to the more stringent new national parameters for wastewater reclamation.

Table 1.7. Typical pollutant load and treatment requirements in South Africa

Pollutant load	Treatment requirements
Suspended Solids (SS - kg/day)	No reaction to chemical treatment, needs retention time and treatment capacity to reduce the load at least to 20 mg/ℓ
Chemical Oxygen Demand (COD)	No reaction to chemical treatment, needs retention time and treatment capacity to reduce the load at least to 65 mg/ℓ
Nitrates (N kg/day) Ammonium Nitrogen	Needs retention time and treatment capacity to reduce the load at least to 15 mg/ℓ and 3 mg/ℓ respectively
Phosphates (P kg/day)	Chemical treatment with added ferric chloride
E.Coli	Chemical treatment with added chlorine

Source: NWA, Act 36 of 1998

The more stringent standards which have emanated from implementing the NWRS (1999) will inevitably increase the costs of urban wastewater treatment and disposal. To meet the required reductions, numerous municipal WWTPs will have to be rebuilt or rehabilitated by advanced process technology.

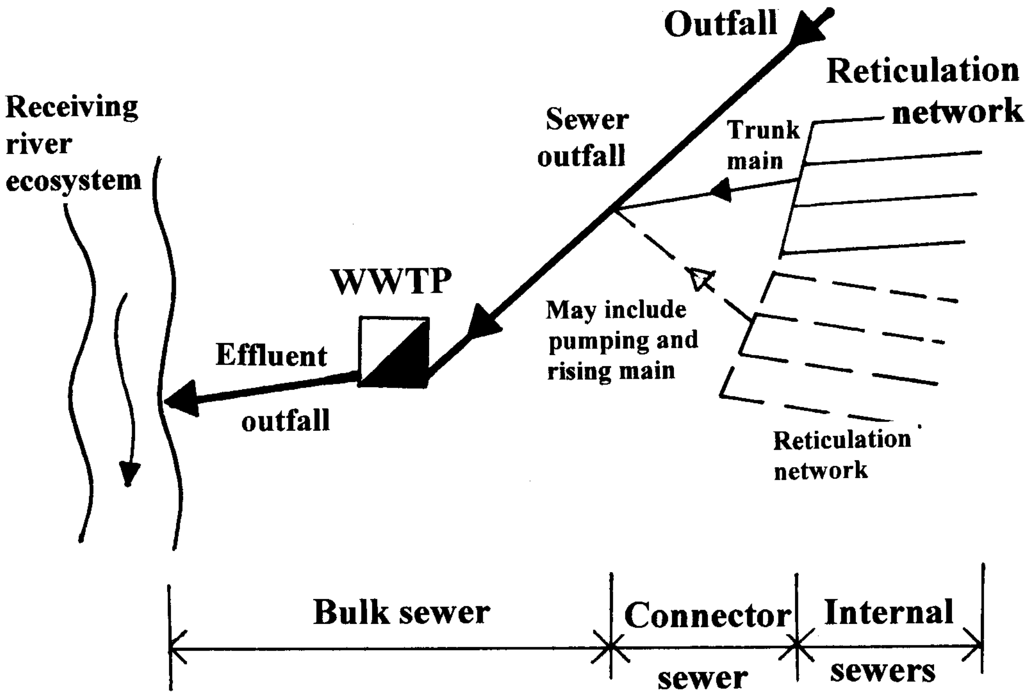


Figure 1.3. Layout of key components of waterborne sewer system

CHAPTER 2. URBAN WATERBORNE SANITATION SYSTEMS

2.1 Typical urban water services systems

The development and management of urban water services (i.e. water supply and sanitation) are controlled by the Water Services Act (Act 108 of 1997). Typically, the provision of sanitation to a community should take place in terms of Water Services Development Plans (WSAPs) which are required to be compiled by the relevant Water Services Authority (i.e. municipality) serving a community. There are at present 253 WSAs in South Africa appointed under the Municipal Systems Act of 202.

If urban water services are looked at in a much broader context, the National Water Act (Act 36 of 1998) also applies. Consequently, two types of water services systems, half cycle and full cycle, are recognized in the urban context.

2.1.1 Half-cycle urban water services system

The half-cycle system is represented by a water services system with primary functions including water treatment, supply and distribution. The sanitation processes represented by wastewater collection, treatment and disposal of treated effluent as return flows are not installed or partially installed without a waterborne component in this type of system.

The wastewater generated is disposed primarily by means of septic tanks and soakaways or as grey water (or sullage, defined as all domestic wastewater other than toilet water) typically onto the ground. According to the Red Book (revised version of August 2003), the following groups of sanitation systems are typically installed in a half-cycle urban water services systems.

- *Group 1* – No water added but conveyance required (e.g. chemical toilets, bucket collection and transport to oxidation ponds, etc.)
- *Group 2* – No water added and no conveyance required (e.g. ventilated improved toilets, ventilated improved double-pit toilets, ventilated vault toilets, urine-diversion toilets, etc.)
- *Group 3* – Water added and limited conveyance required (e.g. septic tank and soakaways, etc.)

2.1.2 Full-cycle urban water services system

(i) Physical linkage of water/wastewater flow

The complete (or full-cycle) water services system is defined as the flow of water and the water use linkage between the intake from a natural water resource and the discharge point to the same or another conventional water source (e.g. a river).

The physical system linkage created from water flow between the water supply input point and the treated effluent output point consists of physical infrastructure components allowing for water storage, conveyance, treatment, supply and distribution of raw or potable water and the collection, treatment and disposal of wastewater, administered typically by the Water Services Authority or Provider on behalf of the community.

Almost all components in a full-cycle water services system are continuously operational and looked after on an on-going basis requiring regular maintenance (preferably preventative) to keep the operational integrity of such a system at its highest level. The WSAs/WSPs administering full-cycle water services systems with conveyance pipeline lengths of over 800 km typically separate the water supply and wastewater collection management functions from each other to keep operation and maintenance at manageable levels.

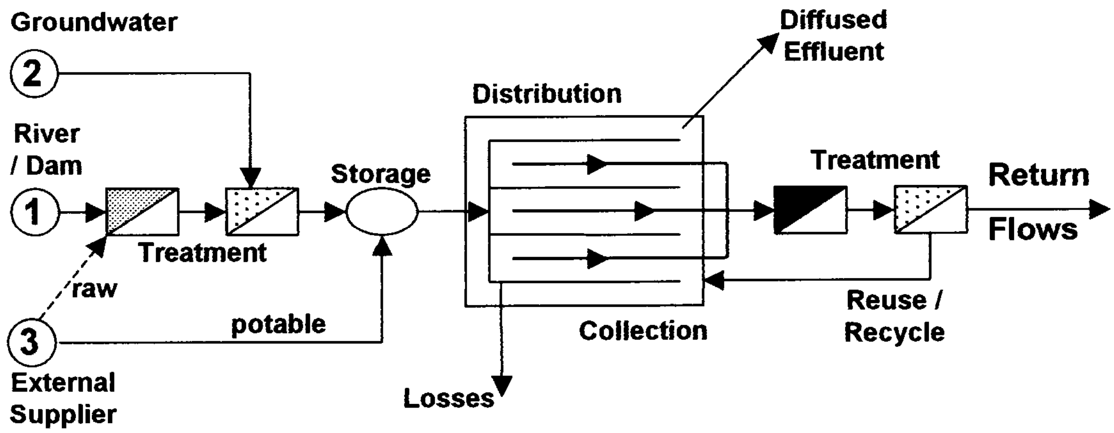


Figure 2.1. Generic full-cycle water services system (also a typical municipal water services system in South Africa)

Any waterborne pipeline system is assumed to be prone to water leakages or extraneous flows. The age of a pipeline, type of materials used and quality of pipeline installation particularly pipe jointing are critical factors influencing the presence of water leakages or extraneous flows.

(ii) Non-pressurised conduit system (NPCS)

There are two types of NPCS that are recognized according to their relevant functions. Both system types are considered non-pressurised most of the time. However, occasionally either of the two can be pressurised for a limited period of time due to extreme events in precipitation and surface run-off.

- *Gravity Wastewater System (WWS)* – is the network of sewer pipes, manholes, pump stations and treatment facilities, collecting waterborne wastes from the built up areas.
- *Gravity Stormwater System (SWS)* – is the network of pipes, channels, retention ponds collecting and disposing of run-off generated from precipitation in the built up areas.

Typically, non-pressurised systems are designed and constructed to drain by gravity stormwater and domestic wastewaters from communities as well as industrial effluent. This type of system, where both stormwater and wastewater are mixed and drained away, is called a *combined system*. If the stormwater is separated from the wastewater, such a system is recognized as a *separate system*. In some instances, a *partially separate system* would be a compromise, allowing for some of the surface runoff to be carried by a wastewater system. In reality, most urban separate sewer systems are subjected to this compromise due to either negligent or illegal diversion of stormwater into a wastewater system. Groundwater infiltration into a pipeline network wastewater systems is a common problem with non-pressurised systems.

2.2 Urban combined and separate waterborne sanitation systems

In most European and North American countries where rainfall is received in the form of light showers or drizzle for an extended period or periods, it has been the established practice to combine both the stormwater and sewer drainage systems into a combined sewer system. This practice may have been copied over to countries with different rainfall patterns. In South

Africa, the installation of a separate sewer system suits the local precipitation conditions manifesting commonly in the form of thunderstorms or storms of short duration and high intensity. Such patterns result in much higher peaks than base flow.

2.2.1 Combined waterborne sanitation system

A combined waterborne sanitation system carries both wastewater and stormwater together in the same pipeline network. During dry weather periods, the system carries predominantly wastewater flow. The flow in the sewer will increase considerably as a result of collecting stormwater during rainfall. The WWTP has to be designed to cope with a certain amount of combined wastewater and stormwater flow. The combined sewer overflow (or CSO) structure has to be built within the system to provide for extreme flows.

2.2.2 Separate waterborne sanitation system

In separate systems, wastewater and stormwater are carried in separate pipelines which are usually laid side-by-side. Stormwater does not mix with the wastewater and is usually discharged directly into a river ecosystem at suitable locations.

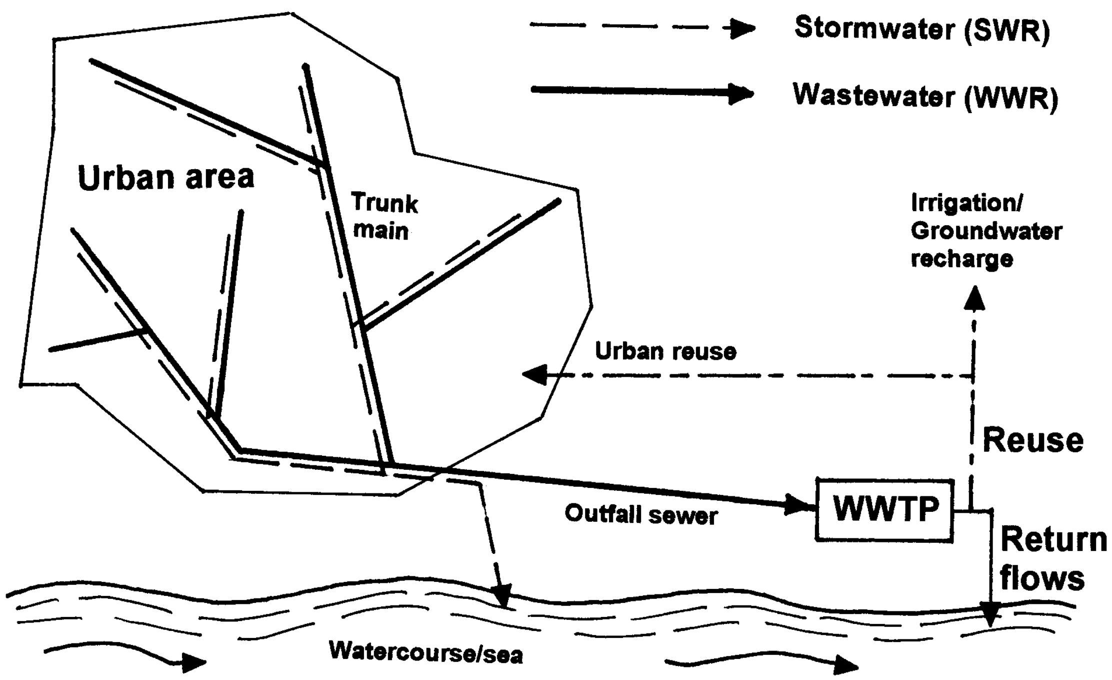


Figure 2.2. Hypothetical layout of separate wastewater from stormwater system

Practically all the municipal waterborne sanitation systems installed in South Africa are separate systems. However, a key drawback of separate systems is that perfect separation of wastewater and stormwater is almost impossible to achieve. Extensive inflow/infiltration into the separate systems can mitigate the function of separation and such a system becomes a hybrid system. Table 2.2 illustrated the key advantages and disadvantages in the development of separate waterborne sewers.

Table 2.2. Summary of advantages and disadvantages of separate systems

Advantages	Disadvantages
• *Smaller WWTP* • *Collection sewer pipe smaller maintaining greater velocities* • *Less variation in flow and strength of wastewater* • *Limited surface area grit in collected wastewater*	• *Extra cost of two pipes* • *Additional excavation space and volumes* • *More house drains with risk of wrong connections* • *No regular flushing of wastewater deposits* • *No treatment of stormwater*

Source: Butler and Davies (2000)

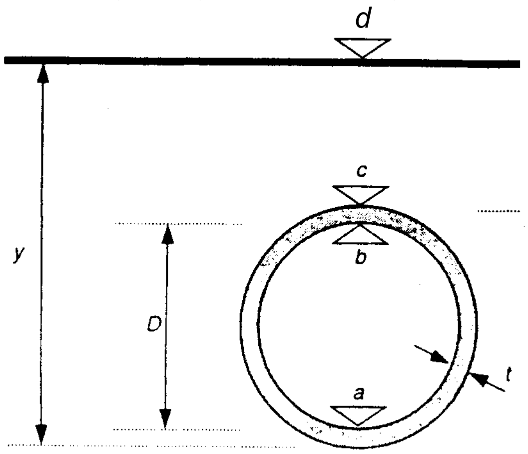
2.2.3 Hybrid waterborne sanitation systems

A hybrid municipal waterborne sanitation system can be defined as a system with a combined system as its core and upper drainage areas drained by separate subsystems. Illegal stormwater connections and cross-connections, together with excessive stormwater inflow and groundwater infiltration will effectively convert a separate system into a hybrid system over time.

2.2.4 Waterborne sanitation system components

(i) Sewer pipe

Most modern sewers comprise circular pipes commonly from 100mm to 2500mm in diameter, typically of vitrified clay, concrete, asbestos-fibre cement and uPVC.



D = internal diameter,
a = invert level (IL),
b = soffit level,
c = crown level,
d = ground level,
t = pipe wall thickness
y = minimum excavation depth (add trench bedding)

Figure 2.3. Terminology associated with sewer pipe

(ii) Manholes

Manholes are required for testing, inspection, cleaning and changes in direction, gradient or sewer pipe size.

(iii) Pumpstations

Typically, most waterborne sewers are installed as gravity flow systems. However, in some locations there is a need to pump the wastewater to a different elevation. The pumping equipment typically operates on a stop-start basis. The pumping system characteristics are determined from the principle: head = static lift + losses and velocity head.

(iv) Wastewater Treatment Plant (WWTP)

The purpose of wastewater treatment is to rectify the quality of the wastewater collected from the urban area before it is released into the receiving water source. The return flows must comply with relevant legislation and environmental requirements. The processes taking place in a typical WWTP are as follows:

- *Preliminary treatment*. This process removes gross solids from the wastewater flow (e.g. sand and grits). Excessive stormwater should be separated from wastewater to protect other processes.
- *Primary treatment*. The function of primary treatment involves sedimentation of organic load on the plant.
- *Secondary treatment*. This phase of treatment introduces biological oxidation to remove remaining organic load.
- *Tertiary treatment*. This is an optional function in the overall process introducing further reduction of residual suspended solids and associated BOD to produce a high-quality effluent for disposal into sensitive ecosystem or reuse.
- *Sludge treatment and removal*. The sludge is a residue collected from the treatment processes, dewatered and treated prior to disposal. The cost of sludge disposal is a major factor in WWTP operational costs.

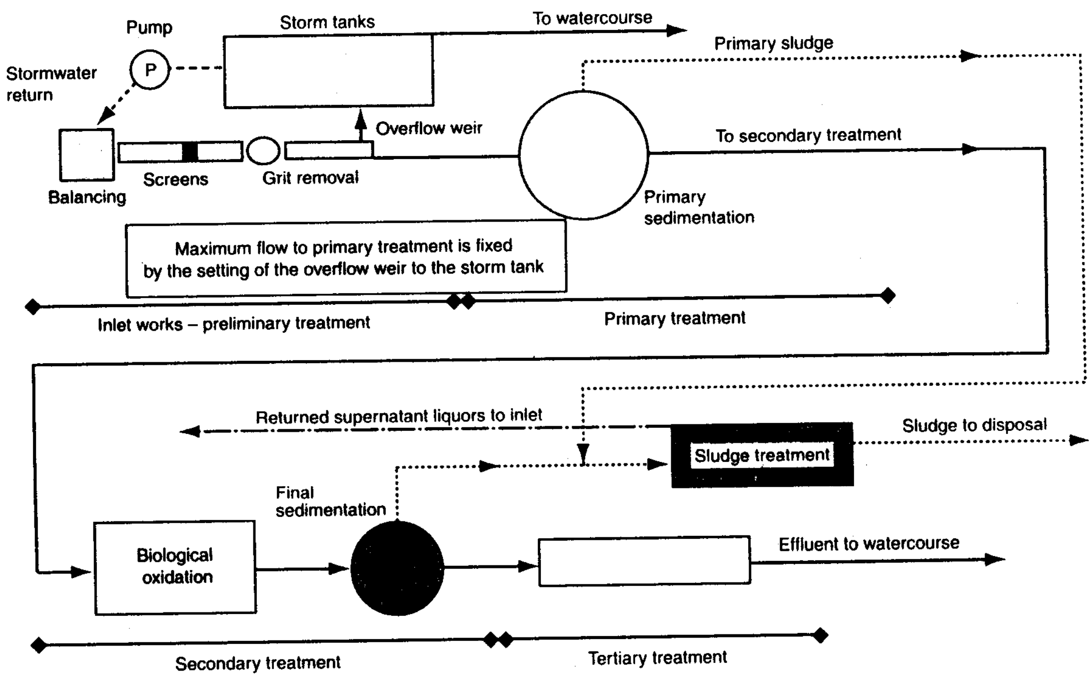


Figure 2.4. Layout of a wastewater treatment plant (WWTP) major process modules (after Rendell, 1999)

CHAPTER 3. CRITERIA FOR DESIGN AND EVALUATION OF A WATERBORNE SANITATION SYSTEM

3.1 Current design and construction criteria

3.1.1. General methodology in determining wastewater flow rates

(i) Flow theory in a non-pressurised conduit system

Sewerage and treated sewer effluent are both Newtonian fluids with a low viscosity. This means that most of the principles for raw and potable water also apply to sewer flow theory. The head loss due to friction in a full flow sewer can be calculated with the Darcy-Weisbach equation. The pipe friction factor can be obtained from the Colebrook-White formula. The Colebrook-White equation is now regarded as the most satisfactory basis for hydraulic design of both water supply and sewerage pipelines.

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{k_s}{3{,}7D} + \frac{2{,}51}{\mathrm{Re}\sqrt{\lambda}}\right] \tag{3.1}$$

Where: λ = friction factor
k_s = surface roughness
D = pipe diameter
R_e = Reynolds's number

It should be noted that various textbooks provide tables and charts of flow rates (ℓ/s or m^3/s), flow velocities (m/s) and hydraulic gradients, typically for pipe diameters from 0.025 to 2.50m. The recommended roughness factor, k_s, is normally also listed. In the case of short pipelines, extra allowance must be made for discontinuities such as change in size, direction, installed valves and junctions, etc.

The transport of biological matter in sewers can result in sliming of the pipeline below the surface of the water, thus reducing flow velocity and changing the characteristics of the pipeline parameters. The capacity of a sewer flowing full can be determined from the Manning equation:

$$Q = (1/n)R_h^{2/3}S^{1/2} \tag{3.2}$$

Where: Q = flow in sewer pipe (m^3/s)
n = Manning's constant
S = friction slope (if the flow depth does not change with distance, S is numerically equal to the slope of the sewer)
R_h = hydraulic radius = cross-sectional area/wetted perimeter

It should be noted that the main flow pipelines within the WWTP are considered sewers until after the secondary stage of treatment. The friction factors for the pipes carrying sludge are functions of the characteristics of the sludge.

(ii) Partially full sewer pipes.

For a pipe flowing partially full, D in the Colebrook-White equation (3.1) is replaced by $4R_h$, the hydraulic radius of the pipe given by the quotient of the cross-sectional area of the water in the pipe and the wetted perimeter.

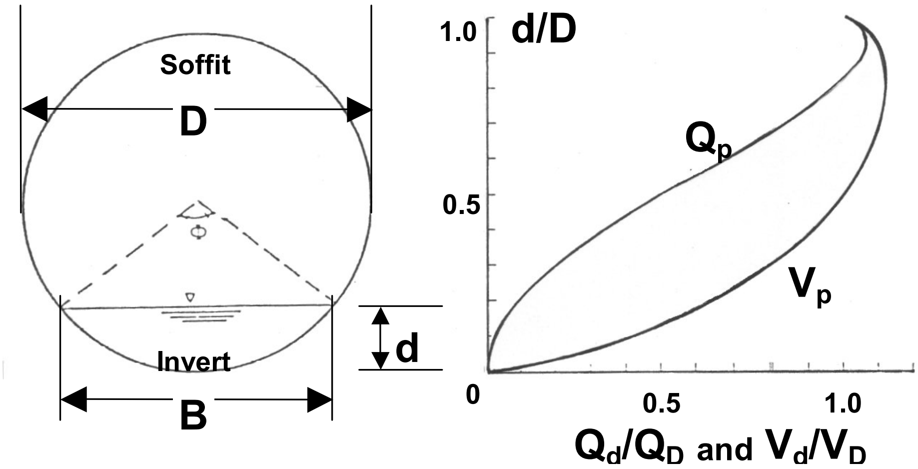


Figure 3.1. Pipe running partially full

For a partially full pipe condition, estimates of discharge and velocity are needed in order to check if self-cleansing velocities are maintained at the minimum discharge. Self-cleansing velocities are most important in the design and operation stages of a partially full pipe network with high loads of suspended solids.

The free flow surface in a partially full pipe introduces a modification to the Colebrook-White equation:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{k_s}{3{,}7*4R_h} + \frac{2{,}51}{Re\sqrt{\lambda}}\right) \tag{3.3}$$

Where: $Re = 4R_h V / \upsilon$

Using the Darcy-Weisbach equation and a pipe gradient of $S(S = h_f / L)$, the following applies:

$$v^2 = 2g*S*D/\lambda \tag{3.4}$$

Subsequently, for a pipe with partially full flow:

$$v = (2g*S*4R/\lambda)^{1/2} \tag{3.5}$$

Using a further ratio $v_d/v_D = v_p$ for proportional value of a partially full pipe (d) and the full pipe depth (D) is:

$$v_p = \lambda_D^{1/2} * R_p^{1/2} / \lambda_p^{1/2} \tag{3.6}$$

Similarly, $$Q_p = \lambda_D^{1/2} * A_p * R_p^{1/2} / \lambda_d^{1/2} \tag{3.7}$$

Where for a circular pipe:

$$A_p = \left(\frac{\phi - \sin\phi}{2\pi} \right) \tag{3.8}$$

$$R_p = \left(\frac{1 - \sin\phi}{\phi} \right) \tag{3.9}$$

The relevant substitutions to the above formulae as well as grades giving essential values can found in textbooks such as, for example, by Chadwick and Morfett (1993) and White (1987).

(iii) Flow velocities

A minimum of 0,6m/s should be maintained in all gravity wastewater mains to ensure that sufficient scouring of the mains can take place. Maximum flow velocity under full flow conditions should not be more than 2,5m/s to prevent damage to pipelines, although up to 4,0m/s velocities may be permitted for a short period. In the rising mains, a minimum velocity of 0,6m/s is recommended to prevent deposition of solids. To avoid a water hammer problem, the maximum flow velocity should be limited to 1,8m/s.

(iv) Hydraulic capacity of sewers

The wastewater system capacity is based on the assessment of essential parameters including the dry weather flow (DWF), average dry weather flow (ADWF), peak dry weather flow (PDWF) and peak wet weather flow (PWWF) as well as estimates of groundwater infiltration and stormwater inflows. Based on CIRIA Report No. 177 (1998), the DWF of mixed residential and industrial urban area may be defined as follows:

DWF = POP * ADW + INF + IED (3.10)

Where:
- DWF = Dry Weather Flow (m^3/day)
- POP = Population served
- ADW = Average domestic wastewater contribution (m^3/cap/day)
- INF = Infiltration (m^3/day)
- IED = Industrial effluent discharged (m^3/day)

The criteria applied in the WSDPs by the WSAs/WSPs for purposes of planning and evaluation stipulate that the pipe size in sewer gravity mains should be such that the peak dry weather flow is accommodated in the pipeline whilst flowing at 70% or less full. The remaining 30% of the pipe flow area is allocated for stormwater ingress. Should stormwater ingress cause this “spare capacity” to be exceeded resulting in pipe overflow, urgent measures should be taken by the water services authority/provider to prevent illegal drainage of stormwater into the sewer system.

(v) Unit sewage flows

Unit sewer flows in urban areas are determined from sewer flows generated by the different land uses. Typical values for planning and evaluation purposes applied by the WSAs/WSPs in South Africa are illustrated below:

Table 3.1. Typical unit sewage flows for urban areas applied in South Africa

Land use	Wastewater flow (ℓ/day/urban erf (UE))	Remarks on land use
Residential erf: Type 1 Type 2 Type 3	1150 1050 950	High density Medium density Low density
Commercial erven	850	Average value
Light density erven	950	Site specific

Source: DWAF Guidelines on Water Services Development Plans as per WSA (1997)

The unit wastewater flows for urban areas in South Africa as listed in Table 3.1 would be commonly superceded by the relevant values determined in municipal by-laws, as for example referred to in Table 1.5.

The procedure in determining design flow for development (or enhancement) of a new or existing wastewater system is based on the application of a peaking factor in the following equation:

$$\text{Design flow} = PKF * UE + INF + IED \tag{3.11}$$

Where: PKF = Peaking factor (between 1,3 and 2,5) depending on the land type and use (consult New Red Book on attenuation of peak flows)

UE = Average contribution from urban erf as per Table 3.1 (ℓ/day/UE or ℓ/day/dwelling unit as per New Red Book)

IED = Industrial effluent discharge (ℓ/day)

INF = Infiltration of groundwater and leakage from plumbing devices (consult New Red Book for determination procedure)

It should be noted that the peak wet weather flow is determined with an allowance of 1 percent to accommodate extraneous flows (i.e. I/I events).

(vi) Pumping of wastewater

Pumpstation equipment should comply with the following criteria:

- *The installation should comprise at least one standby pump;*
- *Pumping capacity is to equal or exceed the peak wet weather flow which might arrive at the pumpstation; and*
- *The sump of the pumpstation should be sized to ensure that the pump does not switch on and off more than six times per hour.*

The pumping stations installed in a waterborne sewer system are almost always equipped with electrically driven automatic pumps operated from level measuring devices or switches in the reception sump, enabling them to operate without full-time pump attendance. Sewage pumps are normally centrifugal or mixed flow devices. Submersible sewage pumps are also frequently installed.

Where the flow of wastewater varies continuously, balancing capacity has to be provided for a constant speed pump by the suction well. If more than one pump is installed, starting levels are arranged so that pumps are started in succession as the flow increases. As hydraulic surges are inevitable in a sewer pipeline, they are typically lowered by the dissolved gasses contained in the sewerage fluids.

3.1.2 Approach to design flows for sewer capacity in specific urban areas

Historically, sewer design flows for the purpose of urban wastewater collection were in many instances based on the City of Johannesburg Wastewater Department: Township Sewer Design Standards/ Procedures (1974). According to this procedure, the basic unit of allocation is the equivalent discharge unit (EDU) which is equivalent to 100 ℓ/day. The EDU values are based on zoning and stand areas since this allows flexibility in the allocation procedure and a closer calibration to actual flows experienced in the system.

This method has also been used by the City Engineer's Department of the Ekurhuleni MM and is not necessarily applicable to other urban areas. The method sets a fixed EDU value for stands up to 1000 m^2. It is assumed that a stand area greater than 1750 m^2 can be subdivided. It is further assumed that the developed area is 80% of the gross area of a stand less than 40 000m^2 and 60% of gross area for a stand greater than 40 000m^2. The allocation of flow based on this approach is shown below:

Table 3.2. Flow allocation for "Residential 1" stands

Stand Area	**EDU Value (100ℓ/day)**
0 – 200 m^2	1
200 – 300 m^2	3
300 – 500 m^2	4
500 – 800 m^2	5
800 – 1000 m^2	6
1000 – 1750 m^2	(420 x ln(Area) – 2200)/100
1750 – 40 000 m^2	Area x 0.008
> 40 000 m^2	Area x 0.006

The approach in flow allocations for other than "Residential 1" zoning is illustrated in Table 3.3 below.

Table 3.3. Flow allocation for "Residential 2, 3 and 4" stands

Zoning	**EDU Value (100 ℓ/day)**
Residential 2	Area x 0.012
Residential 3	Area x 0.012
Residential 4	Area x 0.015

The domestic wastewater outflow from a house connection for sewer design is assumed typically at 1.0 ℓ/min (0,02 ℓ/s). The actual peak discharge is considerably higher, and the cumulative effect of the number of houses should be considered. In South Africa, a typical toilet flush is determined at a rate of 20ℓ in 7 seconds (i.e. 3ℓ/s).

3.1.3 Definitions and criteria for regional sewers

The following definitions and criteria are currently applicable for the regional sewer investigation by metropolitan authorities and ERWAT:

(i) Peak Flow Factor

The peak flow factor (PKF) is the ratio of the expected peak design flow (PDF) to the calculated average daily flow (ADF) calculated by the formula developed by Harman (1918):

$$PKF = 1 + [14/(4 + \sqrt{}POP] \tag{3.12}$$

where POP = population in thousands

(ii) Municipal Outfall Sewer

A municipal outfall sewer is the main sewer which links a developed area (minor or sub-drainage district) with a regional sewer or the water care works.

(iii) Pipe Capacity

The peak design flow will be taken as 60% of the full bore capacity of the pipe, to provide for infiltration and unforeseen peak flows.

(iv) Monitoring Station

A monitoring station is a flow and load measuring point on a regional sewer or at a water care works. The data obtained form a monitoring station will be used, firstly as input for the Technical Information System, and secondly to determine the flow and chemical load from a contributing town.

(v) Regional sewer

For a municipal sewer to be classified as a regional sewer, the following applies:

- *An outfall sewer line serving two or more local authorities or major contributors. The minimum flow contribution from any one contributor should not be less than 10% of the total flow in the pipe.*
- *An outfall sewer line with no other main sewer lines connecting to it, between the last connection and the water care works.*
- *An outfall sewer line with an internal diameter larger than 500m.*
- *The minimum length of a regional sewer shall be 500m.*

Due to the changes taking place within the local authorities environment, it is recommended that to the above listed criteria, the following should be added:

- *An outfall sewer can be classified as a regional sewer by negotiation, and the conditions for a regional sewer may vary from time to time*.

(vi) Existing outfall sewers

The aim of designer's is to identify which sections of existing outfall sewers can be, according to the criteria above, classified as regional outfall sewers.

(vii) Future regional sewers

It should be the aim of each WSA/WSP to identify future regional sewers under the requirement of Integrated Development Plans procedures. The planning process for development of future regional sewers should allow for flow and chemical load measurements to be monitored for the whole wastewater system. The aspect of location of monitoring stations should also be addressed during each investigation.

3.2 Differences in function of water supply and wastewater systems

3.2.1 Relationship between water supply and wastewater flows

Generally, there is a strong link between water supply and wastewater collection and disposal in urban water services systems. Theoretically, almost all water supplied into a system can be collected and returned after treatment back into receiving waters. However, practically, a considerably portion of water supply does not reach the waterborne collection system. This includes mainly water used for street washing, lawn sprinklers, fire fighting and leakages from water mains and service pipes. A small portion of the water supplied may also be consumed in products and manufacturing processes (i.e. so-called consumptive water use).

In a developing country such as South Africa, many households might not be served by a waterborne sewerage collection system and a large amount of the water supplied will end up after use as sullage, disposed of through septic tanks or collected as grey water and re-used. On the other hand, inflow/infiltration and water obtained from private water sources (e.g. boreholes) may make-up the larger quantity of wastewater after collection. The average wastewater outflow may vary from 60 to 130 percent of the water supplied into a system.

3.2.2 Wastewater return factor

In a simple way, the relationship between water supply and wastewater generated in urban areas can be represented as follows:

$$WW_{generated} = r * W_{supplied} \qquad (3.13)$$

Where: $WW_{generated}$ = wastewater generated from a household and/or industry (ℓ/unit/day)
$W_{supplied}$ = water use per household and/or industry (ℓ/unit/day)
r = return factor (see Table 3.4 for typical values recognized internationally as averages)

The water losses for various facilities in urban and industrial water use categories may be estimated by adopting the relevant return factors.

$$W_{loss} = (1 - r) * W_{supplied} \qquad (3.14)$$

Table 3.4: Typical return flow factor for residential and industrial water use

Category	Appliance or unit	Volume per use	Return factor (r)
Residential/domestic	Toilet	9 ℓ	1,0
	Bath	75 ℓ	1,0
	Shower	40 ℓ	1,0
	Wash basin	4 ℓ	1,0
	Kitchen sink	7 ℓ	1,0
	Washing Machine	120 ℓ	1,0
	Car washing	Occasional	zero
	Garden use	Varies seasonally	zero
Commercial	Offices	65 ℓ/employee/day	0,95
	Stores	100 ℓ/employee/day	0,95
	Hospitals	400 ℓ/bed/day	0,98
	Hotels	600 ℓ/bed/day in hot climate	0,98
Industries /manufacture	Brewery/soft drinks	7000 ℓ/m^3	0,5
	Cheese making	3000 ℓ/t	0,65
	Fish processing	15000 ℓ/t	0,65
	Electrical products	1500 ℓ/m^2	1,0
	Small car	5000 ℓ/unit	0,8
	Bicycle	130 ℓ/unit	0,8
	Pair of shoes	55 ℓ/unit	0,9
	Textiles	250 m^3/t	1,0

Sources: CIRIA (1998), Schutte and Pretorius(2000)

3.2.3 Ratio of wastewater discharged to water supplied

Palmer Development Group (1998) analysed and summarized work of CSIR (1983) and Singles (1991) conducted in various urban areas of South Africa on the production of wastewater from the residential water use categories. A sample of 480 houses in Pretoria (Tshwane MM) yielded a mean external water use of 55 percent on properties of between 900 and 1000 m^2. Similar analyses were conducted for Port Elizabeth and Cape Town with results of 33 and 25 percent respectively.

Table 3.5. Ratio of wastewater discharged to water supplied (ℓ/unit/day)

Household category	Water supplied		Wastewater produced		Ratio (%)	
	House	Flat	House	Flat	House	Flat
Low income	470	410	395	394	84	96
Middle income	900	580	568	566	63	98
High income	2470	960	936	933	38	97

Source: WRC Report No. TT98/98

The ratio of volumes of wastewater treated to water supplied is used in determining billing tariffs as well as marginal costs of operation, maintenance and repair (OMR). By recognizing the interaction between supply of water and generation of wastewater, it is possible to maximize the total benefits to the community by jointly taking decisions on water price, magnitude and timing of rehabilitation/expansion to system capacity. The OMR costs for water supply, transport and distribution, wastewater collection, treatment and disposal are typically influenced by the ratio described above.

3.2.4 Return flows to receiving river ecosystem

Both quality and quantity of return flows into receiving river system are critical for the users situated downstream of urban areas. In principle, the return flows generated by a particular urban area (or consumption centre) is the difference between the supply and the consumptive use (i.e. all water lost in a system due to various processes based on use of water), if inflow/infiltration or exfiltration are not considered in the analysis of a system.

For the purposes of the Water Situation Assessment Model (WSAM) used by DWAF in determining deficit/surplus of water in a river catchment context, consumptive water use coefficients were determined for eight urban water use categories. The coefficients adopted by the DWAF are illustrated in Table 3.6 below:

Table 3.6. Consumptive water use coefficients

Coef.	Category of use	Default value for PL_i	Comments
PL1	Full services: houses on large erven >500 m^2	0,45	The value is primarily a function of basic human consumption, out-door garden watering, pool evaporation and re-fills, cleaning of paved areas, car wash, etc.
PL2	Flats, town houses, cluster houses with full service	0,20	Lower than Category 1, because of reduced outdoor water use due to the corporate share of each individual water user in the complex
PL3	Full service: houses on small erven <500 m^2	0,35	Higher than Category 2 due to the size of residential stand and socio-economic standing of water users under this category
PL4	Small houses, RDP houses and shanties with water connection but no or minimal sewerage service	0,80	The value is primarily a function of basic human consumption, garden watering, cleaning of paves areas and services car wash. However, the critical component is the locally diffused domestic effluent increasing the size of overall consumptive water use
PL5	Informal houses and shanties with service by communal tap only	1,00	As per Category 4, however the default value is defined by the extent of sanitation services that, in this case, is assumed to be non-existent
PL6	Commercial/ institutional water use	0,15	This default value is based on an assumption that most commercial establishments will consume water for basic human consumption and corporate outdoor landscape watering
PL7	Industrial water use	0,40	This default value is most difficult to determine as it might vary considerably for different industrial sectors. The value is assumed to be higher than the default for Category 8 because industrial effluent is often recycled
PL8	Municipal water use	1,00	This default value is based on an assumption that water use as fire-fighting, sports ground and street watering, temporary construction water supply, etc., are fully water consumptive

Source: Barta (1999)

The consumptive water use is a function of all water supply and sanitation socio-economic restraints. It is also affected by climate variations and water management factors such as tariff structure and water restrictions. The water that is not consumed becomes wastewater and is discharged into the sewer system, collected, treated and either reused or returned for further indirect use reuse. The return flows from an urban area can be calculated as follows:

$$RFU_i = UWU_i * (1 - PL_i) \tag{3.15}$$

Where: RFU_i = Return flow from urban user in category i
UWU_i = Urban water use in category i
PL_i = Coefficient of consumptive water use for a specific user category i.

The coefficients listed and described in Table 3.6 may be used to determine wastewater quantities generated in the inland urban areas of South Africa. Urban water use in each category is usually location specific and should be determined or measured for each specific urban area.

3.2.5 Relation between water supply and wastewater diurnal patterns

Wastewater flows in a waterborne sewer vary similarly to water demand according to the season of the year, weather conditions, day of the week and time of day. Under dry weather conditions, the daily wastewater flow will show a diurnal pattern. The time required for wastewater to reach the WWTP is an important factor. Generally, commercial and industrial discharges tend to reduce peak flows.

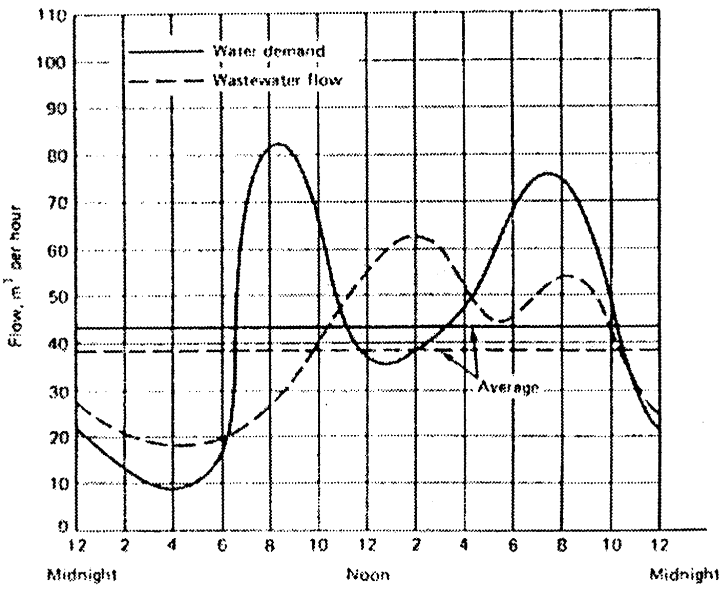


Figure 3.2. Relationship between water supply and wastewater diurnal patterns

3.3 Methods available in determining extraneous flows

3.3.1 Standard evaluation of infiltration into the ground and sewers

The sewers built in urban areas usually follow the watercourses in the bottom of a valley close to (and occasionally below) the bed of a stream. As a result, these sewers may receive comparatively large quantities of groundwater, whereas sewers built at high elevations are likely to receive relatively small quantities of groundwater. With an increase in the percentage of areas paved or built over, comes an increase in the percentage of stormwater flowing rapidly into the storm sewers and watercourses. A decrease in the percentage of the stormwater that can percolate into the earth will take place, however thus tending to increase surface inflows to wastewater sanitation sewers.

The rate and quantity of infiltration depends on the length of the sewers, the area served, the soil and topographic conditions, and, to a certain extent, also the population density (which affects the number and total length of house connections). At some locations, the elevation of the water tables may vary with the quantity of rain percolating into the ground, the leakage through defective joints and porous concrete. The cracks may be so large, in some cases, to lower the groundwater table to the level of the sewer.

The amount of groundwater flowing from a given area may vary from a negligible amount for a highly impervious area or an area with a dense subsoil to 25 or 30 percent of the rainfall for a semi-pervious area with a sandy subsoil permitting rapid passage of water.

3.3.2 Stormwater inflow into sewers (or direct stormwater inflow)

Direct stormwater inflow can cause an almost immediate increase in flow rates in waterborne sewers. The effects of inflow on peak flow rates that must be handled by a wastewater treatment plant could be up to 5 times higher than average dry weather flow (ADWF). Direct inflow rates are usually determined by using a network of continuous flow meters that operate before and during a significant storm.

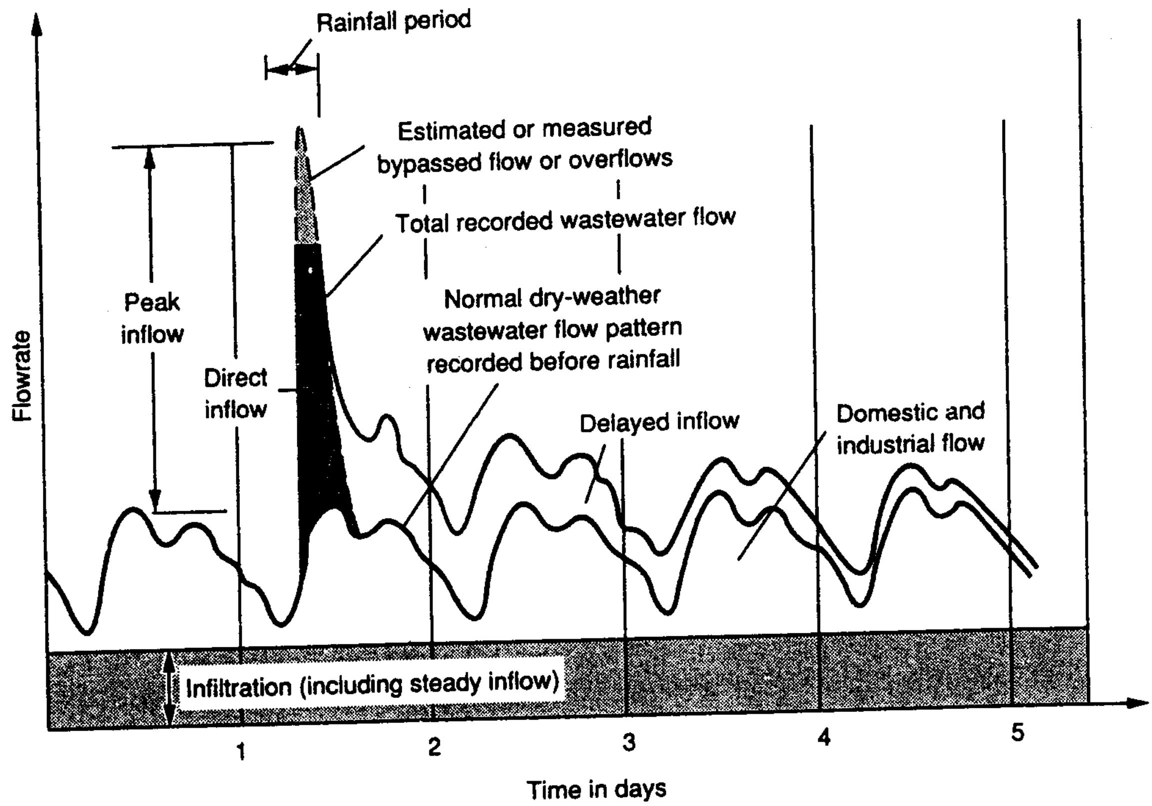


Figure 3.3. Representation of I/I events (after Metcalf and Eddy, 1991)

The stormwater inflow rate can be determined from the flow hydrographs recorded with the flow meters by subtracting the normal dry weather domestic and industrial flow and the infiltration (including steady flow) from the measured flow rate. Useful flow hydrograph methods are for example unit hydrographs, synthetic unit hydrograph, time area diagrams and reservoirs connected in series may be applied in determining the characteristics of the catchment where is the waterborne sewer situated.

It will usually be found from field surveys that only a small part of a collection system contributes most of the infiltration/inflow. As a general rule, about 75 percent of the inflow comes from 20 to 30 percent of the system, whereas 75 percent of the infiltration comes from 40 percent of the area.

- *Total inflow*. This flow is represented by sum of the direct stormwater inflow at any point in the system, plus any flow discharged from the system upstream through overflows, pumping station bypasses and the like.
- *Delayed inflow.* This is stormwater that may require several days or more to drain through the sewer system, including the discharge of sump pumps from cellar drainage as well as the slowed entry of surface stormwater through manholes in ponded areas after heavy rainfall events.

3.4 Factors contributing to inflow/infiltration in urban areas

3.4.1 Type and status of wastewater collection and stormwater drainage

Options of waterborne collection of wastewater typically installed in South Africa are as follows:

- *Septic tanks and soakaways* – this is an on-site option where normal toilet waste, often together with grey water from the house, flushes to a buried tank which discharges into a soakaway (or French drain).
- *Simple waterborne sewers* – this type of waterborne collection of primarily residential wastewater (e.g. shallow sewers) would be considered for highly dense informal communities. However, installation of a simple sewer system requires relaxation of several design characteristics of conventional waterborne sewerage allowing primarily for shallower depths (e.g. only 400 mm cover), smaller sewer pipe diameter (i.e. less than 150 mm) and flatter sewer gradients (e.g. 1:167 slope for 100 mm pipes). Intervals between manholes are allowed up to 100 metres. Although shallow sewer installations are becoming popular in Brazil, India and Pakistan, only a few systems have to date been installed in South Africa (e.g. Ethekwini, WRC, TT 225/04).
- *Septic Tank Effluent Drainage (STED)* – this is another "simple" waterborne sewer based on the concept of "solids-free" sewerage. A septic tank (or aqua-privy tank) is erected on site and the effluent from this tank flows to a collector sewer for treatment at the WWTP.

To date, practically nothing is known about the performance and problems associated with inflow/infiltration of simple waterborne sewers.

- *Full waterborne sanitation* – Inflow/infiltration may take place in all key components of a sewer system (i.e. internal reticulation, connector and bulk outfalls). There is a strong association between waterborne sewerage collection and the treatment technology installed. Conventional fully waterborne systems would typically adopt the bio-filter or activated sludge treatment technology.
- *Urban runoff drainage* – Stormwater retention, groundwater recharge, provision of rough surfaces to retard flow and disconnection of impervious areas, are now practices which are gradually being applied in municipal urban drainage systems.

(i) Inadequate stormwater retention or detention

Retention implies on site recharge or evaporation whereas detention is a temporary retaining and subsequent release of the excess flow. The day-to-day management of urban catchments have an important bearing on the runoff quantity and quality as well as inflow/infiltration events. On-site detention or retention and regular cleaning could relieve the drains considerably of an excessive load. A proactive maintenance management programme should be proposed at the time of design. Negligence to maintain this programme will result in exceedance of the capacity of the system.

Where stormwater infrastructure (including secondary road channel drainage) is inadequately maintained, or not properly planned or installed, the capacity of the existing system may become inadequate and unable to cope with high intensity rainfall (long return flood periods) resulting in flooding gullies and flooded homesteads especially on a sloping landscape.

(ii) Common factors that relate to high stormwater flooding

- *Slope of a catchment or drainage basin* - the central reach of a catchment can be fairly flat with slopes of less than 3% and varies between 3% and 6%. Regarding the position of a river in a catchment, its drainage channel does not have very steep valleys and riverbanks. The topography therefore lends itself to easy flooding as a result of small changes in the water flow sheet or slight positive changes in the river stage.
- *Gutters* - form part of a house roof stormwater drainage system and due to practices adopted at many private properties the downpipes from the house roof are linked to the sewer gullies.
- *Paving of yards* - has become a convenient way of keeping homes looking splendid all year round. However, this practice increases the quantity of water to be drained and therefore the height of the flowing water sheet. To get rid of this water, some individuals construct private manholes later which drain into the sewers or directly into the sewer system. Water is also led into the sewer system via rodding eyes/inspection eyes.
- *Swimming pools* - may also be a contributing factor in that the overflows due to rainfall and their backwash water are linked directly or indirectly to the private sewer drainage system.
- *Excessive littering* - increasing pollution is the most obvious result of failing urban drainage management. The consequences of neglected street sweeping and inefficient refuse disposal and removal are leading to capital requirements elsewhere within urban water services systems (e.g. extensive treatment costs).

3.4.2 Treatment of additional quantities of wastewater at WWTP

(i) WWTP without stormwater detention facility

Wastewater treatment plants are designed for short duration of inflow/infiltration events (about 15% of the Dry Weather Flow), which is normally caused where gullies are channeled into the sewers on properties. The sewer flow pattern with such an extent of I/I events should allow the WWTP to cope without any negative effects on discharged effluent quality.

Due to increases in stormwater inflow and groundwater infiltration, the flow peaks at the WWTP are higher and of a much longer duration. Although extraneous flows are less polluted than sewerage, the effect on treatment works is dramatic as the effluent does not comply with the required standards, and costs of operating the WWTP will certainly increase.

In combined systems, a combined sewer overflow (or CSO) structure is normally introduced to take care of overflows before they reach the WWTP. Improperly maintained separate systems or systems reaching their design capacity generate commonly flows which cause severe problems to the existing WWTP. By-pass storages are built at the WWTP sites to cope with problems of overloading. This means additional capital expenditure and operational/ maintenance costs.

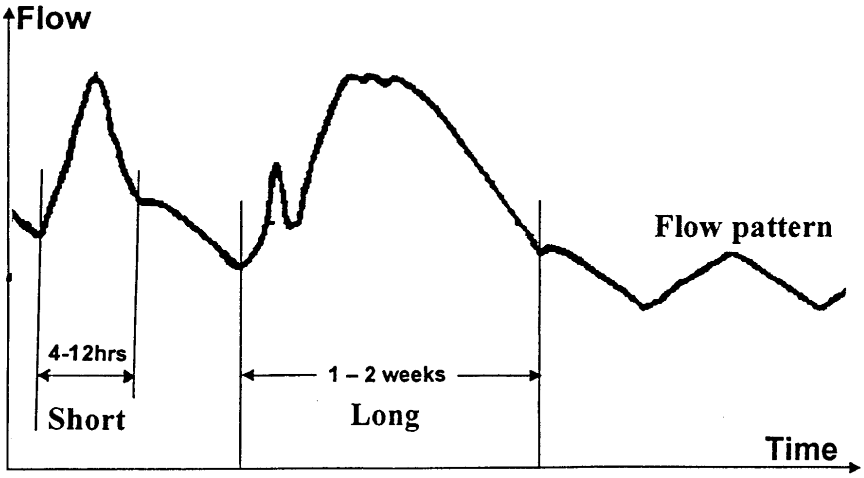
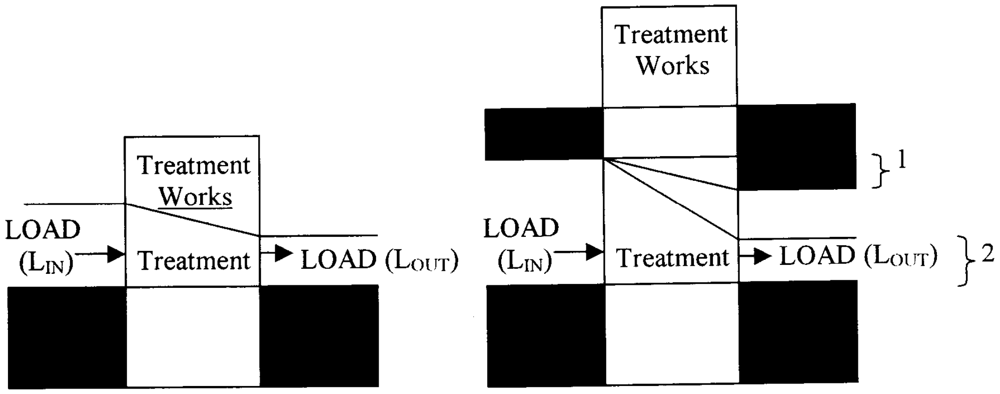


Figure 3.4 The flow pattern due to excessive inflow/infiltration events

Under normal flow conditions, the treatment works will receive a flow (volume of wastewater) with a certain load of pollution. After treatment the pollution leaving the works will be reduced to comply with the effluent standards and very little or no pollution of the receiving water stream is assumed to take place.



Normal condition Excessive condition
1 = untreated, 2 = treated

Figure 3.5. Overloading as a result of excessive inflow/infiltration events

During heavy and extended storms, when infiltration will also increase, an additional volume of water will flow through the treatment works reducing the retention time of the process. This will reduce the effectiveness of the biological process leading to a higher pollution load leaving the WWTP. This additional load leaving the plant is normally higher than the effluent standards and needs to be reduced due to environmental requirements. In terms of legislation, as listed below, it is the duty of WSA/WSP to ensure that no environmental pollution or damages takes place and the load in excess of the effluent standards must be removed.

In terms of legislation and permit requirements for the WWTPs, penalties are payable for non-compliance to effluent standards. The polluter is also responsible for the clean up of the pollution, which can run into thousands of Rands.

Table 3.7. National performance parameters for wastewater reclamation

Critical parameter	Old standards (mg/litre)	New standards (mg/litre)	Required reduction (mg/litre)
COD	65	40	25
SS	20	10	10
Ammonia – N	3	1	2
Nitrate – N	15	3	12
Inorganic – P	1	1	nil

Source: Adapted and adjusted from DWAF (1999)

(ii) WWTP capacity increase versus other options

If increasing the capacity of a WWTP instead of controlling the rate of inflow/ infiltration within the collection system is the aim, there are available wastewater treatment technologies to increase treatment capacity. One known technology is a submerged fixed-film biological process which maintains the bacterial growth and at the same time handles increased flow rates during I/I events. However, the application of a method such as this will require significant capital and operation investments which should be balanced against the costs of proactive maintenance of the collection sewer system before any changes are committed.

Before introducing additional stormwater detention storage at the WWTP, it is important to investigate what operational changes to the whole wastewater collection/treatment system can be done at the lowest costs. The spheres of investigation are as follows:

- *modification/upgrading of the wastewater collection system*
- *modification to the WWTP considering, e.g. rehabilitation of settling tanks (i.e. chemical coagulation, microsand, etc.) and dissolved air flotation, and*
- *construction or refurbishment of additional storage facilities in the wastewater collection system (e.g. satellite treatment plants, storm flood detention storage, reuse of abandoned treatment works, etc.)*

CHAPTER 4. LOCAL AND INTERNATIONAL EXPERIENCE ON EXTRANEOUS FLOWS

4.1 Literature, reports and Internet fact-finding review

4.1.1 Local experience

There appears to be a lack of adequate awareness about problems and solutions to extraneous flows among the Water Services Institutions in South Africa. The issues of combined sewer overflows (CSOs) are fairly well documented in the available literature and on the Internet, however not really applicable to local conditions.). About 70 percent of existing urban waterborne sewer system in Europe are combined systems. This is in contrast to South Africa where some 90 percent of existing municipal waterborne sewer systems are separate or hybrid systems.

With the exception of Stephenson (1988), Pollet (1994) and Broome (1998), very little data and consistent research work are available on inflow/infiltration or generally on the problems of extraneous flows in waterborne sanitation systems in South Africa. Although the consulting engineers operating in the field of municipal sanitation do share their experiences by means of technical papers through regional conferences, there is an obvious shortage of relevant data and guidelines on issues of inflow/infiltration in urban sanitation based on local experience.

4.1.2 International experiences of extraneous flows in sewers

In most European countries, critical issues concerning public health and the environment are controlled by legislation, namely from European Directives. The key directive that now sets the direction for wastewater treatment in several European countries is the Urban Waste Water Treatment Directive (UWWTD).

In the USA, most of the issues and needs of the wastewater industry are attended to by the US Environmental Protection Agency (USEPA) and the American Society of Civil Engineers (ASCE). There are several other interest groups enabling the process of evaluating wastewater infrastructure needs.

It is now recognised by the engineering fraternity of the USA that the municipal infrastructure is deteriorating rapidly and that the need to rehabilitate is becoming an urgent national priority.

International references with a strong focus on inflow/infiltration issues in municipal waterborne sanitation systems have been published by Qasim (1986), Metcalf and Eddy (1991), CIRIA (1998) and Tafurio and Selvakumar (2000).

Inflow/infiltration problems and estimation methods applicable in urban sanitation systems are extensively covered by Metcalf and Eddy (3 editions available, 1981, 1986 and 1991) and Qasim (1986). Groundwater infiltration ranging between 0,01 m^3 and 1,0 m^3 per day per mm diameter per km length of sewer as determined by Metcalf and Eddy (1991) is used world-wide as the criterion for assessing infiltration in urban waterborne sewers. Qasim (1986) proposed a benchmark value for infiltration as 0,15 m^3 per day per mm diameter per km. Generally, rates of infiltration over 0.10 ℓ/min/m/m are considered excessive.

CIRIA (1998) adopted the infiltration rates for existing sewers of Stanley (1975) ranging from 15 percent to 50 percent of ADWF (UK context with 20 to 105 litres per capita per day).

Butler and Davies (2000) also refer to Stanley (1975) and Metcalf and Eddy (1991) on issues of inflow and infiltration.

4.1.3 Factors to consider when assessing inflow/infiltration

The search of available references indicates that the amount of inflow/infiltration taking place in a waterborne sewer system will depend on the length and age of the sewer collection line and the state of its appurtenant works. As the extent of inflow/infiltration events is highly site-specific, other factors need to be considered when evaluating extraneous flows:

- *Topographic conditions resulting in inadequate design in some instances*
- *Standard of materials and methods of construction (pipe joints, number of joints, etc.)*
- *Standard of workmanship in procurement*
- *Settlement due to ground movement*
- *Height of groundwater level (considering seasonal variations)*
- *Type of soil and ground cover*
- *Aggressive chemicals in the ground*
- *Situation with illegal roof and drain connections*

ASCE (1994) estimated that many of the wastewater collection systems perform only at about 50 percent of intended capacity. The current US nation-wide average of 0,3 blockages (or collapses) per km is estimated to be growing at a rate of some 3 percent annually. Factors which have a serious impact in causing blockages are root intrusion, material corrosion, soil movement and inadequate construction. Other factors contributing to sewer pipeline failures are deterioration of jointing materials, pressure surges, disturbances caused by construction works or direct tapping of the sewers as well as seismic activity in some areas.

Table 4.1. Summary of parameters from literature search on sewer flow components

Components of sewer flow			**Source**
Population-generated flow	**Leakage direct to sewers**	**Infiltration**	
60 to 85% of per capita water use to sewers	Included in domestic flow	15 to 50 percent of average DWF	Stanley (1972)
60 to 85% of per capita water use to sewers	Included in domestic flow	0,10 ℓ/min/m dia/m pipe	Qasim (1986)
60 to 85% of per capita water use to sewers	Included in domestic flow	0,01 to 0,7 ℓ/min/m dia/m pipe	Metcalf and Eddy (1991)
0,60 ℓ/min/urban erf (UE)	0,15 ℓ/min/urban erf (UE)	0,10 ℓ/min/m dia/m pipe	GLS Inc (1997)
0,42 ℓ/min/for every 100 m^2 of erf size	Included in domestic flow	15 per cent allowance for extraneous flows	New Red Book CSIR (2003)

4.1.4 Trends in international decision-making on wastewater services project development

The key objective of a water services project development is to compare various options to find the most suitable solution to the problem. This is particularly important in the decision process involving comparison of alternative solutions to rehabilitation, upgrading or developing a new system. Internationally, the common approach when developing a wastewater services project will include the following process stages:

- *Identify the problem and associated issues*
- *Determine who are the stakeholders in the solution to the problem*

- *Propose a course of action*
- *Develop an adequate and reliable database*
- *Evaluate funding source and commitments, environmental implications, public opinion, rehabilitation or new system (or component)*
- *Decide what methods be used to determine solution to the problem (e.g. experience for decision-making and/or mathematical modeling)*
- *Seek an optimum solution to ensure the most economic solution is obtained*
- *Implement the decision and arrange for observation of the project to record post-development behaviour of the adjusted system*

It is important to note that the solution to the problem relies wholly on the experience and knowledge of the decision makers. If an entirely new problem is encountered or a whole host of constraints need to be dealt with, the decision makers should be assisted by a mathematical modeling process. There is however a choice to be made in the method of assistance to problem solving between modeling the process mathematically or using a utilitarian approach. This approach to decision making attempts to assign a value to certain situations and then uses a mathematical technique to select a preferable option in respect of the assigned value.

Table 4.2. Key factors (constraints) in water services decision-making

Factor or constraint	Description of decision making	Degree of importance
Legal requirements	The operation of water and wastewater institutions is controlled for society by legislation. The decision process will be influenced by a need to comply with sets of standards, codes of practice and legislative directives	Important
Public opinion	It should be established how does the public perceive the need for the project and how will it react to the final product, particularly if there are financial or environmental implications for them	Important
Socio-economic trends	How will patterns in use of water or wastewater generate change? How will finance be raised to develop the project? How can the expenditure be justified? Cash flow issues must be considered	Most important
Engineering and environmental protection methods	Engineering, environmental protection, public safety issues and methods based on the projected life of the project concerning its development and post-development.	Most important
Future changes in technology application	What are the long-term requirements taking into consideration that water services schemes should last 30–50 years. Material and technique advances should be assessed.	Optional

Source: Adapted and adjusted from Rendell (1999)

An observation made from analyzing several technical reports during the course of this research project indicates that there is a lack of understanding on the part of engineers in the wastewater industry about the purpose and application of a benefit-cost analysis for the decision-making process. The most common shortcoming is that the benefits derived from upgrading or rehabilitating a system or a new project are not properly determined and evaluated. This leads to skewed results in comparing various feasible alternatives.

4.1.5 Common problems of municipal wastewater collection systems

From evaluation of available literature, sewer pipeline stoppages and collapses take place due to a combination of roots, corrosion, soil movement and inadequate construction. Generally these would be the most common cause of most of the structural failures. Besides stoppages and collapses, infiltration and inflow (I/I) can rob capacity from a waterborne sewer system and negatively affect operation of the entire sewerage collection system. In the late 1980's, the term "rainfall-induced infiltration" was first used to describe infiltration with flow characteristics resembling inflow (i.e. a rapid increase in flow which coincides with a rain event). Rainfall-induced infiltration takes place when stormwater runoff causes a rapid groundwater recharge around sewers, including manholes and building connections, which then enters the system through defective pipes, pipe joints, or manhole walls.

In developed urban areas, old sewer systems constructed before 1970 applied mortar or mastic jointing materials, which have deteriorated contributing substantially to I/I and exfiltration.

Due to low awareness about I/I/E problems in municipal wastewater sanitation systems, generally the O&M of these systems can be classified as inadequate. Most WSA/WSPs in South Africa assume that there are no immediate problems with I/I/E, consequently the maintenance budgets are commonly low and are based on the previous year's expenditure on clogging and collapses. In the meantime the deterioration of a system's components continues to the point of failure and beyond at several locations around South Africa.

Generally, the maintenance strategy of many WSAs in South Africa is essentially reactive maintenance, where problems are dealt with on a corrective basis as they arise. Although this is a standard approach, planned or proactive maintenance involves identifying elements that require maintenance in advance according to the frequency or risk of failure.

4.1.6 Problems identified primarily in local municipal waterborne sanitation

(i) Consequences of improper design

- *Drainage of low lying areas.* Individual urban properties are commonly affected when overflows from sewer systems start spilling out at gullies in lower lying areas due to improper design. It causes pollution due to overflowing pumpstations. It affects the running costs of sewer pumpstations, wastewater works and also impacts on the quality of the wastewater works' effluent, and ultimately the natural river system where the polluted water and the untreated water end up.
- *Topographical configuration*. Typically, the slope of a catchment or drainage basin at the central reach of a catchment can be fairly flat with slopes of less than 3% and varies between 3% and 6%. Regarding the position of a river in a catchment, its drainage channel does not have very steep valleys and riverbanks. The topography therefore lends itself to easy flooding as a result of small changes in the water flow sheet or slight positive changes in the river stage.

(ii) Consequences of improper construction

- *Improper installation*. Defects in a sewer may have been generated during installation (e.g. deflections, punctures, cracks, rolled joints, etc.) or over the life span (e.g. corrosion, erosion, root penetration, etc.). Installation of sewer systems may at times be of an unacceptably low standard and can be shadowed from building inspectors. Also, alterations can be made later after the system construction plans have been approved.
- *Gullies and terraces*. At most stands (erven), proper excavations and terracing were never considered in detail, leaving yard drainage compromised. Gullies were constructed facing the stormwater flow direction and mostly at the same level as the surrounding ground. This connects the stormwater directly to the sewer system. Stormwater should be directed into the gardens where space is available, or to roads

where it will be collected by the stormwater system. The serviced area can become drenched and the water flow sheet becomes pronounced in heavier downpours which may result in stormwater flowing directly into gullies.

(iii) Problems arising from civil disobedience

- *Theft of manhole covers*. In the past, the lids of manhole covers were made of cast iron, but became vulnerable to theft and to trading as scrap metal. These manhole covers are being systematically phased out and replaced by lockable or concrete manhole lids. Where the covers have been stolen and have not been replaced or broken during routine maintenance or deteriorated (rundown), the stormwater gains free or unhindered entrance into the sewer system.
- *Linkage of stormwater to sewer*. Gutters form part of a house roof stormwater drainage system. However, due to practices adopted, it will be noticed that in many homesteads the down pipes from the roof are linked to the sewer gullies. Yard paving has become a convenient way of keeping homes looking splendid all year round. However, this practice increases the quantity of water to be drained and therefore the height of the flowing water sheet. To get rid of this water, some individuals have constructed private manholes which drain into the sewers or directly into the sewer system. Water is also led into the sewer system via rodding eyes/inspection eyes.
- *Swimming pool overflows*. Swimming polls may also be a contributing factor as overflows due to rainfall and backwash water are linked directly or indirectly to the private sewer drainage system.
- *Excessive littering*. Increasing pollution is the most obvious result of catchment management. Because of neglected street sweeping, inefficient refuse disposal and removal, monitoring and treatment of urban runoff are considered a luxurious investment.

(iv) Other factors causing less common problems

- *Inadequate inspecting*. It can be alleged that most problems stem from cumulative innocent discharges of excess water into the sewer networks potentially due to lack of knowledge or improper advice, illegal discharging of processes effluents into the sewer network, inadequate design of certain system components and also by changing drainage capacities of the natural systems.
- *Inadequate budgeting*. Inadequate inspections and attention to existing infrastructure will cause inadequate budgeting for operation, maintenance and further development.
- *Lack of maintenance planning*. Pro-active maintenance represents maintenance work carried out in a planned manner at key points in a sewerage system to ensure that the hydraulic capacity is not reduced by blockages or by the build-up of sediment deposits or excessive sliming in the pipes. Monitoring of the results of the work must be carried out to determine its effectiveness and, if necessary, to adjust the frequency of cleaning.

Pro-active maintenance can sometimes be a very cost-effective way of dealing with sewer flooding problems. However, while such a solution may save a considerable amount of capital expenditure, it will usually cause some increase in operational costs. It is important that the potential benefits of pro-active maintenance are not lost because of the way in which financial controls on the capital and operational budgets of the sewerage undertakers are operated. If there are sound economic reasons for choosing this approach, sewerage services providers should explain them to consumers and supply relevant information justifying appropriate increases in their operational budgets.

4.1.7 Specific physical factors which may cause problems

(i) Pipe component age factor

- *Blockages/stoppages and collapses*. Stoppages and collapses, I/I events and exfiltration which take place in municipal sewer pipes can increase on account of the high levels of pathogenic micro-organisms, suspended solids, toxic pollutants, floating objects, nutrients, oil and grease, and other pollutants over the life-span of a pipe. Blockages due to intrusion of tree roots are a significant problem in many municipal waterborne sanitation systems.
- *Obsolete or inadequate pipe material*. Over time, specific pipe materials have proved inadequate (e.g. SANTAR) and are no longer used or recommended for installation. Existing sewer pipelines built from these materials need to be replaced.

(ii) Long-term performance factors

- *Changes in loading or stress conditions*. A specific problem encountered in the mining areas of South Africa (particularly in the Gauteng province) is that some gold mine dumps are being reprocessed through improved extraction technologies, resulting in the removal or even relocation of the overburden on the natural ground. During the period of the dumps' existence, the natural ground experienced loading strain, stress and consolidation. Once the overburden is removed, the soil undergoes stress relief, with the potential for recourse on the loading history over time causing misalignment and possible damage to buried sewer pipes.
- *Changes in soil retention conditions.* The mining industry also practices reed bed reclamation and these soils may develop spongy characteristics, retain more water and become swampy, increasing the potential for infiltration to the sewer pipeline.

4.1.8 Non-effective utilisation of existing storage in sewer systems

(i) Reduction of peak flows

Options for making better use of existing storage in sewerage systems tend to be specific to the particular circumstances but can either be passive or active. An example of a passive type is the addition of flow control devices in the upstream part of a system to make use of unused storage in manholes and thereby reduce peak flows farther downstream.

(ii) Use of auxiliary facilities

Active types of solution interaction between flow conditions and the operation of equipment such as pumps, gates and off-line storage tanks. The interaction may be achieved through the application by operations staff of written rules based on past experience, perhaps supported by analysis of the behaviour of the system using a hydraulic model. Alternatively, the operating rules may be implemented automatically by means of electronic links between the flow control equipment and sensors located at key points in the system. In the next generation of solutions, full real-time control may become an option, with a computer model forecasting flow conditions in the sewerage system and evaluating alternative strategies for operation of the control equipment; this type of option will normally tend to be applicable only on a catchment-wide basis.

(iii) Malfunction of diversions and bifurcations

External overflows, bifurcations or diversions can be located at points of hydraulic overloading to remove excess flows from sewerage systems (more common with combined sewer overflows), and allow for discharge of excess flow to open land or a watercourse. Diversions, together with bifurcations, are used to divert excess flows either into another part of the same system with spare capacity or into another adjacent system.

(iv) Inadequate overflow facilities

Adequate storm peak overflow facilities can be provided before the WWTP within a sewer system. The overflows should flow to two containment dams and be recycled back to the inlet of the works when the storm flows have subsided. The dams must be designed so that they can be periodically drained, dried out and cleaned (desludged) during the dry season. For this purpose, they must be fitted with vehicle ramps for access by front-end loaders and tipper trucks. The outlets of the dams should be fitted with scum baffles.

4.1.9 Low attention to overall enhancement of sewer system

(i) Pipeline network

- *Hydraulic enhancement*. Together with the provision of storage, this is the most commonly used method of solving flooding problems. Existing systems are replaced or enhanced to remove the hydraulic restriction(s) that cause the sewer flooding problems. In some cases this may be achieved by replacing an existing length of sewer by one of high flow capacity (i.e. having a larger diameter or smaller hydraulic resistance). Alternatively, a length of by-pass sewer may be constructed to carry some of the flow from the existing sewer over the section where it has insufficient capacity. There is a significant risk that works to improve conditions at one location may transfer the problem farther downstream unless equivalent improvements are made all the way through the system. This possibility should be carefully investigated before new construction works are undertaken.
- *Flow control devices*. Together with construction works to increase sewer capacity, this is the most commonly used method of solving flooding problems. Flows upstream of a critical part of the system are restricted to the capacity of the pipes by controlling and storing excess flows until the system can cope. Peak flows may be attenuated by providing purpose-built storage, usually in the form of on-line or off-line detention tanks, or by temporarily holding back surface water run-off (e.g. in detention ponds or in open areas such as car parks). Flow control devices are used to control the onward flow to the downstream part of the system and/or to divert flows into storage.

(ii) Problems associated with pump size and pumping

- *Small pumping system configuration*. Pumping systems for dealing with sewer-flooding problems can be considered in three categories of size and complexity:
  - *(a) Installed packaged systems, consisting of pumps and storage chambers that cannot take gravity flow from a group of properties into a surcharged public sewer (either directly or via a gravity pipe).*
  - *(b) Intermediate-size pumps installed in inspection chambers that cannot discharge flow from a single property or basement into a surcharged sewer (either directly or via a gravity pipe).*
  - *(c) Small macerating pumps with small-bore pipework that cannot discharge flow under pressure from individual units such as baths, sinks, WCs and showers.*

Higher-rated units are able to pump from below ground level into surcharged sewers (either directly or via a gravity pipe).

Pumps used for the first two categories are normally either small submersibles with macerators or grinder pumps.

- *Large scale wastewater pumping*. Pumpstation equipment should comply with the following criteria:
  (a) *The installation should comprise at least one standby pump;*
  (b) *Pumping capacity is to equal or exceed the peak wet weather flow (PWWF) which might arrive at the pump station; and*
  (c) *The sump of the pumpstation should be sized to ensure that the pump does not switch on and off more than six times per hour.*

Pumping stations installed in a wastewater subsystem are usually equipped with electrically driven automatic pumps operated from level measuring devices or switches in the reception sump, enabling them to operate without full-time pump attendance. Sewage pumps are normally centrifugal or mixed flow devices. Submersible sewage pumps are also frequently installed.

Where a very wide range of flows has to be covered or when pumping the spill from an overflow, consideration may have to be given to using pumps of two sizes. As the flow of wastewater varies continuously, balancing capacity has to be provided for a constant speed pump by a suction well. If more than one pump is installed, starting levels are arranged so that pumps are started successfully as the flow increases. Hydraulic surges occur in a sewer pipeline. However, they may be materially lowered by dissolved gasses contained in the sewage fluids

4.1.10 Other possible problems

(i) Installed vacuum systems

These systems transport sewage by inducing and maintaining a vacuum in the collecting pipes by means of central vacuum pumps and a reservoir. Conventional gravity drains connect one or more properties to a sewage collection chamber. When the sewage reaches a preset level, a pneumatic “interface” valve opens and the contents of the chamber are sucked into the vacuum line. When the chamber is almost empty, the valve closes.

Vacuum systems should normally deal only with domestic wastewater because satisfactory performance depends on their being sized accurately in relation to the maximum design rate of flow. These systems should not be expected to cater for overland flow, infiltration or roof run-off, and additional flows should not be added without considering the limitations of the original design.

(ii) Existing protective structures

The existence of a wall or bund constructed around a single property, or a group of properties, offers protection from sewer flooding, but may have not been used and should be reconsidered. Bunds placed around manholes in sewerage systems as a temporary measure to minimise the extent of flooding at low points in gardens and open spaces should be removed or regularly maintained.

(iii) Operation of purchased land (or properties)

This involves the sewer services provider who purchases a property either to remove it from a list of occupied properties affected by flooding or to change its usage so as to mitigate the effects of flooding. In some cases where flooding is confined to an occupied basement, only a change of usage for that part of the building might need to be negotiated.

(iv) Further changes in water legislation and environmental laws

The ongoing process of implementation of the environmental protection law and promulgated water laws has increased the urgency to observe the new criteria and constrained evaluation procedures in municipal wastewater collection and treatment services.

All Water Services Authorities/Providers administering municipal waterborne sanitation are urged to review their municipal by-laws to adjust and comply with recent legislative changes.

4.2 Nation-wide survey of stormwater inflow/groundwater infiltration awareness

4.2.1 Background and purpose of survey

In order to determine general awareness about inflow/infiltration (I/I) problems in municipal sanitation systems in South Africa, a nation-wide e-mail survey was conducted under this research project. The survey generated new valuable information and verified several parameters for development/ enhancement of urban separated sewer systems.

The survey proved very strongly that there is a very low level of awareness among WSAs/WSPs about I/I, the essential causes, methods of monitoring and determining magnitude of I/I events, as well as the ways of preventing these events.

As a direct spin-off from the survey, it was established that new and comprehensive guidelines on how to monitor, evaluate, mitigate or completely prevent extraneous flows into municipal sewers are urgently needed. The contemporary standards/procedures have to be re-evaluated and tested against the new findings and guidelines.

Most municipal sewer systems in South Africa have been in existence for 30 to 50 years and the aging process is taking its toll so that issues related to rehabilitation or replacement are becoming more important to the WSAa/WSPs. The type of materials used in the construction of sewer systems have also changed from formerly clay and concrete to uPVC and AC piping, generating different sets of problems.

The surveyed sample indicated that the per capita daily water use ranges between 100 to about 670 litres, with an average of about 350 ℓ/c/day. The range is rather wide owing to the levels of water services installed at the surveyed municipalities. Assuming that only about 60 percent of water input finds its way back to the natural surface sources for indirect reuse in an acceptable qualitative form, there is an urgent need to concentrate on sanitation services at all municipalities in South Africa.

4.2.2 Approach to data gathering

The requirements for developing and compilation of this research study include an extensive literature survey as well as field surveys. In the recent past, field data has been typically obtained by means of surveys through direct interviews, postal, telephonic and fax techniques. However, this time it was decided that owing to extensive availability of electronic facilities (i.e. e-mail and Internet) at most of WSAs/WSPs, data acquisition would be based exclusively on the e-mail facility. A two-fold questionnaire (see Appendix A and B) was compiled for distribution to various municipalities.

Table 4.3. Ranking of surveyed municipalities and size of annual sewer maintenance budget, fiscal year 2001/02

Rank by size of supply	Water Services Authority/ Provider (Old municipal name)	Total potable water supply ($10^6 m^3$ p.a.)	Population in 2001 ('1000)	Budget on sewer maintenance (R 1000)
1.	Joburg Water (Johannesburg Metro)	432,672	3225,8	71 000
2.	Newcastle	30,350	315,0	16 500
3.	City of uMhlathuze (Richards Bay)	28,315	115,0	31 361
4.	Emnambithi (Ladysmith)	15,756	107.9	?
5.	Drakenstein (Paarl)	12,255	* 88,5	6 967
6.	Middelburg	9,107	140,0	5 403
7.	George	8,030	113,2	7 700
8.	Mossel Bay	7,800	63,2	99
9.	Randfontein	6,786	175,0	8 100
10.	Westonaria	4,330	* 97,8	?
11.	Sesotho (Ficksburg)	2,676	73,0	1 200
12.	Knysna	2,620	39,0	150
13.	Mpofana (Mooi River)	1,205	17,0	1 100
14.	Utrecht	0.683	15,5	345
15.	Sesotho (Clocolan)	0,401	7,3	389
Subtotal for useful sample		**562,986**	**4593,2**	**150 314**
16.	Southern District (Klerksdorp)	12,556	223,6	?
17.	Makhado (Louis Trichardt)	2,482	52,8	?
18.	Mbombela (Nelspruit)	* 13,100	235,4	?
Total for whole sample		**591,124**	**5105,0**	**?**
*Estimate				

The survey resulted with 21 responses received (i.e. about 30 percent of the surveyed municipalities). However, only 15 returns were classified useful to the survey criteria and analysed (i.e. some 21 percent success rate).

Table 4.4. Ratio of volume of wastewater treated to the volume of water supplied (2002)

Rank by size of supply	Water Services Authority/ Provider (Old municipal name)	Metered flows (* 10^6 m^3 p.a.)		
		Total supply (IN)	Return flows (OUT)	Ratio (r) (OUT/IN)
1.	Johannesburg Water (Pty) Ltd (Johannesburg Metro)	432,672	287,664	0,66
2.	Newcastle	30,350	4,868	0,16
3.	City of uMhlathuze (Richards Bay)	28,315	13,339	0,47
4.	Emnambithi (Ladysmith)	15,756	* 11,029	* 0,70
5.	Drakenstein (Paarl)	12,255	7,943	0,65
6.	Middelburg	9,107	4,636	0,51
7.	George	8,030	4,986	0,62
8.	Mossel Bay	7,800	3,600	0,46
9.	Randfontein	6,786	* 3,054	* 0,45
10.	Westonaria	4,330	3,623	0,84
11.	Sesotho (Ficksburg)	2,676	0,924	0,35
12.	Knysna	2,620	1,1477	0,56
13.	Mpofana (Mooi River)	1,205	* 0,542	* 0,45
14.	Utrecht	0,683	0,292	0,43
15.	Sesotho (Clocolan)	0,401	* 0,160	* 0,40
Totals per useful sample		**562,986**	**348,137**	**0,62**
* Estimates are based on municipal yearbook data prior to 2002				

A national average ratio, r, or wastewater discharged to water supplied derived from the municipal survey sample is of the order of r = 0,62. This value corresponds well with the average figure usually quoted in South Africa. According to Barta (2003), the overall ratio, r, determined for the whole Gauteng Metro Complex amounted to r = 0,65 in 2000.

4.2.3 Evaluation of WSA/WSP awareness about extraneous flows

Both of the survey questionnaires were designed to assess in the first instance the extent of awareness of the Water Services Authorities/Providers about the problem of extraneous flows. The questions asked were related to the following:

- *What is the status of your sewer network(s)?*
- *Are there records on infiltration/inflow of groundwater/stormwater into the sewer systems?*
- *What is the extent and volumes of infiltration and inflow (i.e. surges in sewers) a problem measured in ℓ/min / m pipe / mm dia?*
- *What are the ways and means used in monitoring extraneous flows?*
- *What arrangements are in place to prevent stormwater inflows to sewers?*

4.2.4 Survey generated factors associated with extraneous flows

(i) Common problem associated with I/I events

The sample survey indicated that the most common causes of stormwater inflows and groundwater infiltration in the South African context are as follows:

- *Inadequate design of certain system components,*
- *Illegal house down-pipe connections to the municipal sewers (all surveyed municipalities operate separate instead of combined sewers),*
- *Open gullies serve primarily as sullage disposal (this typically in most formal and informal townships),*
- *Gutters/down-pipes linked illegally to sewer gullies,*
- *Unsealed manholes primarily due to theft of the manhole covers,*
- *Faulty pipe joints due to improper construction,*
- *Unwise man-made stormwater channelisation (e.g. road crossings and culverts) and unattended overgrown vegetation in natural channels, and*
- *High groundwater table.*

(ii) Specific problem associated with inflow/infiltration to municipal sewers

Next to the common causes generated by this survey, other factors were identified which can contribute locally to inflow/infiltration to sewers:

- *The dramatic topography may lead to easy flooding due to marginal changes in stormwater flows,*
- *Re-considered flood lines,*
- *Swimming pools can be a contributing factor if additional stormwater or backwash water is linked directly to the sewers,*
- *Ground movement due to removed mine dumps destroying continuity of sewers, and*
- *Thunderstorms of short duration and higher intensities in various locations.*

(iii) Current preferred methods of sewer maintenance

Tables 4.5 and 4.6 contain details of the preferred methods (as determined from the survey) in sewer maintenance/rehabilitation, municipal sewer systems (e.g. sewer lengths, pumping station) and pipe materials most commonly used in developing urban sewer systems in South Africa. The preferred methods of sewer maintenance/rehabilitation applied by the municipalities are:

- *Rodding for every day maintenance*
- *Jetting of blockages*
- *Replacement or relining*
- *In-situ relining*
- *Sliplining*
- *Other trenchless technologies*

(iv) Typical sewer blockage rate

From the limited but representative sample of the nation-wide survey, it is concluded that the typical average sewer blockage rate is 3,3 blockages/km sewer pipe/p.a. This figure is more than double the average commonly quoted in the limited literature in South Africa of 1,2 blockages/km pipe/year and far superceded international averages. This aspect will influence the calculation of costs if trenchless technology is adopted in sewer maintenance programmes. The most common materials used by the municipalities are identified as uPVC and AC piping, in pipe diameters ranging from 100 to 1000 mm. A surprising aspect of municipal sewer systems is the large number of pumping stations built in some of the existing systems.

Table 4.5. Common problems associated with sewer maintenance and rehabilitation

Rank by size of supply	Water Services Authority/ Provider (Old municipal name)	Blockage/ km pipe/ year	Preferred methods of sewer rehabilitation	Most common causes of inflow/infiltration
1.	Johannesburg Water (Johannesburg Metro)	3,3	Replacement or relining	Illegal house connection, missing manhole covers
2.	Newcastle	-	Replacement	-
3.	City of uMhlathuze (Richards Bay)	5,7	Jetting for block-ages, replacement	Open gullies, unsealed manholes due to theft
4.	Emnambithi (Ladysmith)	1,8	Replacement	Illegal connection
5.	Drakenstein (Paarl)	3,4	Sliplining	Faulty pipe joints and manholes
6.	Middelburg	N/av	Trenchless techn.	Illegal connections
7.	George	-	Jet cleaning and replacement	Illegal connections
8.	Mossel Bay	3,3	-	-
9.	Randfontein	4,7	In-situ lining	-
10.	Westonaria	-	-	-
11.	Sesotho (Ficksburg)	1,0	Jet cleaning and replacement	Illegal connections
12.	Knysna	3,0	Sliplining	High water table
13.	Mpofana (Mooi River)	-	-	Illegal connections
14.	Utrecht	0,2	Rodding	-
15.	Sesotho (Clocolan)	-	Rodding	-
? Information not provided				

(v) Impact of I/I events on Waste Water Treatment Plant (WWTP) processes

None of the surveyed municipalities indicated any particular problems associated with the consequences of I/I events on the treatment processes at municipal WWTP. However ERWAT indicates that the effect of I/I events on treatment processes can be rather dramatic as the final effluent produced will not comply with required effluent standards. Their experience shows that where the I/I events are a serious problem, seasonally the built-in capacity of WWTP can be exceeded by up to twice for a limited time. Taking into consideration that DWAF is about to introduce new effluent (waste) discharge standards, the conventional Biological Nutrient Removal (BNR) will have to be refurbished to enable old plants to comply with the new requirements particularly removal of suspended solids and nitrates.

Table 4.6. Survey details on municipal sewer systems (2002)

Rank by size of supply	**Water Services Authority/Provider (Old municipal name)**	**Sewer system details**			**Pipe materials used**						**Range of pipe dia used (mm)**
		Gravity mains (km)	**Pumping mains (km)**	**Pumping stations (No)**	**Clay**	**Concrete**	**HDPE**	**uPVC**	**AC**	**Other**	
1.	Johannesburg Water P/L (Joburg Metro)	9179	?	35	X	X	X	X	X	X	100 to 3000
2.	Newcastle	?	?	10		X		X	X	X	150 to 900
3.	City of uMhlathuze (Richards Bay)	610	32	69				X	X	X	160 to 450
4.	Emnambithi Ladysmith)	648	15	18				X	X	X	100 to 450
5.	Drakenstein (Paarl)	339	3,2	8			X		X	X	100 to 900
6.	Middleburg	547	4,1	4				X	X	X	150 to 900
7.	George	520	30	30				X	X		?
8.	Mossel Bay	420	38	43				X	X	X	100 to 500
9.	Randfontein	220	11,7	10	X		X	X	X		100 to 1000
10.	Westonaria	25	4,5	5	X			X	X		?
11.	Sesotho (Ficksburg)	90	-	-	X			X			110 to 400
12.	Knysna	50	10	43	X			X	X		110 to 500
13.	Mpofana (Mooi River)	10	8	7	X			X			110 to 250
14.	Utrecht	5	16	2							50
15.	Sesotho (Clocolan)	62	12	4				X			110 to 200
WSA/WSP responses without any useful information											
16.	Southern District (Klerksdorp)	?	?	?							?
17.	Makhado (Louis Trichardt)	?	?	?							?
18.	Mbombela (Nelspruit)	?	?	?							?

? Information not provided. NB. Contrary to survey results, large diameter concrete sewer pipes are still being widely used.

CHAPTER 5. FIELD INVESTIGATION OF EXTRANEOUS FLOWS PROBLEM

5.1 Field investigation in the Ekurhuleni MM

5.1.1 Sewer flow measurement concept and location

The consulting engineer firm, GLS (2003) specializing in investigating sanitary waterborne sewer flows, made available field measurements conducted in the Ekurhuleni Metropolitan Municipality. The approach in flow measurements is based on the concept of unit erven (UE) applied for the analyses and planning studies. The following definitions for four components and their indicative values of sewer flow are considered:

- *Domestic flow.* The basic input is a typical 24-hour unit hydrograph of sewer flow for each unit erf (UE). The unit hydrographs may differ significantly in terms of shape, volume and peak flow for the various land uses. Typical values determined for residential unit hydrographs in urban areas are 1,0 ℓ/min/UE for the peak flow and 800 ℓ/day/UE for the total daily flow.
- *Leakage.* This component (generated by leaking toilet cisterns, leaking taps, etc.) represents a portion of the base sewer flow. A typical leakage flow is determined at 0,15 ℓ/min/UE..
- *Infiltration.* Groundwater seeping through joints/cracks in the pipelines and manholes makes up the rest of the base sewer flow. Infiltration rates are expressed in ℓ/min/m pipe/m pipe diameter, and a typical value is 0,10 without specifying pipe material.
- *Stormwater inflows.* During rainstorms, surface run-off finds its way into the sewer system via illegal connection of gullies and inundated manholes. Such inflows should ideally not occur in a "closed" system, it is nevertheless a reality and typically between 0,5 and 4% of all rain falling within 25 m on either side of a sewer pipe may find its way into the system, causing significant temporary effect on the sewer flows.

The parameter values illustrated above were determined primarily from sewer flow measurements taken in the Greater Alberton and Tshwane MM municipal systems with the objective of establishing typical unit hydrographs for the different land use types and magnitudes of leakage, infiltration and stormwater inflows in order to calibrate the mathematical model computer software. A further purpose of the measurements was to establish a relationship between water usage and sewer flows.

5.1.2 Flow measurement methodology

Altogether six flow measurement locations were selected by GLS (2003) based on the following criteria to satisfy the modeling approach:

- *Measurement location be on a straight section of pipe (in a horizontal plane).*
- *Pipe must not change gradient at location.*
- *Areas draining to the different locations must be more or less homogeneous, and must represent varying land uses, as well as varying sizes of drainage area.*

The actual field measurements were conducted by the Town Engineer's department with ERWAT's consent. The methodology adopted was as follows:

- *Approximately two weeks of continuous recording was done at each location during November 2000.*
- *The Town Engineer's department recorded the flow depths (above invert) at each location.*

- *The Town Engineer's department surveyed the locations to verify the pipe diameter and slope.*
- *The flow depths were converted to flow rates using Manning's equation for steady, uniform flow, with a roughness coefficient n = 0,012, or in the case of flumes, flow depths at the flumes were converted to flow rates, using the appropriate flume formula.*
- *Rainfall information for the corresponding period was obtained from the Town Engineer's department and ERWAT at the four locations.*

5.1.3 Relationship between sewer flow and water use in a specific urban locality

The Greater Alberton Water Master Plan established the annual average daily demand (AADD) for water supply to the Greater Alberton area at ± 61 685 kℓ/d (including UAW). The results of the flow measurements were used to estimate the total daily sewer flow for the Alberton, Eden Park and Greenfields drainage areas at 34 820 kℓ/d. These results were extrapolated to the Thokoza/Bassonia Rock and Linmeyer drainage area, resulting in an estimated daily sewer flow of 5 710 kℓ/d. This brings to 40 530 kℓ/d the total daily sewer flow measured in the Greater Alberton drainage area.

As a fraction of water demand, this daily sewer flow can be expressed as 66% (return factor = 0,66) of AADD. This compares relatively well with the typical norm of 65%. The measurements were taken in a reasonably wet season during November 2000, which also coincided with a period of relatively high water demand. The measured daily flows were therefore regarded as the peak daily dry weather flows (PDWDF). The average daily dry weather flow (ADDWF) is expected to be somewhat less, perhaps in the region of 60% x AADD.

The urban collection sewer network investigated is illustrated in Figure 5.1 below:

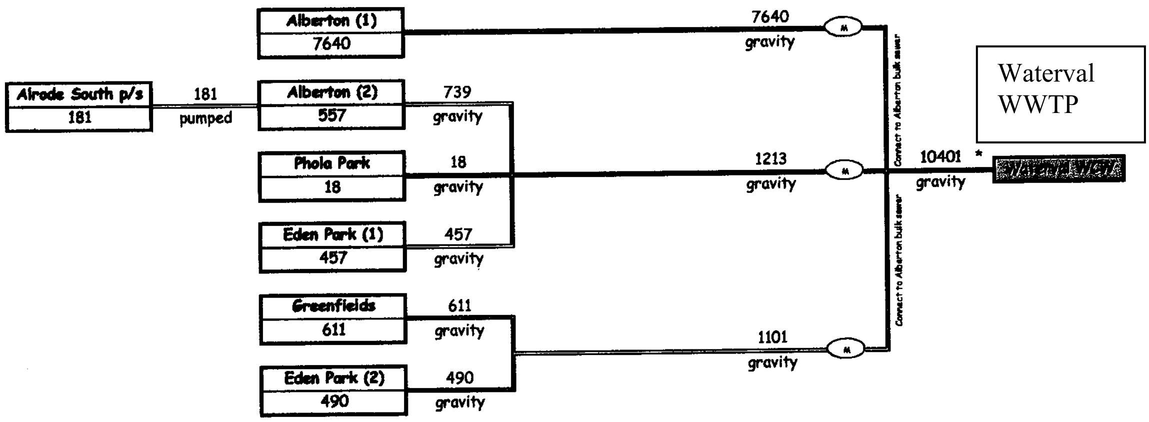


Figure 5.1. Layout of sewer network draining to Waterval WWTP (after GLS, 2003)

Wastewater flows vary similarly with the demand for water according to the season of the year, weather conditions, day of the week and time of day. Under dry weather conditions, the daily wastewater flow will show a diurnal pattern. The wastewater pattern parallels that of water demand with a lag of several hours. The fluctuations in wastewater flows are less than that of water supply due to the storage available in the sewers. The time required for the wastewater to reach the WWTP is also an important factor. The commercial and industrial discharges tend to reduce the peak flows. The infiltration/inflow into sewers can change the diurnal flow pattern. Diurnal waterborne sewer flow patterns determined for developed residential, commercial and industrial areas in the Ekurhuleni MM are illustrated in Figure 5.2.

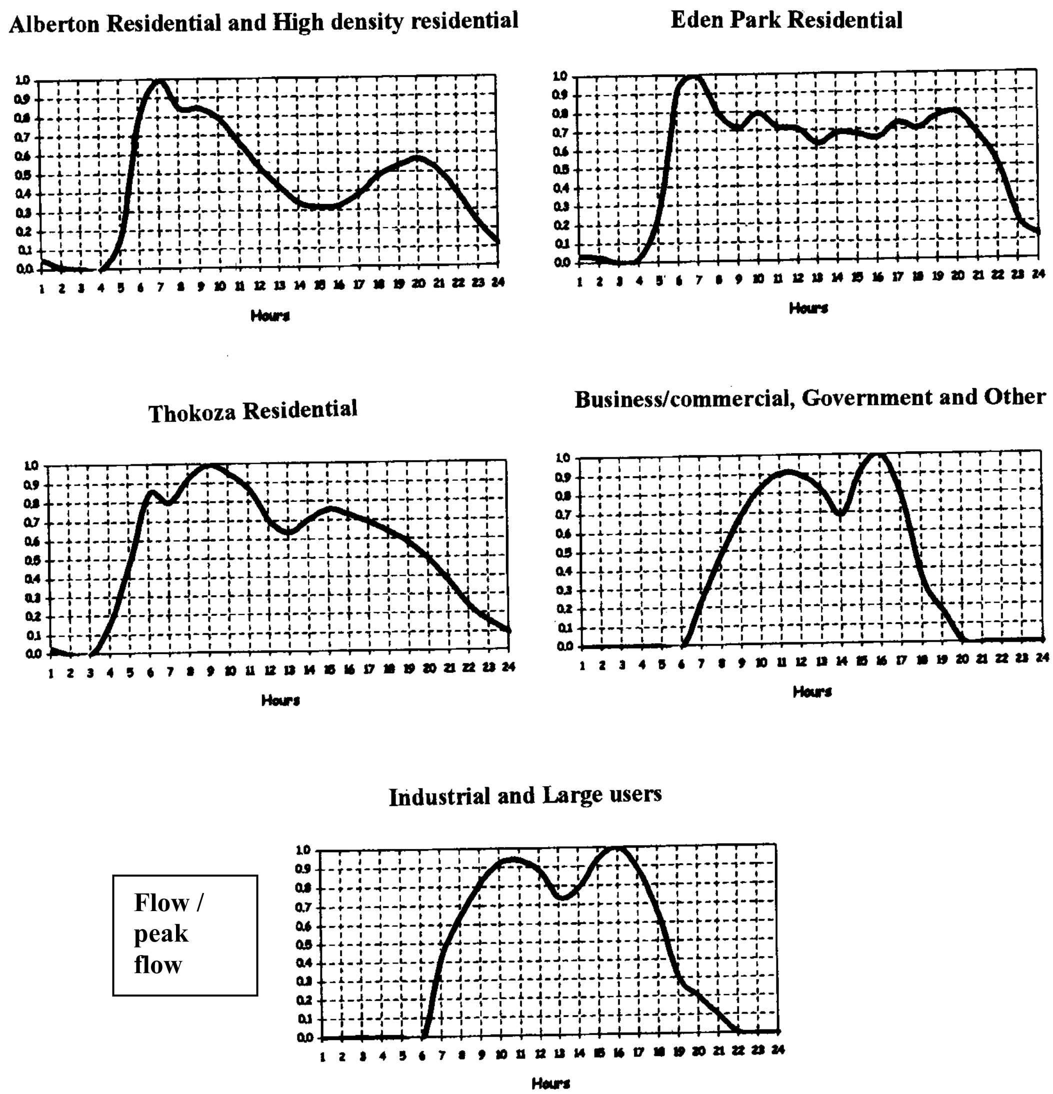


Figure 5.2. Diurnal waterborne sewer flow patterns for urban areas in Ekurhuleni MM (adapted from GLS, 2003)

The following three WWTPs in the Ekurhuleni Metro area were also studied as they had been identified as being subjected to significant inflow/infiltration events:

Table 5.1. Treatment works subjected to extreme infiltration

Treatment Works	Design capacity	Wastewater influent	Estimated extra load	Remarks
J.P Marais	14 Mℓ/d	27 Mℓ/d	13 Mℓ/d	Local WWTP
Vlakplaats	83 Mℓ/d	99 Mℓ/d	16 Mℓ/d	Local WWTP
Waterval	105 Mℓ/d	120 Mℓ/d	15 Mℓ/d	Regional WWTP

Source: Phalafala (2003)

As can be seen from Table 5.1, approximately 44 Mℓ/day infiltration takes place at these three works, which cannot always be treated properly and leads to the likelihood of pollution of the receiving water streams.

Phalafala (2003) pointed out that the high cost of chemicals needed to reduce chemicals and to disinfect the effluent, in order to meet effluent standards, increases treatment cost dramatic. Calculations have shown that the additional cost for chemical treatment as a result of infiltration is R350 000 per annum at these three plants only.

5.2 Field flow monitoring on a dedicated sewer line

The main objective of the field investigation has been set in obtaining first-hand information on the condition of the wastewater collection system comprising typically pipelines, pumpstations and appurtenant works (e.g. manholes, wet wells, siphons, etc.) and focused primarily on areas within the Ekurhuleni Metropolitan Municipality (EMM) which were identified by ERWAT and various consultants as highly prone to groundwater infiltration.

5.2.1 Site inspections conducted

Four site inspections were conducted in the early stages of the research project. The first was in the Benoni municipal sewer collection area to establish the physical state of the collection sewer system feeding the JP Marais Wastewater Plant and to prioritise groundwater infiltration as well as to become familiar with the land use aspects in the catchment area (e.g. large mine dumps).

Three site inspections were made in the Boksburg northern areas to assess the conditions for a possible pilot monitoring site and the Boksburg Stadium area was eventually selected. The prime purpose of the site inspections was to identify and select flow monitoring locations at suitable manholes. A visual check-up of missing manhole covers and possible sewer blockages was considered important and the researchers consulted the Boksburg Engineers Department who assisted in cleaning a blocked manhole upstream of the proposed monitoring site.

5.2.2 Locality of flow monitoring

Most of the field and monitoring work under this project was done within the Boksburg stadium catchment situated within the Ekurhuleni MM which administers a pipe network of about 8 261 km, 37 water pump stations from as many as 145 distribution zones. This metropolitan water services system comprises the urban waterborne subsystems of the former municipalities of Alberton, Benoni, Boksburg, Brakpan, Nigel, Springs, Germiston, Kempton Park, Edenvale and Midrand.

The necessary field data for this study has been collected with the assistance of GMKS (1999). Data obtained from the field measurements were correlated with the records of GMKS as they have done extensive investigations in this catchment prior to this study. Field measurements from other locations in the study area were also obtained and analyzed to become familiar with the problems and issues of I/I events elsewhere.

5.2.3 Description of investigated drainage catchment

(i) Land use in drainage catchment of Boksburg sewer outfall

A monitoring programme study for this project was conducted in the tributaries of the Boksburg outfall in order to determine which areas were subjected to the worst infiltration flow rates. Figure 5.3 shows the location and layout details of the studied sewer. The catchment has been divided into a number of sub-catchments to show different sections of the catchment contributing to different outfalls. The monitored sewer line is situated in subcatchment 212.

The areas around the Boksburg Lake and the Cinderella dam were identified as being a suitable study area for seasonal monitoring. Previous surveys in this subcatchment found the rate of infiltration to be significant due to the presence of small dams and wetlands located along the watercourses. The status of manhole covers and their elevations above ground level was evaluated prior to monitoring.

Urban development in this catchment is predominantly residential, with several surrounding industrial and business enterprises. The nature of future development is expected to be consistent with the present development assuming that no single user defined the shape of the flow hydrograph from the outfall.

(ii) Sewer pipe materials

The catchment under study investigation is fairly steep and therefore gravity flow is easily maintained. The sewer network consists of pipes that are circular in cross section, therefore, the principles of partially full pipes can apply to this network. Two different pipe materials were used throughout the entire Boksburg catchment: namely, vitro clay and concrete. If the entire Boksburg wastewater collection system is considered, the oldest pipe was laid in 1934. In the Boksburg Stadium catchment, pipes were laid in three different stages: the first set of pipes was laid in 1964, the second set in 1966 and the last set in 1981. These pipes were laid with a design life of 60 years, meaning that most of these pipes are about to pass their design life.

(iii) Soil conditions

Analysis of the soil indicates that the soil, in most parts of Boksburg, exhibits expansive and shrinkage properties. This implies that the wetting and drying cycle of the soil surrounding the pipes has a negative impact on the structural stability of the pipe. The stresses due to strains resulting from the expansion and shrinkage of the soil can initiate cracks in the pipe and cause joints to open and thus, leak. Leakage from sewers can cause groundwater contamination and may even initiate sinkholes in dolomitic sensitive areas. Open joints mean that during periods of high ground water levels in summer, infiltration of groundwater into the sewer pipes would occur.

The soil structure is composed of 10 – 40% rock build-up and highly expansive and shrinkage soils. Rock build-up implies that groundwater has good mobility and therefore can easily penetrate through the rocky soil structure into the buried pipe. Leakage (i.e. exfiltration) from the pipe also has free mobility into the underground water structure.

Some parts of the study area fall into dolomitic sensitive areas. Dolomites are characterised by their formation of sinkholes which can interfere with the structural integrity of the pipe or even result in complete pipe failure. Figure 6.2 shows the likely stages of pipe failure over time.

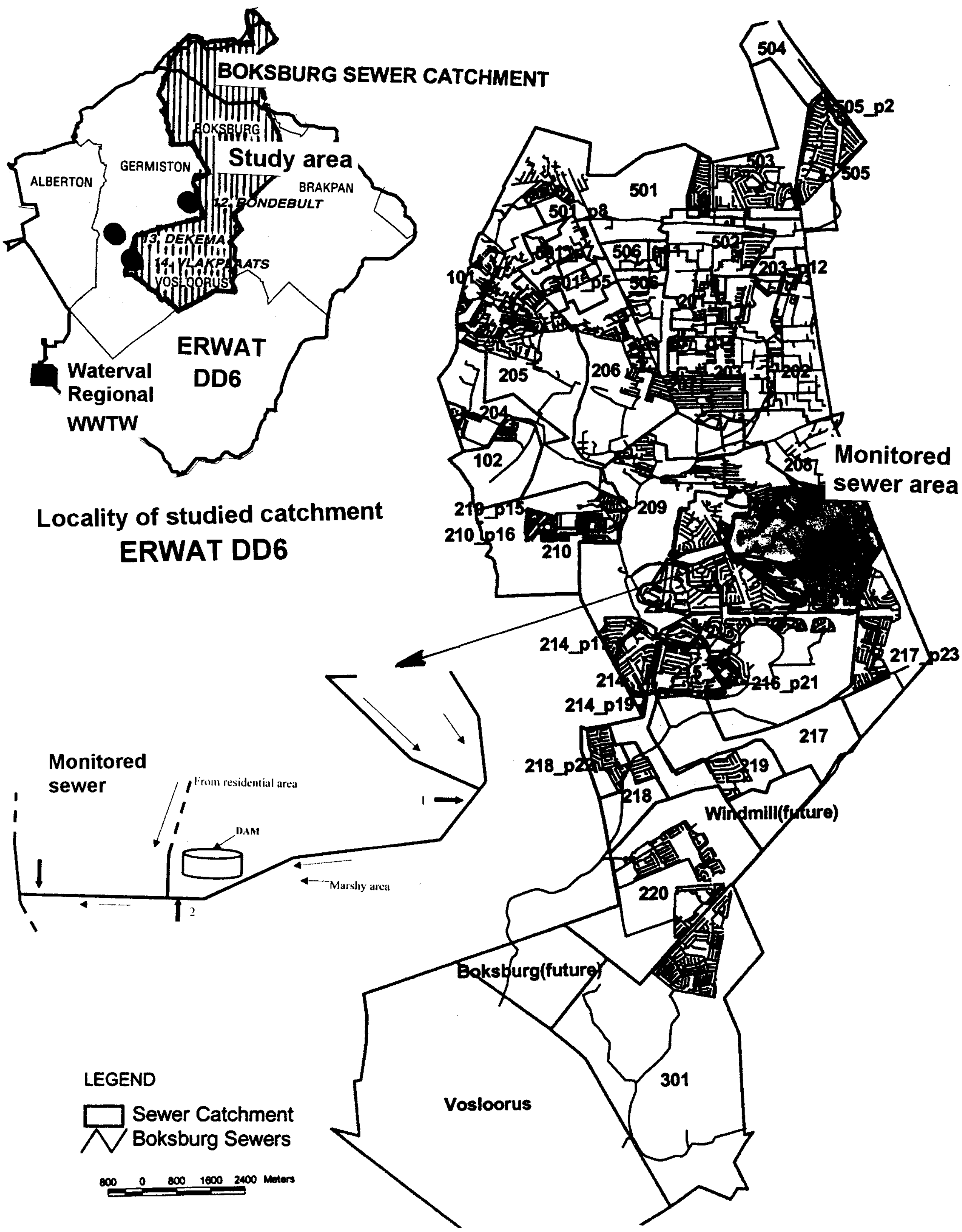


Figure 5.3. Locality of study area and studied sewers

(iv) Timing of field monitoring

The objective in selecting and installing flowmeters was to measure the winter base flows (i.e. July and August) and compare them to measurements of summer flows intended to be monitored in the next summer rainy season flows and the flow measurements taken previously by the GMKS consultants for the same area.

5.2.4 Extent and programme of monitoring of field flows

The researchers, together with GMKS Consultants who were extensively consulted in the past by Ekurhuleni MM on issues of sewer capacity expansion and maintenance, endorsed the choice of selected the dedicated sewer collector near Boksburg Stadium as the pilot flow monitoring site to start the project off. The timing of the installation of the monitoring devices was considered critical to allow for both dry and wet season flows to be equally monitored. Actual field monitoring was outsourced to a specialist subcontractor.

The study obtained two sets of records from three impulse reading devices installed. Installation of flow measuring devices and down-loading of readings took place every two weeks took place from 20 August 2002 at the following manholes:

Manhole No. 21261 (Main Road, Atlas Road line) – Station 1
Manhole No. 212054 (Boksburg Stadium Lake) – Station 2
Manhole No. 212027 (Langehoven Street) – Station 3

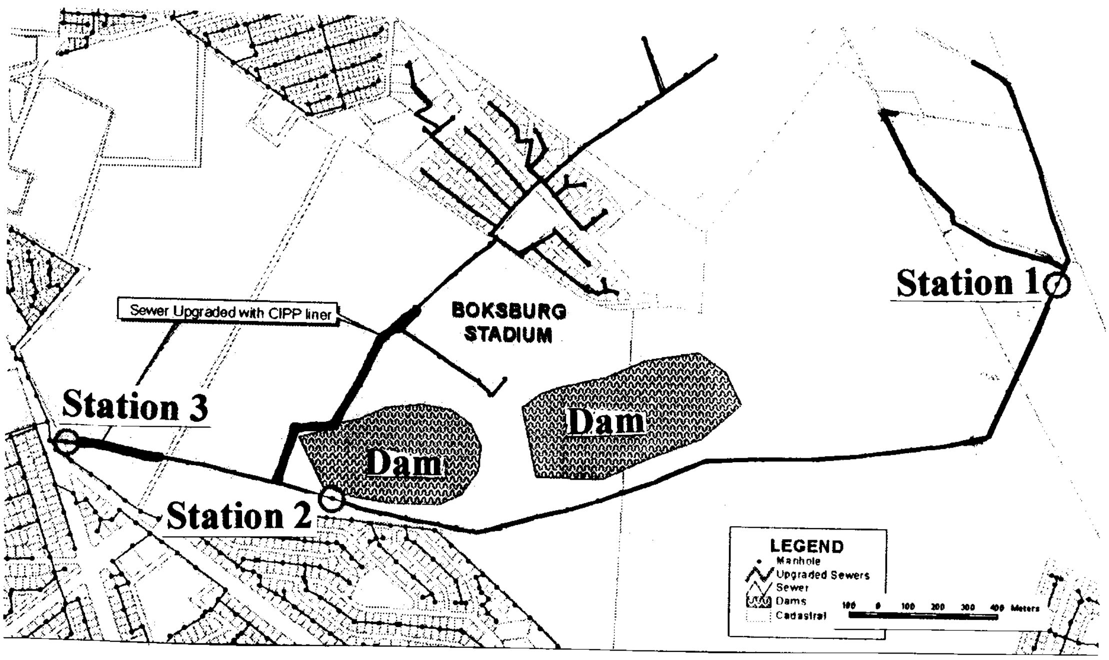


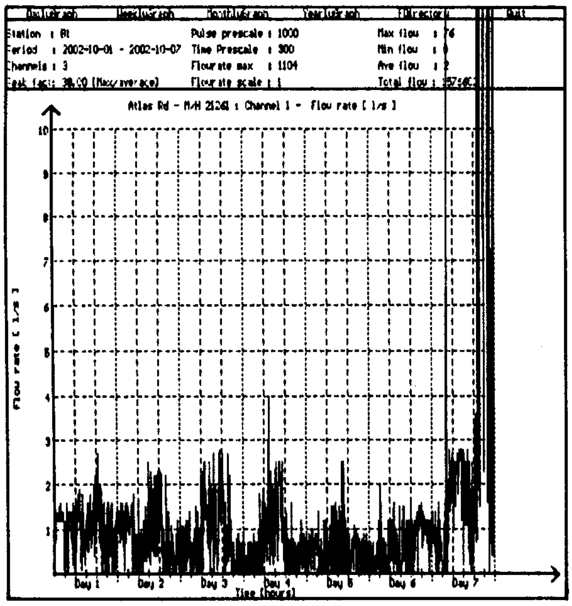
Figure 5.4. Location of sewer monitoring at Boksburg Stadium

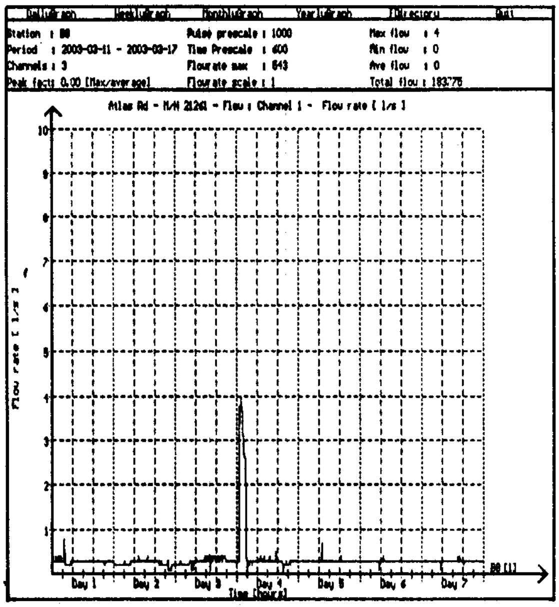
Figures 5.5 and 5.6 illustrate examples of monitored sewer flowed at Boksburg Stadium for each monitoring point at different times.

Manhole 21261 Atlas Road Line(Van Dyk Rd) - Station Number: B1/B8

Pipe Diameter: 300mm Slope: 1:85 Roughness: 0.013

Station 1

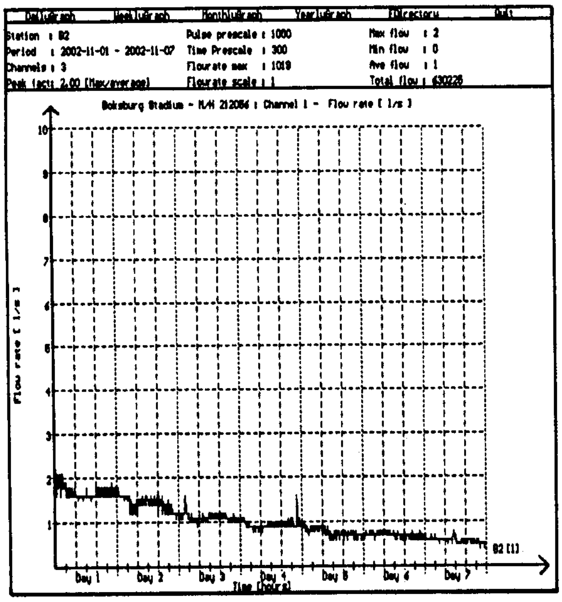




Manhole 212054 Boksburg Stadium Line - Station Number: B2/B5

Pipe Diameter: 240mm Slope: 1:118 Roughness: 0.013

Station 2



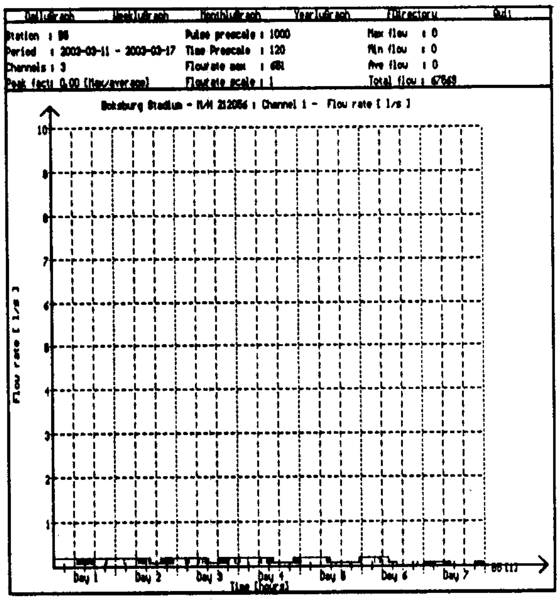


Figure 5.5. Flow monitoring records from the Boksburg Stadium outfall for Station 1 and 2

Manhole 212027 Trichard/Langehoven Line - Station Number: B3/B2
Pipe Diameter: 350mm Slope: 1:140 Roughness: 0.013

Station 3

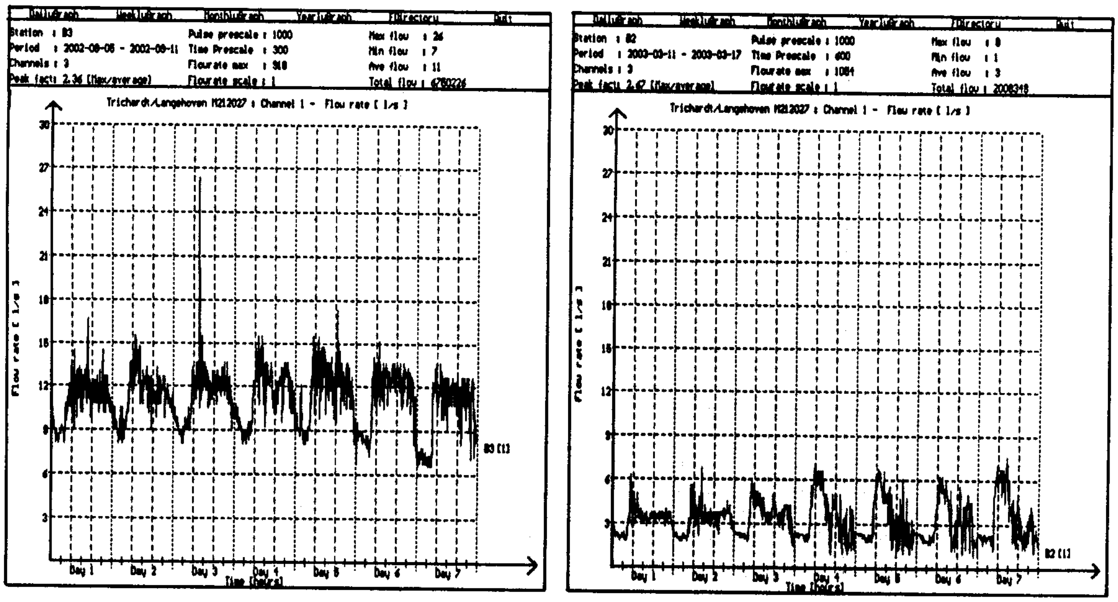


Figure 5.6. Flow monitoring records from the Boksburg Stadium outfall for Station 3

5.2.5 Interpretation of monitoring results

Several sets of monitoring records of sewer flows were obtained for the Boksburg Stadium sewer line at the three selected manholes as shows in Figure 5.4. Not all recorded yielded satisfactory results and monitoring equipment has been reset and calibrated to obtain the best possible results due to the rainfall circumstances. The whole monitoring line of some 3 km was inspected and one blockage was identified and cleared. The installation periods of the monitoring equipment were as follows:

Period 1: From 20 August 2002 to 28 November 2002
Period 2: From 10 March 2003 to 18 March 2003

See Appendix C for a detailed breakdown of monitoring periods and the daily rainfall patterns for the Boksburg urban area.

The monitoring records obtained during the first period (August – November 2002) indicated about three times higher flows than those for the second period (March 2003) at the end of the 2002/03 rainy season. Analysis of the rainfall patterns for the 2001/02 rainy season in comparison to the patterns in 2003/03 when most of the recordings of sewer flows took place indicated that the previous season happened to be extreme and the later season on the contrary well below the long-term average. This leads to the impression that there had already been a reduction in percolation of groundwater into the Boksburg sewer outfall from the beginning of the 2002/03 rainy season instead of an increase as normally expected. However, November 2002 rainfall was the lowest in recent twenty years. No particular stormwater events were recorded. The rate of infiltration of groundwater was most probably at its lowest due to low percolation of surface water during the poor rainy season.

From the records available (examples given in Figures 5.5 and 5.6) it has been determined that groundwater infiltration on the dedicated sewer line between monitoring Station 1 (Atlas Road, Mh 21261) and Station 2 (Boksburg Stadium Mh 212054), 225 mm average pipe diameter and some 3 km pipe length, amounted to 0,09ℓ/min/m-dia/m-pipe (or 0,05 ℓ/s/225 mm dia/ 100m pipe). This would be about 2,5 times higher than the permissible infiltration rate from a 225 mm diameter sewer pipe diameter as indicated by the current Guidelines of Johannesburg Water (Pty) Ltd (2003).

Another phenomenon observed on the monitored line indicated that the quantitative size of flows recorded between Station 1 and Station 2 decreased some 3 km downstream to about half during the extreme dry period. This can be only explained by an anticipated drop in groundwater levels most probably below the sewer invert and the likelihood of exfiltration instead of infiltration cannot be ruled out.

The behaviour of groundwater fluctuation in the Boksburg urban area has been discussed with the regional office of the DWAF, however, no particular measurements are available in close proximity to the monitored sewer to be correlated with the monitored periods undertaken by this research.

Phalafala (2003) used records compiled by GMKS (1999) and the SEWSIM model to determine the rate of groundwater infiltration for the Boksburg stadium catchment aiming to calibrate the measured and calculated wet weather flow. Groundwater infiltration rate was set in the model at 0,0125 m^3/s. Subsequently, for the Boksburg Stadium catchment with total sewer length of 10094m and average diameter of 200mm, the average rate of groundwater infiltration was determined at 0,372 ℓ/min/m diameter/m length. Figure 5.7 illustrates a method in determining the groundwater infiltration rate versus total sewer flow.

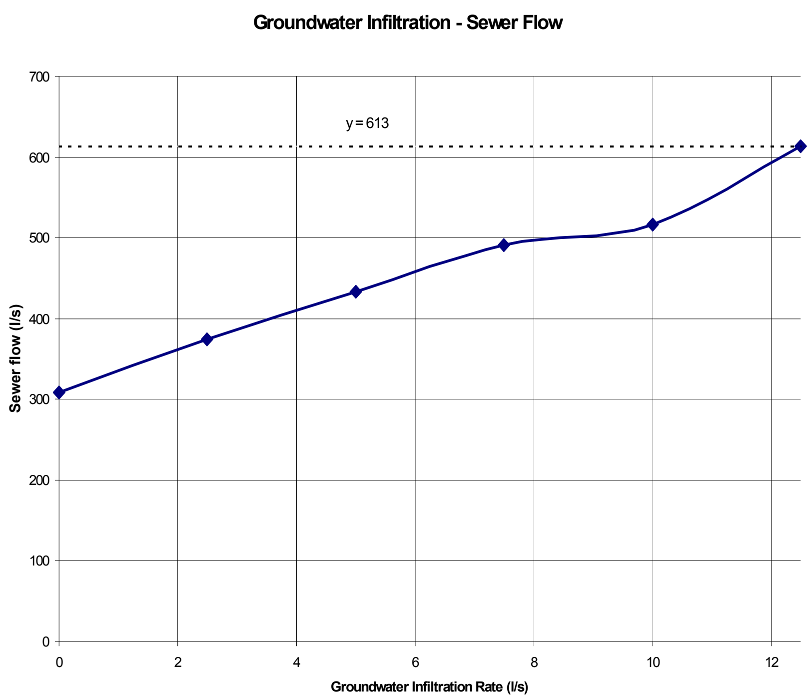


Figure 5.7. The determination of rate of groundwater infiltration

5.2.6 Typical field causes of inflow/infiltration in Boksburg and Benoni

The main causes of stormwater inflows into the sewer systems in the Benoni and Boksburg areas have been identified as follows:

- *Manholes without covers*
- *Stormwater systems connected directly to sewer reticulation system*
- *Sewer gulleys below ground level or rainwater downpipes connected to gulleys*
- *Broken/cracked sewer pipelines*
- *Infiltration through benching in manholes*
- *Infiltration through pipe/channel connections at manholes*
- *Infiltration through deteriorating rigid sewer pipes*

From the technical reports available on the subject of inflow/infiltration in Boksburg and Benoni, the following have been identified as important to the research project:

- *Stormwater and surface inflows account for dramatic peak flows (up to 3 times the AADWF) experienced, particularly in the Boksburg Outfall. The source of the inflows can be attributed predominantly to household stormwater being directed into the sewer system through gulleys, and to a lesser extent, due to missing or damaged manhole covers.*
- *Ground water infiltration produces a steady base flow in the sewers, which increases treatment costs and reduces the operating capacity in downstream sewers. It appears that only in severe cases where the extent of groundwater infiltration may cause structural collapse or substantial reduction in the capacity that pipe replacement or repair makes financial and practical sense.*
- *Both stormwater inflow and groundwater infiltration are continuing to increase due to reactive maintenance instead of planned preventative maintenance and planned rehabilitation programmes.*

CHAPTER 6. METHODS FOR ASSESSING EXTRANEOUS FLOW IN A WATERBORNE SEWER

6.1 Modelling flow conditions in waterborne sanitation systems

6.1.1 Basic methods for determining groundwater infiltration

According to SABS1200, the water losses from a newly installed sewer of 100mm in diameter should not exceed 6 litres in one hour along a 100 metres of pipeline length. When converted to a common unit representation used in evaluating infiltration/exfiltration rates, the following may apply: 0,01 ℓ/min/m-dia/m-pipe (i.e. benchmark permissible value for a new sewer regardless of pipe material).

As pointed out previously, the best way to determine groundwater infiltration rates for a specific sewer installation is a field survey supported by a flow monitoring programme over we and dry periods. However, there are various indicative methods that can be used to determine the rate of groundwater infiltration into sewers. Determining the rate of groundwater infiltration is complex and therefore, the majority of these methods are only approximations. This complexity arises because it is not easy to determine the proportion of sewage flow that represents leakage and which represents groundwater infiltration.

- *Night flow method.* The first method that can be used is to assume that normal night flow in a sewer during summer represents groundwater infiltration. This method is based on the assumption that there should be no flow at night since everybody is asleep. This method can be used only as a very rough guide because it is inaccurate. There is some flow at night from people who are on night duty (e.g. 24-hour shops, filling stations, etc). Urinals with automatic flushing, leaking taps and cisterns also contribute to the flow in the sewer.
- *New vs. old sewer method.* The second method is similar to the first to some extent. Night flow in an old sewer is measured and compared to night flow in a relatively newer sewer. The flow is expected to be more in the older sewer than in the new one. The difference in flows gives groundwater infiltration. This method assumes that the newer sewer has fewer defects than the older one and that the joints are still properly sealed. When using this method, one has to ensure that the two sewers are within the same catchment to ensure that the same soil conditions apply to both sewers. A drawback of this method and the first one is that the volume of water that is thought to represent groundwater infiltration also contains leakage from taps, etc. However, the second method is more accurate than the first one.
- *Dry vs. wet period method.* The third method that can be used takes into consideration the combined effect of stormwater inflow and groundwater infiltration and is explained by Metcalf and Eddy (1991).. To differentiate between the amount of leakage and inflow. With this method, the flow measured at night during the dry weather season is assumed to represent mainly leakage and omits infiltration. This assumption is reasonable because during the dry season when there is no rainfall (this is particularly valid in South Africa), the groundwater table is low, probably below most pipelines.
- *Modelling and calibration analysis.* It is generally recognized that measured data should be given greater credibility than model prediction. Inflow/infiltration/exfiltration events are distinctly site specific. The data collected at each investigated site will be appropriate for a local problem. High resolution data would be required to calibrate any model on inflow/infiltration/exfiltration.

During the wet weather season the groundwater table usually rises above the sewer lines. Flows measured during the wet weather season, therefore, represent leakage, groundwater infiltration and stormwater inflow. It is assumed by most applicable methods that the difference between the flow measured during the wet weather and the dry weather season represents the amount of inflow from groundwater and stormwater. One has to be cautious, however, when using these methods for sewers that are in the vicinity of lakes or dams, because groundwater infiltration in these regions may occur throughout the year.

From literature search and analysis of available studies supported by field testing and monitoring, the following interpretation of generic inflow infiltration/exfiltration conditions are illustrated in Figure 6.1. below:

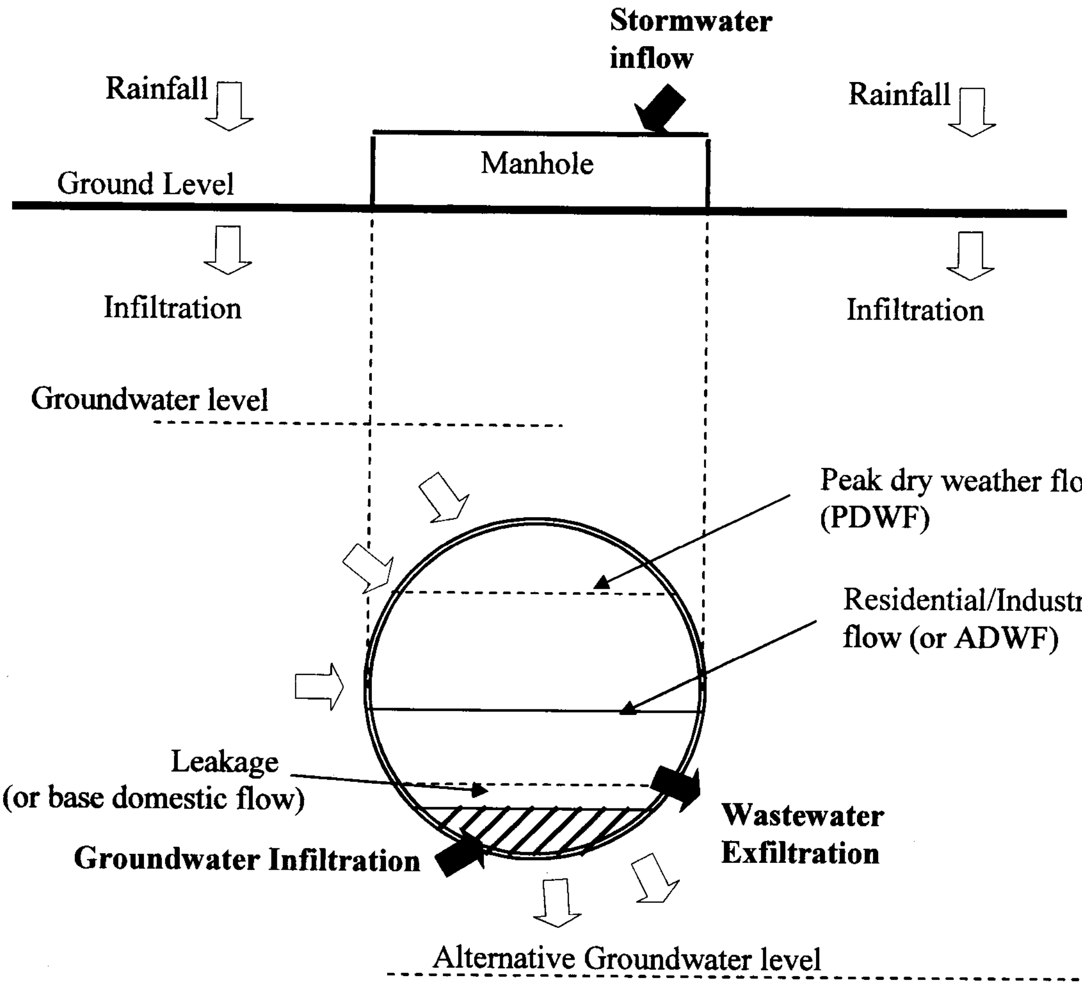


Figure 6.1. Hypothetical illustration of extraneous flows in a waterborne sewer

Infiltration/exfiltration can increase over time due to changes in the environment around a rigid sewer pipe. Depending on pipe material, pipe installation, stiffness, chemical resistance and abrasion/corrosion resistance, over time a rigid sewer pipe will be subjected to three distinct stages leading to failure if not maintained or remedied. Figure 6.2 and Table 6.1 illustrate typical conditions and properties of a rigid sewer pipe with regard to four types of sewer pipe materials.

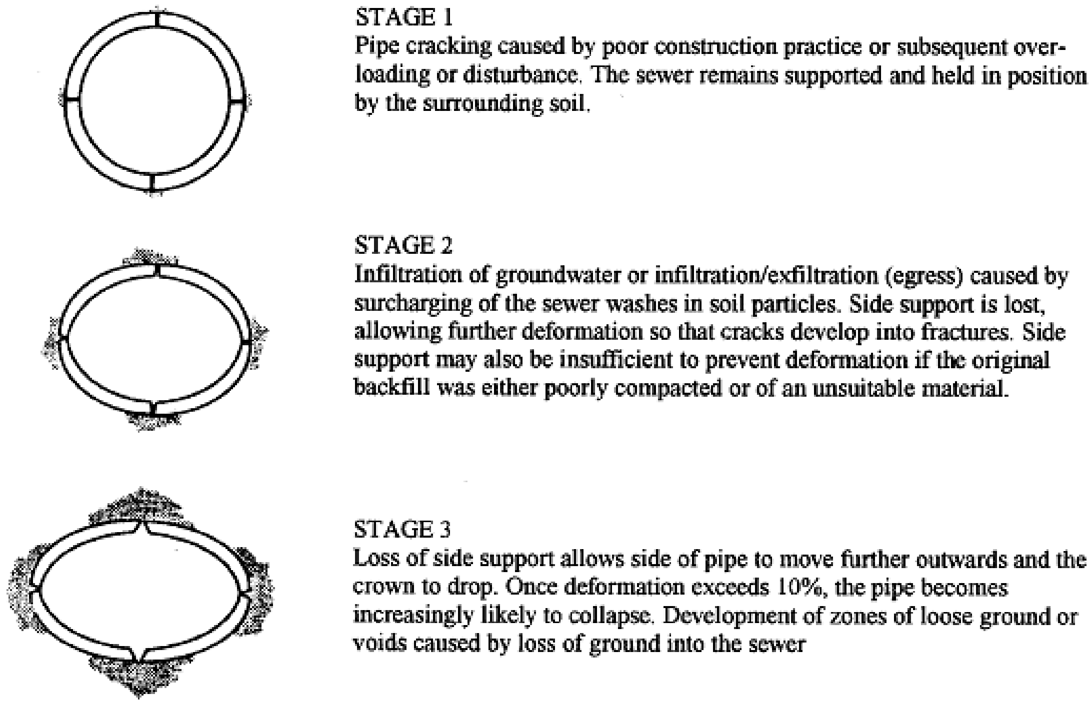


Figure 6.2. Typical progress in failure of a rigid sewer pipe

Table 6.1. Relative properties and life expectancy of sewer materials

Pipe material	Life span	Chemical resistance	Stiffness	Abrasion resistance	Corrosion resistance
Concrete	Medium	Low	High	Low	Medium
Fibre cement	Medium	Medium	High	Low	Medium
PVC	High	High	Medium	Medium	High
HDPE	Medium	High	Low	High	High

Source: Adapted and adjusted from Moss (1983)

6.1.2 Selection of modelling methods and techniques

The purpose of modelling and real-time control in municipal sanitation services reflects the key issues faced by a WSA/WSP in development and operation of urban sanitation infrastructure. The issues can be summarized as follows:

- *Meeting expected levels of service*
- *Improving on design methods*
- *Increasing efficiency of WSA/WSP in managing their assets*
- *Keeping OPEX and CAPEX business targets*
- *Developing an maintaining GIS and asset databases*
- *Reducing OMR costs*

The main approaches adopted in modelling of sewer behaviour and real-time control in urban drainage systems refer to the following:

- *Design of a new system* – the parameters of physical components of a proposed system are determined in order to respond to the specific conditions to derive sufficient system capacity due to extreme flows

- *Analysis or simulation of existing* system – all physical components of the system exist and it is necessary to determine the responses of a system to the particular conditions (e.g. changes in collection magnitude in terms of flow rates and depth, sedimentation, corrosion, surface flooding, etc.).

The contemporary approach in modelling urban drainage systems (i.e. stormwater and wastewater) is conducted with the aid of GIS together with hydraulic modelling software developed either according to deterministic or stochastic methodology. Both methods involve some element of simplification in accepted mathematical relationships between the physical parameters of the components of a system.

- *Deterministic models – one combination of input data will give the same output result, randomness of physical phenomena is not accounted for.*
- *Stochastic models – the randomness is taken into consideration giving an indication of uncertainty in system simulation.*

It should be noted that most simulation models concerned with the hydrological and hydraulic aspects of municipal waterborne sewers are deterministic models (i.e. not necessarily concerned with the random effects of physical phenomena).

6.1.3 Modelling of parameters in municipal waterborne sewers

Realistic simulation of wastewater generation is an important function in a flow-modelling package. The crucial element in such a package is the way in which it simulates unsteady flow conditions.

It has been highlighted previously that wastewater flow varies with the time of day and can vary dramatically during a storm event. This is also when inflow/infiltration into the sewers increases considerably. Stormwater inflow creates a storm wave which is to some extent attenuated as it moved along the sewer system. The rational prediction of unsteady flow may reduce the common overdesigning of municipal waterborne sewers.

The methods used in determining unsteady flow conditions in partially-full sewer pipes are based on approximations of full flow theories when the physics of flow are applied. These theories are derived from the Saint-Venant dynamic and continuity equations. Chadwick and Morfett (1998) can be consulted for further details. However, most contemporary flow-modelling software packages are developed using simplifications in excluding the non-uniform steady state conditions accounted for in the Saint-Venant equations. A common assumption implies that the relationship between flow rate and depth of flood wave are the same as they would be in a steady uniform flow. For practical applications, Ponce et al (1978) quantified non-uniformity condition simplifications as follows:

Kinematic wave: $$TS_o \frac{v_o}{d_o} > 171 \tag{6.1}$$

Diffusion wave: $$TS_o \left(gd_o\right)^{0,5} > 30 \tag{6.2}$$

Where:
- T = duration of flood wave (s)
- S_o = bed slope
- v_o = initial velocity (m/s)
- d_o = initial depth (m)
- g = gravitation acceleration (m^2/s)

6.1.4 GIS data requirements for hydraulic modelling

Since the early 1970s, increasing application of electronic computing power has enabled the development of complex models for urban drainage design and analysis. The same denominator for all available hydraulic modeling software packages is the automation of repetitive tasks and the provision of links to the sources of essential data. Geographical Information System (GIS) databases are considered the most suitable source.

The main purpose for providing a direct link between GIS and modelling software is to facilitate model building and automatically allocate demand using a combination of GIS data with Microsoft Office data files such as Access or Excel and text files (e.g. comma-separated variables). Typical GIS data requirements for a wastewater hydraulic modelling process would comprise the following:

- *Network asset data (e.g. conduits, manholes, etc.)*
- *Contributing areas (or subcatchments) to the collection system*
- *Surface area breakdown (i.e. impermeable areas vs permeable areas)*
- *Population data (present and future)*
- *Rainfall profiles (events, etc.)*
- *GIS facility providing for viewing, image formats, and maps*

GIS generates in computerized form spatial data to assist in the design and management of engineering infrastructure. However, capturing data in electronic format into databases is a crucial task for most WSAs/WSPs. An advanced approach is to capture the relevant data into text format to enter into a computerized script, which then automatically generates a drawing which means conversion of data on computer to a special format.

6.1.5 Approach in selecting available models and computer packages

Several stand-along hydraulic modeling software packages or packages using hydraulic modeling software together with GIS are available to designers and managers operating municipal sanitation services. Although most of these packages are useful tools in the development and management of urban drainage systems, the aspects of inflow/infiltration and exfiltration are only attended to as an arbitrary factor or so-called extraneous flow factor. An approach in selecting any hydraulic modeling software package should account for the following:

- *Automatic derivation of spatial data to all nodes (generally manholes) and links (i.e. pipes and pumps and other features)*
- *Automatic allocation of "demand" at any node*
- *Provision for incorporating geo-referenced information to support the modeling process (e.g. hydraulic condition of a component, etc.)*
- *Provision for importuned data to produce reports*
- *Evaluate the extent of approximations and/or simplifications built into the modelling software package*
- *Assess the means and costs of software maintenance*

6.1.6 Overview of available computer packages on urban drainage issues

(i) SEWER CAD (www.haestad.com/sewerCAD)

Sewer CAD by Haestad Methods (USA based) is a data management tool used in handling large amounts of data required for hydraulic modeling. The application of Sewer CAD enables the designers and managers of wastewater services systems to analyze, design, map, manage and plan the development of wastewater collection systems. Sewer CAD is compatible with AutoCAD, ArcInfo, ArcView or stand-alone applications. It is an integrated framework for hydraulic modeling, graphical editing, results presentation and data sharing and exchange. Although Sewer CAD is know to professionals in South Africa, due to its rather high purchase price it is not widely use by local WSAs/WSPs.

(ii) Civil designer (Design Centre – www.knowbase.co.za)

This software package is based on and redeveloped from Ally CAD-Sewer and Stormwater module by Knowledge Base (011 675-3959). The original sewer and stormwater modules provided a facility for inflows calculated according to the Unit Flow method, the Harman formula, a constant point source and inflow hydrographs. Both sewer and stormwater modules allow for combining of new and existing pipes in one model and include the stardust spreadsheet form-based data input/editing capability. The pipe quantities are calculated according to SABS 1200 with user-defined depth increments.

(iii) Integrated ArcView sewer package

More recent wastewater management models and computer programs are based on the overall sewer system analysis and in most instances with the aid of a geographical information system (GIS). However, most of these programs are the stand-alone sewer programs. The integration of these two components is made possible through specially designed interface to transfer data from GIS to the main program component.

Table 6.2. GIS supposed system framework using ArcView GIS

Program stage	Framework description	Required data and attributes
1.	Define sewer network and assign attributes	Location of manholes, pipe slope, diameter, material, roughness, etc.
2.	Select subsystem to analyse	Pipe-manhole topology, main collectors and branches
3.	Determine sewage flow contribution parameters	Income class categories, per capita sewage production, hourly peak factors, extraneous flow factors
4.	Select sewage contributing areas to specific manholes	Drainage area production (based on per capita product), per area unit flows from non-residential drainage zones
5.	Specify additional flows, perform hydraulic analysis	Min and max flow velocities, capacity of pipes to carry PWWF, Darcy-Weisbach and Colebrook-White formulae, v/v_{full} ratios, Q/Q_{full} ratio, check against allowable limits
6.	Display results	Visualised and analysed geographically

Source: Sinske and Zietsman (2002)

It should be noted that this framework is applied through the geographical user interface (GUI), and the object-oriented internal language avenue is used in programming of sewer system analysis.

(iv) SEWSIM model

The Water Systems Research Group at the University of the Witwatersrand under the supervision of Professor Stephenson initiated and developed a micro-computer oriented version of a program called SEWSIM, providing for simulation of flows down sanitary sewerage networks. The program is designed using the Visual Basic language and is recorded into Microsoft Excel Visual Basic Application (VBA). The SEWSIM model is intended to allow for capacity building of students participating in wastewater and stormwater research projects. The program operates on an input of gravity flow in a sewer and uses pipe connections to route the flow instead of pipe elevation. The program requires the following parameters as an essential input:

- *Sewer diameters (mm)*
- *Sewer lengths (m)*
- *Sewer slopes*
- *Zone types*
- *Equivalent house units (100 m^2)*
- *Leakage (ℓ/min/house equiv.)*
- *Sewage inflow (ℓ/min/house equiv.)*
- *Manning's Constant (n)*
- *Groundwater infiltration rate (ℓ/min/m dia./m length)*
- *Stormwater inflow (mm/h)*

The results of simulation gives the flow in all sewers modelled in the network, and also gives the flow as a percentage of the full sewer capacity. For each sewer, the program requires, in units of millimetres per hour, the amount of rain that enters the sewer network, i.e., what percentage of rainfall ends up in the sewer system. There are several ways in which this proportion can be determined. Stephenson (1986) suggests that the amount of rain that enters the sewer network through gulleys, manholes and leaks is about 1% of the precipitation rate or 10% of the storm runoff. In addition to the above listed data, further information would be required in the application of the SEWSIM model:

- *Contributor Area (m^2):* The area required is the catchment area that contributes to the flow in the sewer system.
- *Rainfall (mm/h):* This is the peak rainfall in units of mm/h recorded on the catchment.
- *Outfall Extra Rain (m^3/s):* At the outfall (or collector sewer), the amount of extra rain in units of m^3/s is required. That is, the peak wet weather flow less the dry weather flow.

In the main program, an additional sheet called "Storm" was introduced to the program with the relevant computation results after the execution of the extra program code. Firstly, the contributor area is multiplied by the rainfall to obtain the discharge, which becomes converted into units of m^3/s. This discharge represents the total catchment discharge due to the rainfall. The extra amount of rain measured at the outfall sewer divided by this value gives the percentage of rainfall that becomes inflow into the sewer system. The rainfall from the input sheet is multiplied by the above percentage to determine how much rainfall (in mm/h) may be considered as a sewer inflow

(v) SEWSAN model

The SEWSAN program was developed by GLS (1997) using SEWSIM as a basis and takes into consideration four components of sewer flow as follows:

- *Domestic flow* – the basic input here is a typical 24-hour unit hydrograph of sewer flow for each erf or land use unit (e.g. 100 m^2 office space). These unit hydrographs may differ significantly in terms of shape, volume and peak flow for the various land uses. Typical values for residential unit hydrographs based on field surveys are:
  Peak flow measured in ℓ/min/erf (typically 1,0 ℓ/min/erf)
  Daily flow measured in ℓ/day/erf (typically 800 in ℓ/day/erf)

- *Leakage* – this component is caused by leaking toilet cisterns, leaking taps, etc. and represents a portion of the base sewer flow (typical leakage flow is assumed at 0,15 ℓ/min/erf). The leakage can be regarded as the base domestic flow.
- *Infiltration* – groundwater seeping through joints/cracks in the pipelines and manholes. Infiltration rates are expressed in ℓ/min/m pipe length/m pipe diameter (typical value is assumed at 0,10).
- *Stormwater inflow* – during rainstorms, runoff enters typically into the sewer system via illegal connection of gullies and inundated manholes. It should ideally not occur in a "closed" sewer system, however it is a reality and typically between 0,5% and 4,0% of all rain falling within 25m on either side of a sewer pipe may find its way into the system, sometimes with significant effect on the sewer flows.

GLS (1997) conducted numerous sewer flow measurements in the Tshwane MM system with a view to establishing typical unit hydrographs for the different land use types, as well as the parameters for leakage, infiltration and stormwater inflow in order to calibrate the SEWSAN model. A further objective was to establish a relationship between water usage and sewer flows. The following criteria were applied for flow measurement locations and methodology:

- *Flow measuring device must preferably be on a straight section of pipe (in a horizontal plane)*
- *Pipe must preferably not change gradient at location*
- *Areas draining to the different locations should represent varying land uses, as well as varying sizes of drainage area*
- *The flow depths were converted to flow rates using Manning's equation for steady, uniform flow, viz.*

$$Q = \frac{A}{n}.R^{2/3}.i^{1/2} \tag{6.3}$$

Where Q = flow rate in m^3/s
A = flow area in m^2
n = Mannings roughness coefficient
R = hydraulic radius in m = flow area/wetted perimeter
i = slope of the pipe in m/m

Verification of flow depth to flow rate conversion is based on the Venturi meter flow depths using the British Standard (BS) equation for the specific flume and comparing to the corresponding flow rates. Good correlation for all flow depths is typically obtained for a Manning, n, of 0,013 for concrete sewer pipes. The following constraints are commonly experienced:

- *That there is a calibration error in the conversion of flow depth to flow rate on the chart recorder of the Venturi meter, leading to inconsistency in the Manning's n of the pipeline*
- *That the Manning's equation with sufficient accuracy converts flow depth to flow rate at a specific location*

Several general adjustments are normally required to be made at different measurement locations for the following reasons:

- *Sewer flow may have been recorded during a period of unnaturally low or high sewer flow for a particular catchment*

- *Some flow dept measurements may contain a “constant” error, i.e. showing a depth either too full or too shallow, which may be ascribed to inaccurate installation. A small “constant” error in flow depth may lead to a fairly significant error inflow rate.*
- *A change in gradient at the measurement location may also cause a “backwater” effect*
- *Obstructions downstream of the measurement location may cause a “backwater” effect, resulting in non-uniform flow.*

The SEWSAN program has been used extensively in the Tshwane MM as well as Ekurheleni MM for the evaluation of wastewater sewer flows. The groundwater infiltration rates observed in these urban areas amounted to 0,03 and 0,04 ℓ/min/m/m respectively. The parameters used in the model with regard to the ADDWF and PDDWF represented about 50 percent and 59 percent of the annual average daily demand (AADD) respectively. Groundwater infiltration during peak flows reached about 20 percent of the PDDWF. The ratio of sewer flow to water AADD was determined to fluctuate between 0,4 and 0,6 in the urban areas where the SEWSAN model has been used.

6.2. Effects of inflow/infiltration/exfiltration on return flows from urban areas

6.2.1 Background on return flows from reclaimed urban wastewater

Only about half of the total urban and industrial water demand is typically released as treated wastewater from South African urban areas. This amount of wastewater increases as the losses from the water services system decrease. A principal tool to reduce such losses is water demand management methods as well as efficient infrastructure asset management.

Grobicki and Cohen (1998) highlighted that increasing demand for water caused by urbanisation and industrialisation must ultimately be matched by increasing intensity of land-based treatment and recycle, or in other words, wastewater reclamation. Depending on the extent of losses from the water services system, and the number of times water is recycled through the system, reclaimed wastewater represents another reliable water source for those water users situated downstream of large urban areas.

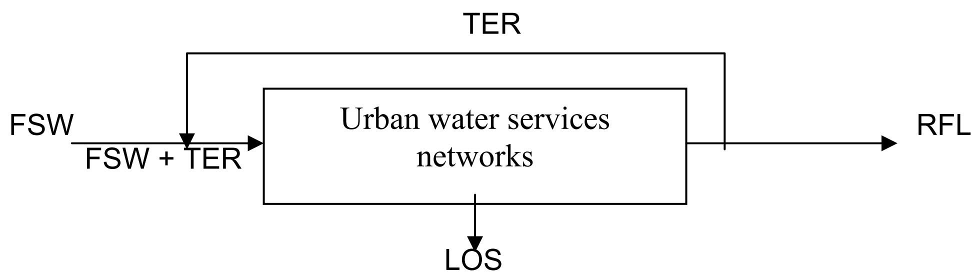


Figure 6.3. Generic model of a simplified water services system with a return flows reuse loop

The inflows to a system must be equal to the outflows, hence a water balance over the entire system gives:

$$\text{FSW} = \text{RFL} + \text{LOS} \qquad (6.4)$$

A water balance over the system itself, which takes into account the flow of reclaimed water, gives:

$$\text{DMD} = \text{FWS} + \text{TER} = \text{RFL} + \text{TER} + \text{LOS} \qquad (6.5)$$

This is equivalent to the water balance shown in eq. (6.5) above. However, this formulation is useful because it shows the total flow of water that is available to satisfy water demand in the system. If the return ratio, r, is introduced, we have:

$$LOS = (1 - r)\ (FSW + TER) = (1 - r)\ DMD \tag{6.6}$$

Where:
- FSW = Fresh water
- TER = Reclaimed water (re-used treated sewage and industrial effluent)
- DMD = FSW + RFL = Total water demand
- LOS = Losses from the system (leakage, irrigation and evaporation)
- RFL = Discharge of effluent to surface water or sea (so-called return flows)

Table 6.3. Reuse applications of reclaimed wastewater

Reuse field	**Possible applications**	**Trade-offs**
Environmental	• Stream flow regulation • Wetlands feed • Recreational use	Baseflow supplement Evaporation balance Relief to water supply
Agricultural	• Irrigation of crops and fodder • Stockfeed water • Aquaculture	Relief to water supply
Groundwater recharge	• Aquifer recharge • Salt intrusion control	Increase in source potential Reserve extension
Industrial	• Cooling and boiler feed	Relief to water supply
Other non-potable/potable uses	• Landscape irrigation • Fire protection • Treatment for potable use	Relief to water supply Reserve extension Reduction in capital

Source: IWSA World Congress (1997)

The losses occurring in a return flows reuse loop can manifest through leakage, evaporation or reuse of treated wastewater. The choice of wastewater reuse applications from the list given in Table 6.3 may lead to the development or refurbishment of a wastewater sanitation system. The concepts available to select from are as follows:

- *Full discharge of treated wastewater (i.e. return flows, no direct reuse of wastewater),*
- *Partial reuse of return flows with reuse loop to non-potable standards (e.g. irrigation, groundwater recharge, etc.)*
- *Partial reuse of return flows with reuse loop to potable standards (e.g. water supply, boiler water, etc.)*

The type and level of wastewater treatment processes and the size of infrastructure will be dictated by the choice of reuse field.

6.2.2 Urban Return Flows Audit (URFA) Model

(i) Structure and assumptions of URFA model

The linkage of water flow between water supply input point and treated effluent output point enables observation of the whole urban water cycle. Water services audit modelling allows for an assessment of the extent of consumptive water use and the actual size of return flows. The return flows are critical for evaluating future water demand in water resources development studies. To assess the degree of utilisation of water in the urban water services systems of South Africa, Barta (1998) proposed a model allowing for sizing of consumptive water and actual return flows. The model is based on the following assumptions:

- *an urban water services system is operated under conditions similar to a developing region (or country)*
- *the climatological conditions are relevant to that of a semi-arid region*
- *the urban water services system has no combined stormwater/sewerage collection and the stormwater is fully separated from sewerage flows. However, stormwater inflow/ groundwater infiltration are included in this model, and*
- *the default values adopted in the model are based on current design criteria applied at present by most water services authorities providers in South Africa*

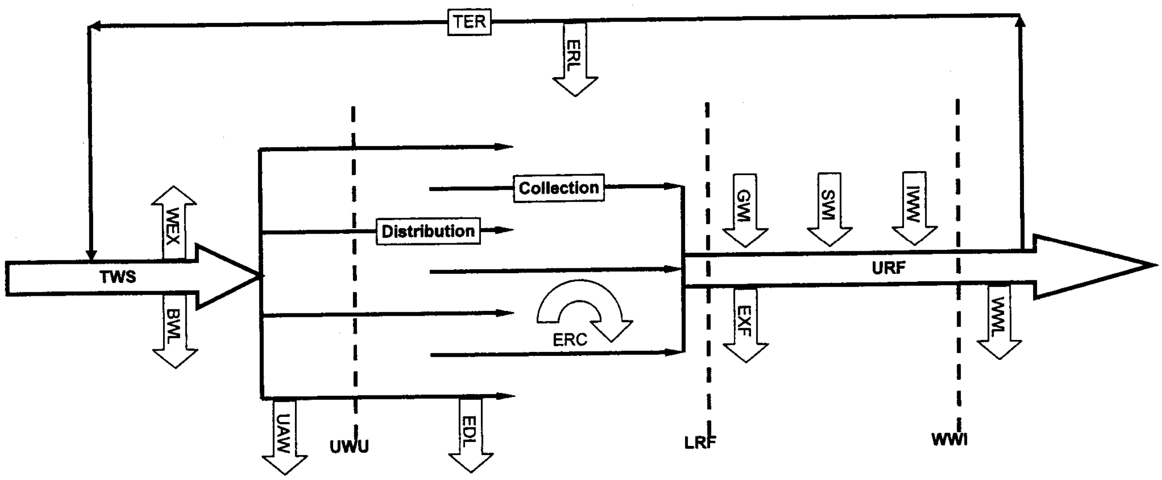


Figure 6.4. Generic layout of urban return flow audit model (URFA) including a reuse loop

(ii) Inflow/infiltration/exfiltration components of URFA model

Applying the URFA model may help funding agencies, developers and managers of water services systems primarily in two ways. The first one is to quantify the real economic efficiency in water use of a system, and the second one enables determination of the actual size of return flows available for further use by another interested water user. The parameters on water quality can be attached to the return flow quantity to evaluate further the real value of water available from an urban water services system. Representation of a water services system without the reuse loop is as follows:

$$TWS + SWI + (GWI - EXF) = WWI + CWU + WEX \qquad (6.7)$$

Where:
TWS = Total water supply from all sources (i.e. local and external)
SWI = Stormwater inflow
GWI = Groundwater infiltration
EXF = Exfiltration from sewers
WWI = Wastewater influent (i.e. waterborne sewerage)
CWU = Consumptive water use
WEX = Water exports

The WWI and CWU components on the right-hand side of Eq. (6.7) are both functions of the processes taking place in the urban water services system.

$$CWU = BWL + UAW + EDL \qquad (6.8)$$

Where:
BWL = Bulk water losses (between 2 and 8% of TWS)
UAW = Unaccounted-for water
EDL = Effluent diffused locally

The UAW component in Eq. (6.8) is a function of technological performance of the urban water services system on the supply side. The UAW value for model operation can be substituted by default values based on typical supply conditions in South Africa. However, good knowledge of a system's water losses is essential in selecting the default value.

$$UAW = ERP + ERS + TFT + UMU + BAW + LKG \quad (6.9)$$

Where:
UAW = Unaccounted-for water
ERP = Error in water purchased
ERS = Errors in water sold
TFT = Water theft
UMU = Unmetered uses
BAW = Bursts and wastages
LKG = Leakages and overflows

Typical default values which can be applied in absence of relevant values for a specific case are shown in Table 6.4.

Table 6.4. Typical values for unaccounted–for water (UAW)

Parameter	**Typical value (%)**	**Reference**
UAW	10 to 35	Of the total water input
ERP	1,5 to 3,0	(+) or (-) of UAW
ERS	10 to 12	(+) or (-) of UAW
TFT	Average 10	As of UAW
UMU (e.g. fire-fighting)	15 to 25	As of UAW
BAW	Average 20	As of UAW
LKG	Average 30	As of UAW

Source: Barta (2000)

The EDL model component representing the effluent diffused locally is based on assessment of wastewater which in principle does not reach waterborne sewer and is predominantly locally disposed by means of septic tanks, oxidation ponds or simply thrown on the ground in most urban areas without sanitation facilities. EDL is a function of the level of services of the end water user and a household access to sanitation services (e.g. pit latrine, chemical toilet, etc.). To determine the indicative values for the EDL component, typical water use values are provided in Table 6.5 below:

Table 6.5. Typical domestic water usage for different levels of service

Type of water supply	**Typical consumption (ℓ/cap/day)**	**Range (ℓ/cap/day)**
Well or standpipe >1000m	7	5-10
Wells at 250m to 1000m	12	10-15
Well <250m	20	15-25
Standpipe <250m	30	25-50
Yard connection	40	20-80
House connection: Single tap	50	30-60
Multiple taps	150	70-250

Source: International Reference Centre (1981)

The WWI component represents the amount of wastewater influent collected from all end water users connected to waterborne sewers providing services to a specific urban area.

$$WWI = WWC + SWI + (GWI - EXF) + IWW \quad (6.10)$$

Where: WWI = Total wastewater influent received from collection network
WWC = Wastewater generated according to specific urban land use (i.e. residential and industrial development)
SWI, GWI and EXF = as previously explained
IWW = wastewater imported into a system, if applicable

$$WWC = (UWW * POP) + IED \qquad (6.11)$$

Where: UWW = Unit wastewater contribution per capita or urban dwelling (or erf)
POP = Population served by waterborne sewer
IED = Industrial effluent discharged (ℓ/day)

Table 6.6. Typical units for urban wastewater production (South African context)

Urban land use development	**Unit wastewater production**	
	Family dwelling outflow	**Per capita outflow**
Residential: • low income • middle income • high income	 0,5 kℓ/day/unit 0,75 kℓ/day/unit 1,00 kℓ/day/unit	 70 ℓ/capita/day 125 ℓ/capita/day 200 ℓ/capita/day
Commercial	0,85 kℓ/day/100 m^2 of floor area	
Industrial	Specific to either dry or wet industry, minimum 0,30 kℓ/day/100 m^2 of floor area	

Source: Based on Red Book (2003) and Tshwane MM By-laws (2003)

6.2.3 Methods to determine inflow/infiltration components for URFA model

(i) Method using field records and measurements

This method is based on the approach of Metcalf and Eddy (1991) in determining SWI/ GWI when essential parameters on inflow/infiltration and physical components of a system are available from field records and measurements. Recommended criterion for an excessive GWI is set at X_9 = 0,10 kℓ/day/mm diameter per km of sewer pipe length (i.e. for old sewers).

Step 1: Determine the average sewer flow during the dry period of the year: X_1(kℓ/day)
Step 2: During the wet period of the year, the flows are averaged, excluding flows subsequent to significant rainfall events: X_2(kℓ/day)
Step 3: Calculate infiltration component as follows: GWI = $X_2 – X_1 = X_3$(kℓ/day)
Step 4: The peak flow generated during a recent storm has been recorded or estimated to determine the inflow to sewers from the hydrograph as the difference between maximum hourly wet-weather flow (X_4) during the storm and comparable flow (X_5) on the *preceding/following* day: SWI = $X_4 – X_5 = X_6$ (kℓ/day)
Step 5: Determine unit infiltration of investigated sewer considering the composite dia-length of the sewer system is X_7 mm-km: $X_3 / X_7 = X_8$ (kℓ/day / mm-km)
Step 6: Evaluate if the GWI is excessive against recognised criteria X_9 (kℓ/day / mm-km)
Step 7: Compare X_8 if $\leq X_9$

(ii) Method using wastewater influent records at the WWTP

Another useful method to determine extraneous flow in an urban area (e.g. wastewater district) where all wastewater is drained into a specific WWTP, is based on a volumetric balance of the water input and output determining the net return flow into a river ecosystem.

The analysed system is assumed to be a typical municipal water services system, where both water supply and sewer reticulation networks are interconnected within the reticulated urban area. A sewer reticulation network must separate residential and industrial effluent from stormwater. Records of water supplied and wastewater inflows collected should be available, preferably for at least a few recent years. The infiltration of groundwater (in kℓ/day/mm-km) for the whole system including the wastewater reuse loop may be determined from the bulk water balance as follows:

$$GWI - EXF = URF - (UWW*POP) - SWI + TER + WWL - IWW \qquad (6.12)$$

Where:
- GWI = groundwater infiltration
- EXF = exfiltration from sewers, typically zero
- SWI = stormwater inflow
- UWW = unit wastewater contribution from a specific urban area
- POP = number of urban dwellers or erf units connected to a waterborne sewer
- TER = total effluent reused
- IWW = wastewater imported into a system
- WWL = wastewater treatment losses
- URF = urban return flows typically metered at a WWTP or may be determined from Equation (6.10):

6.2.4 Net return flows from urban water services systems

The return flows released from point discharge sources include treated wastewater discharges from primarily municipal wastewater treatment plants, industrial effluent plants and mine dewatering processes in some locations. In principle, return flows supplement the base flows of river ecosystems and numerous water users situated downstream of large urban areas are now dependent on regular releases of urban return flows.

Urban return flows are recognized by the DWAF in their NWRS (1999) as an indirect reuse of treated wastewater and important additional water resource in a semi-arid country. Urban return flows are also considered integral to the licensing process in the allocation of permits to mainly industrial and agricultural water users. The regularity of returns, their quantity and quality are critical issues in the allocation of permits.

The size of return flows generated in an urban water services system with a wastewater reuse loop may be determined as follows:

$$URF = WWC - TER - WWL - IWW + (GWI - EXF) + SWI \qquad (6.13)$$

Subsequently: $WWC = TWS - UAW - EDL - WEX - BWL + (TER - ERL) \qquad (6.14)$

Where:
- TWS = total water supplies into investigated area (say 12 months average)
- WEX = water exported from a system
- BWL = bulk water losses (2 to 8% of bulk supply)
- UAW = unaccounted for water including unmetered use
- EDL = effluent diffused locally (i.e. wastewater not reaching a WWTP)
- WWL = wastewater treatment losses (2 to 5% of total wastewater influent)
- WWC = wastewater generated in a specific land use area
- TER = total effluent reused
- IWW = wastewater imported into a system
- ERL = effluent reuse losses
- SWI/GWI/EXF = inflow/infiltration/exfiltration

The net return flows may be subsequently determined as follows:

$$\text{NET URF} = (\text{WWC} - \text{TER} - \text{WWL} - \text{IWW}) - (\text{GWI} - \text{EXF} + \text{SWI}) \qquad (6.15)$$

Refer to Appendix D for worked out example and application of URFA model procedure.

CHAPTER 7. EVALUATION OF CONDITION OF WATERBORNE SEWERS

7.1 Assessment of wastewater system integrity

7.1.1 Wastewater system evaluation analysis

(i) General approach

A thorough evaluation analysis is required to determine the extent of the problems related to the integrity of the whole or a particular component of a wastewater system. An evaluation of the problems will indicate what alternative approaches and costs for rehabilitation versus replacement would be required. The required information must come essentially from the flow monitoring and physical condition assessment (see Appendix E for an overall assessment model). Four phases of evaluation are usually considered.

- *Planning and data gathering*
- *Field inspection programmes*
- *Action plan for measures to be taken*
- *Implementation*

(ii) Planning of investigation and data gathering phase (Phase 1)

- *Information database.* A Water Services Authority/Provider should evaluate the extent and severity of a problem as well as the risks associated with attending to or postponing finding a solution. A preliminary costing outlay has to be prepared. To build up an adequate information database, the following categories of data are required:

  (a) *Maps and drawings, preferably in GIS data form*
  (b) *Maintenance and operation records*
  (c) *Geophysical and weather data*
  (d) *Water supply and wastewater works inflow rates and pollutant loadings*
  (e) *Relevant information from the stormwater and water supply sectors, such as losses, control, etc.*
  (f) *Records of land development in the catchment, i.e. rate of expansion in industrial and residential construction*

The wastewater collection system is normally divided into network zones based upon hydrographic or grade parameters. The location of monitoring points should be identified and methods of diverting flows during inspections and repairs should be considered. An assessment of staff capabilities is essential for successful implementation of OMR programmes. In some instances, staff should been trained beforehand according to the OMR requirements.

- *Data requirements.* The contemporary approach in data requirements for the purposes of effective decision-making are listed in Table 7.1:

Table 7.1: Data attributes and required procedures

Data attribute	Required procedure
Accuracy	All pipe and channel sizes and other physical attributes are known and the connectivity of the system is confirmed
Completeness	All constructed works are identified with no gaps existing in the pipe and channel networks unless confirmed by field study
Spatially defined (GIS)	The location of the network should be referenced to the cadastral or property and road base for presentation of the data in a GIS format
Data transfer (GIS)	Information should be easily transferred to the format required by modern hydraulic modelling products and GIS software
Asset management/ asset condition	Business decision rules using asset condition (likelihood of failure) and consequences of failure should be used to define proactive maintenance, inspection or rehabilitation programmes
Maintenance management	The data information system should link to a maintenance management system for recording incidents and for recording the nature of field operational work undertaken
Quality assurance (QA)	The procedures for editing existing information or adding in more information need to be covered by sound QA programme and incorporate security on who can edit the data
Maintenance reports	The compilation of structural and maintenance grading reports through capturing CCTV inspections in digital and database mode

(iii) System assessment phase/inspection programmes (Phase 2)

Three categories of inspection programmes are recommended to be developed and implemented:

- *Re-evaluation of hydraulic performance*. The original hydraulic parameters need to be re-evaluated by means of existing or developed hydraulic model against real field conditions and demands on a system.
- *Extraneous flows assessment programme.* This programme includes assessment of inflow/infiltration and exfiltration conditions.
- *Structural condition assessment*. A critical issue for all inspections is misinterpretation of the severity of physical defects. A key objective of physical/structural condition assessment procedure is how to effectively detect and locate defects/potential failures to prevent extensive I/I events, exfiltration and collapses which can cause street surface hazards and pipeline blockages and subsequent flooding of properties.

(iv) Action plan for rehabilitation/replacement measures (Phase 3)

During this stage, all feasible rehabilitation/replacement options should be considered and optimised in order to propose and develop suitable and affordable methods and procedures.

- *Preventative and remedial measures* – including sealing of sewers replacing of missing or broken manhole covers, raising manholes above flood lines, introducing regular measures and training programmes, increasing the capacity of the sewer system or the WWTP, etc.
- *Rehabilitation measures* – including non-structural lining and/or structural lining.
- *Replacement measures* – including trenchless replacement, pipe bursting microtunneling, horizontal directional drilling and open trench replacement.

(v) Rehabilitation/replacement implementation phase (Phase 4)

This phase is organised and procured according to the implementation schedule including all project activities. Recommended corrective actions will be implemented according to recognised tendering (or public-private-partnership, outsourcing, etc.) procedures. Factors influencing the choice of methods and materials during the implementation phase are as follows:

- *Accessibility to the construction site*
- *Magnitude of flows during the implementation period*
- *Availability of bypassing or rerouting flows during construction*
- *Soil conditions*
- *Stress conditions*
- *Level of groundwater conditions*
- *Lateral connections and dissolved oxygen levels*
- *Length and size of damaged pipelines*
- *Bedding and backfill materials*

This phase should also include preparation of monitoring programme for the post rehabilitation/replacement period. As-built details should be strictly recorded for further reference and adjustment of hydraulic model parameters.

Table 7.2. Typical groundwater infiltration values (ℓ/min/m-dia/m-pipe)

Groundwater infiltration	**Type of sewer**	**Remarks on sewer characteristics**	**Source of information**
0,05	Separate	Monitored value from Johannesburg clay/ concrete sewers typically 30 to 60 years old	Hine & Stephenson (1985)
0,10	Combined/ separate	Textbook value. No details know on sewer material and age	Qasim (1986)
0,01-0,70	Combined/ separate	Internationally recognised range of values. No details known on sewer materials and age	Metcalf & Eddy (1991)
0,05	Separate	Measured value from Cape Town clay/concrete sewers typically 20 to 40 years old	Pollet (1994)
0,03-0,04	Separate	Measured values from Pretoria clay/concrete sewers of 150 to 900 mm in diameter typically not older than 40 years	GLS Inc (1997)
0,02-0,08	Combined	Estimated value for UK purposes predominantly for old clay sewer pipes	CIRIA (1998)
0,048 to add to design rate	Separate	Design allowance mainly for clay and concrete sewer pipelines	Johannesburg Water (Pty) Ltd
0,01	Separate	Permissible wastewater loss from new sewer	SABS1200
15% allowance of ADWF to add	Separate	Predominately for clay and concrete sewer pipes	Red Book (2003)

7.2 Maintenance and enhancement of waterborne sewers

7.2.1 Poor wastewater infrastructure asset performance

The water services authorities (i.e. predominantly municipalities in South Africa) manage their water services infrastructure (i.e. water supply and sanitation assets) under the practices of scheduled and unscheduled maintenance programmes. However, the capital budgets for the maintenance of wastewater collection and treatment subsystems or their components are commonly determined from and based upon historical unscheduled (or reactive) maintenance events. This leads to inadequate maintenance, particularly of the buried infrastructural assets (e.g. pipelines). The "vicious circle" of poor wastewater infrastructural asset performance is caused usually by resuming to reactive maintenance as shown below:

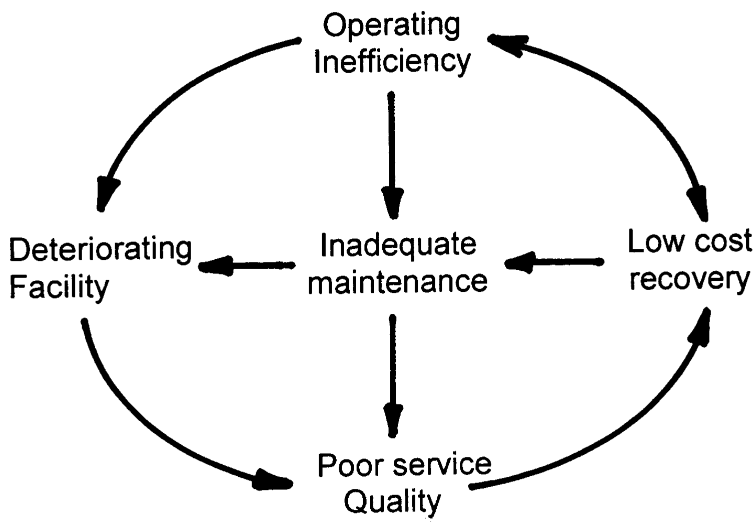


Figure 7.1: The vicious circle of inadequate performance of infrastructural assets

Typical consequences of inadequate infrastructural asset maintenance are as follows:

- *Inability to expand, modernise or improve the service*
- *Increased economic and financial costs*
- *Lost growth in income, lost development opportunities, environmental quality and social welfare*

7.2.2 Assessment of reliability of a wastewater system

The economic management of a municipal waterborne sewer system can be ensured through effective system operation, maintenance and rehabilitation programmes. To enable this, a WSA/WSP must provide that the following objectives are accomplished:

- *the structural integrity of each component of the system is maintained most of the time,*
- *hydraulic parameters should comply with recognised standards and codes of practice,*
- *extraneous flows (infiltration/inflow) are reduced to an acceptable minimum, and*
- *exfiltration and the potential for groundwater contamination and other environmental impacts are limited and preferably avoided altogether.*

Many WSAs/WSPs in South Africa do not have a formal method for determining how much maintenance is needed to achieve a specific level of system performance and therefore adequately justifying maintenance and expansion costs. Figure 7.2 illustrates the principle of regular maintenance against a no maintenance approach.

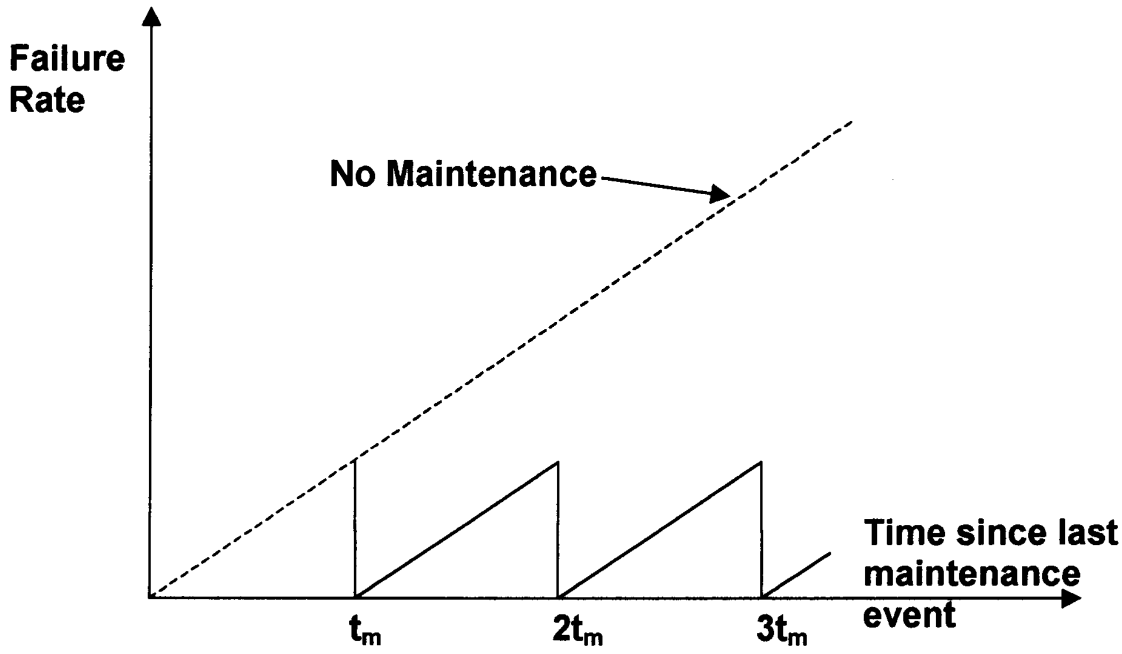


Figure 7.2. Maintenance event time vs. linear failure rate

From Figure 7.2 it can be seen that regular adequate maintenance will considerably decrease the failure rate of a system's components and subsequently upholding the integrity of the whole system.

7.2.3 Corrective (or unplanned) maintenance

Corrective maintenance is an unscheduled activity in reaction to unexpected outages, blockages and breakages.

- *Mean corrective maintenance time (MCMT)* – is the ratio between the total number of maintenance hours to the total number of maintenance actions taken. It should distinguish between existing and new components (or equipment).

7.2.4 Preventative (or planned) maintenance

Preventative maintenance is scheduled activity which is proactive in maintaining a system's components to avoid possible outages, blockages and breakages.

- *Mean preventative maintenance time (MPMT)* refers to the procedures required to retain a system (or its components) at a specific level of performance and include periodic inspections, servicing, scheduled replacement of critical items, calibration and overhauls.
- *Mean active maintenance time (MANT)* is the average elapsed time required to perform scheduled (preventative) and unscheduled (corrective) maintenance.

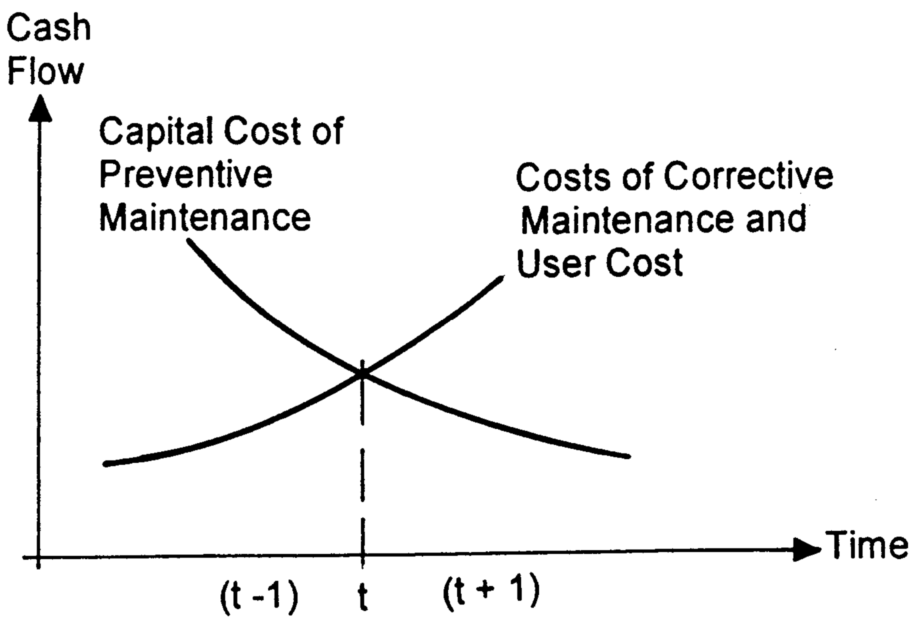


Figure 7.3: Optimal point in time for preventative maintenance

7.2.5 Approach to asset condition assessment

It is considered critical if the decision-makers within a WSA or WSP do not have a clear knowledge of the condition of the waterborne sewer system with regard to the collection, treatment and disposal components. All management decisions regarding operation, maintenance and rehabilitation revolve primarily around the condition of a system and its components. Not knowing the current condition and performance levels of a system's components leads usually to the premature failure of a component or the whole system. This leaves the WSA or WSP with only one option and that is to replace the components which would be generally considered as the most expensive option. The consequences of not providing routine services for a system might constitute irreplaceable losses, litigation and general loss of consumer confidence in the WSA or WSP.

7.2.6 Check-list of critical issues for system management

The most pertinent issues which are critical to the condition assessment procedures can be summarized as follows:

- *Where is the system component in its life cycle?*
- *When was the component constructed, replaced or rehabilitated?*
- *What is the component's effective (or theoretical) life?*
- *What would be the residual life (actual or estimated) until:*
  - *(a) Replacement is necessary?*
  - *(b) Rehabilitation is required?*
- *What methods or techniques were used in the condition assessment?*
- *Can the decay (deterioration) rate be predicted?*
- *Can the complete failure be estimated?*
- *Can planned maintenance prevent failure or extend the time to failure?*
- *Can the component be rehabilitated (Yes/No)?*
- *What is the cost of the component's rehabilitation?*
- *What level of service will the asset deliver once rehabilitated?*
- *Is the component technically or commercially obsolete?*

7.3 Assessment of the condition of wastewater infrastructure assets

(i) General assessment methods

Assessment of the physical condition of the various components of a wastewater system is critical for the repair or replacement programmes based on the inspection programmes (or sewer testing) determining I/I and exfiltration. Although increased flows at the WWTP would indicate problems in the system, location and the associated risks have to be determined from field inspection and testing. Assessments are based on inspections and include smoke testing, man entry, flow isolation, dye-water-flooding and use of closed circuit television (CCTV).

It should be noted that most inspection techniques depend on visual observation and subjective judgments. The location of potential defects may be missed or misinterpreted if the evaluator has not had adequate training.

(ii) House-to-house surveys

This type of field survey is conducted in order to identify sources of inflow originating within homes and other buildings. During a home inspection, the evaluator (or inspector) may identify the non-compliance of residential properties with municipal stormwater and wastewater disposal by-laws requirements.

(iii) Visual inspection by man entry to sewers

Physical inspections by workers are costly and potentially dangerous due to possible rapid flooding, toxic gases and potential sewer collapses, and used only if no other means are available.

(iv) Testing by smoke draft method

The smoke test method cannot usually locate small leaks. However, this method of testing is relatively inexpensive and quick in detecting inflow sources in a sewer system, particularly from roof down pipes, area drains, foundation drains, abandoned building sewers and faulty service connections. The smoke will escape from all inflow sources that are cross-connected to the sewer section being tested.

(v) Flooding or dye-water testing

The dye (Rhodamine B) is used in table form to minimise exposure to field personnel. The nearest downstream maintenance hole is used to watch for the appearance of the dye. Dye testing is normally used to complement smoke testing of suspect areas.

(vi) Closed Circuit Television (CCTV) monitoring

In principle, all CCTV inspection methods are limited by the diameter of the sewer, type of pipe material used and odd shapes and sumps built into the collection system.

- *Mainline CCTV monitoring.* The speed and travel direction of the camera is controlled by the operator who can identify actual leaks, pipe cracks or accumulations of mineral build up. Significant flows of clear water from a service tributary line can also be identified. It is imperative to clean the sewer system prior to CCTV monitoring for effective observations.

- *Service line mini-camera CCTV monitoring*. Inspection of service (or lateral) lines is conducted with specially designed equipment and the objective is to gather detailed information on the sources of the infiltration and an essential insight into rehabilitation costs and techniques.

(vii) Testing by pressure on seals

It is internationally recognized that this testing method is economical to apply, but requires a specific type of equipment which includes a cylindrical packer with inflatable end elements. Defects might be present at lateral service mount connector fittings and sewer joints. The rubber end elements are inflated to isolate the pipe joint, which is then tested under air or water pressure. The test is normally controlled and monitored by CCTV.

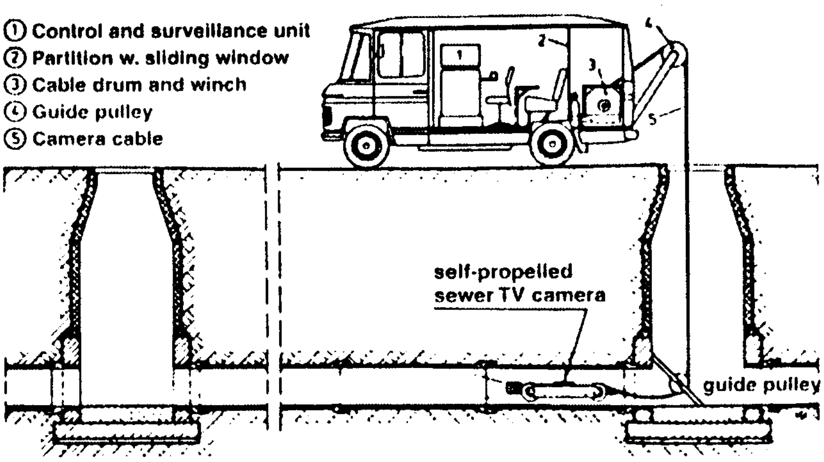


Figure 7.4. Schematic representation of current visual sewer inspection

(viii) Sewer manhole and junction chamber surveys

Manhole surveys are normally conducted during wet weather (preferably during heavy rainfall periods) when physical observations can be effectively conducted. Rainfall inflow or ponding round or over the manhole cover will indicate that corrective action needs to be taken such as sealing the manhole or lifting it above ground level, etc.

(ix) Pumpstations and other appurtenances

Pumpstations and other appurtenances (e.g. wet wells, siphons, etc.) are typically inspected during routine maintenance of mechanical and electrical components. It is estimated that between 30 and 50 percent of sewer system I/I events are due to defects in or near a system's appurtenances. The rehabilitation of manholes, pumping stations, wet wells and siphons can include spray-on coatings, spot repairs, structural liners (e.g. high density polyethylene), grouting or replacement of whole components.

7.4 Management alternatives in waterborne sewer collection systems

(i) Management of sewer flushing

To alleviate sedimentation particularly of new sewer systems due to dry-weather deposition, regular sewer flush waves can effectively convey sewer deposits including organic matter.

(ii) Polymers to increase sewer capacity

Current international research has shown that polymeric injection can greatly increase flow capacity by reducing wall friction. Cost savings are realized by eliminating construction of relief structures.

(iii) Management of cross-contamination in separate sewers

In some instances extensive contamination between residential and industrial sewerage loads requires balancing by means of chemical and/or biochemical intervention. Investigation of domestic and industrial sewerage loadings in municipal wastewater systems can be done using visual observations and screening/mass balance techniques by quality sensing to determine the loading proportions. In extreme cases when industrial sewerage loadings are excessive, pretreatment of industrial sewerage has to be prescribed.

(iv) Managing lack of flow-control ability

Devices such as fluidic regulators, swirl and helical flow regulators, and vortex energy dissipators can be installed in the sewer systems. The main objective of these devices is to impact on liquid-solids separation or to sustain virtually constant flow rates, compatible with the sewer system capacity downstream.

7.5 Condition assessment of a waterborne sewer system and its components

7.5.1 Lifespan of wastewater infrastructure assets

By all practical terms, most of the existing urban water services infrastructure in South Africa (e.g. water supply distribution and wastewater collection networks) is relatively new and technologically compatible with international standards. Urban water services facilities are built on average for a minimum period of 30 years before full utilization is reached. The lead-time from inception to full commissioning of an urban water services project can reach up to five years. Some projects are augmented in stages over a period of 15 years before reaching full capacity utilization.

In the municipal sector, South African Government Treasury Department GAAP and GAMAP standards are required to be applied in the assessment of financial matters regarding infrastructural management. Standards on financial reporting by local government authorities (i.e. Water Services Authorities) are set out to assist them in making and evaluating decisions on allocating their scarce financial resources. life-cycle costing principles must be applied.

The municipal wastewater infrastructure assets are the long-life passive assets and highlight the difficulty and inability to predict with a high degree of confidence the point of time when failure or decline in level of service is likely to occur. The ability of a WSA in managing its infrastructure asset base and particularly the ageing asset problem with its associated risks is a major issue for most WSA/WSPs in South Africa. Health, environmental and community complaints and hazards will increase with the deterioration particularly of service levels of the wastewater infrastructure and subsequently extent of recurring expenditure problems.

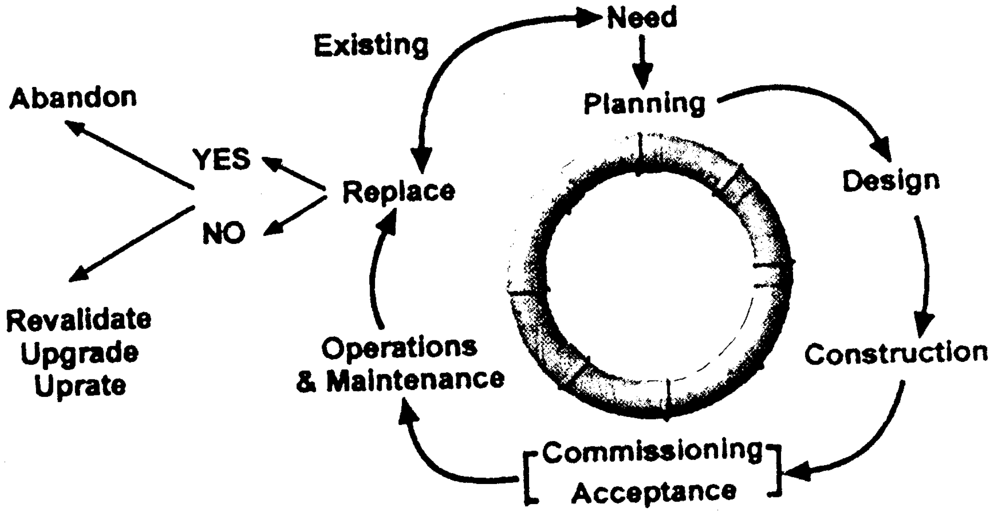


Figure 7.5. Life-cycle of a waterborne sewer pipeline (after Haswell, 1999)

7.5.2 Larger or smaller wastewater system development

To optimize a partial or stage sewerage development, large schemes should be considered for eventual development. The shorter the period over which a facility is used at less than capacity, the higher will be the discounted cost of under-utilization, resulting in an increase of the unit cost. In general terms, the faster the utilization, the lower the unit cost in the intermediate years of the life span of an installation. Also, regional schemes rather than local schemes would produce services at a lower unit cost. This involves many smaller schemes being interlinked to form a regional grid. Although some WSPs in South Africa lean towards a regionalization of rural and semi-urban water services schemes, it appears that centralized urban wastewater plants are not favoured by municipalities. There is an ongoing debate as to whether to centralize or decentralize municipal wastewater treatment. The main argument is economies of scale versus a shift to placing more responsibility for treatment towards developers and/or individual house owners.

7.5.3 Capacity building of technologically educated staff

It is now becoming obvious that the local pool of technologically educated operators and managers undertaking maintenance and operational procedures started to lag behind the demand for such technological qualifications. Another worrying issue is a scarcity of general resources which is rapidly setting in within the aging and deteriorating South African civil engineering infrastructure industry. This is most obvious and urgent at local government level (i.e. WSAs).

7.5.4 Typical quantification methods for condition of a system's components

(i) Sewer component condition ranking

The circumstances prevailing in infrastructure asset management in the highly diversified South Africa water services provision sector would be satisfied by a simplified and practical approach for the asset condition assessment as illustrated in Figure 7.4. Both passive system components (e.g. mains, pipe networks, etc.) and dynamic system components (e.g. pumps, plant and equipment) can be assessed according to five categories (or ranks) to determine the condition of the relevant asset.

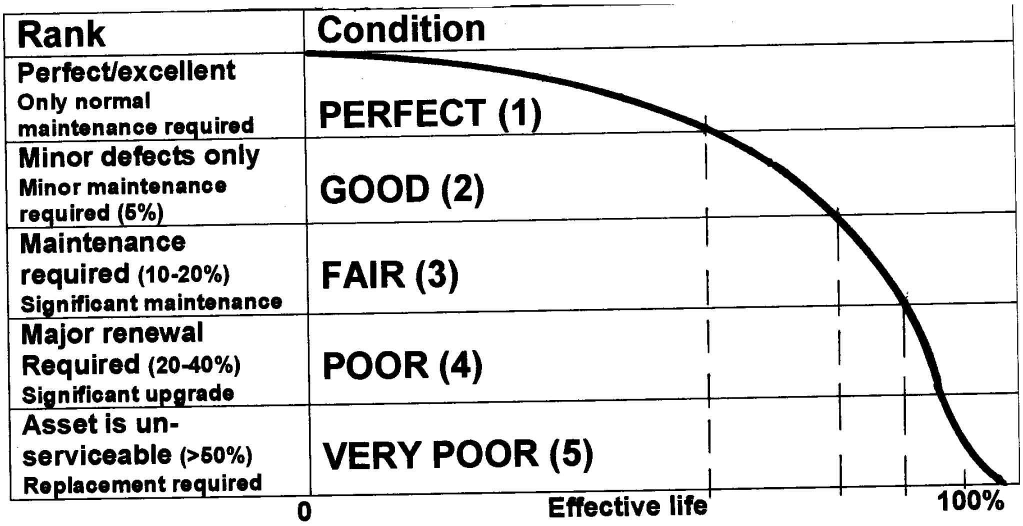


Figure 7.6. Typical asset condition ranking (adopted and adjusted from NZ Infrastructure Asset MM, 1996)

(ii) Written down value (WDV)

Using straight line depreciation, the written down value can be determined as follows and in South Africa, the values to use are recommended by GAAP and GAMAP:

WDV = (effective life – life to date) * replacement value / effective life (6.4)

(iii) Methodology in determining economic life of an existing asset

The economic life of an infrastructural asset is defined from the so-called "bath tub curve" by various techniques (e.g. age factor or utilisation factor method).

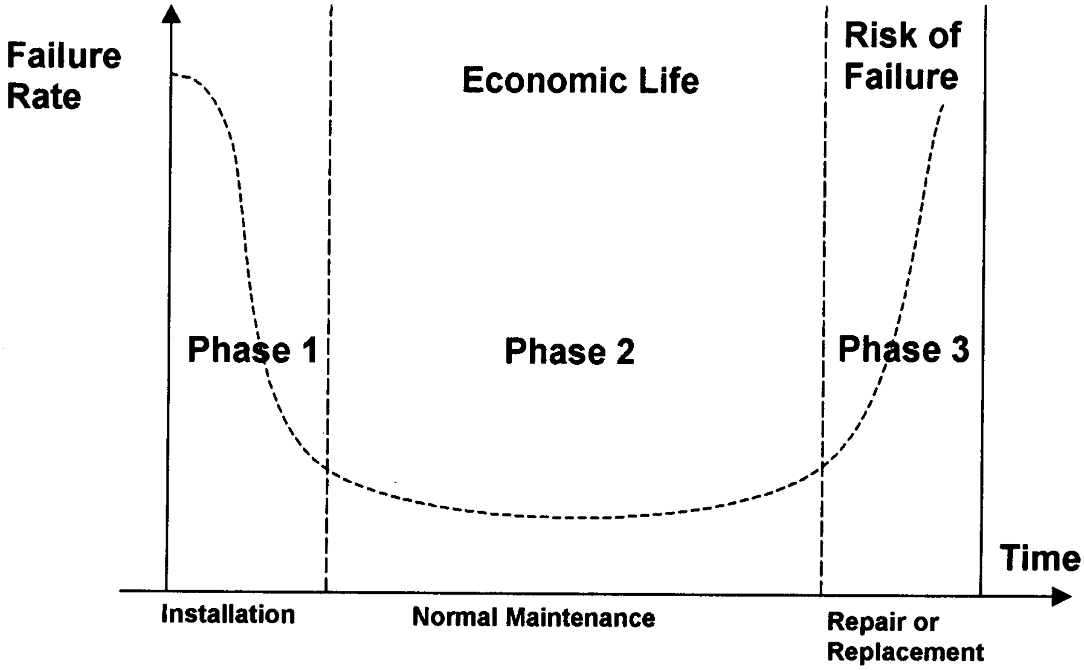


Figure 7.7. Representation of reliability by the “bath tub curve”

(iv) Determination of economic life using Age Factor technique

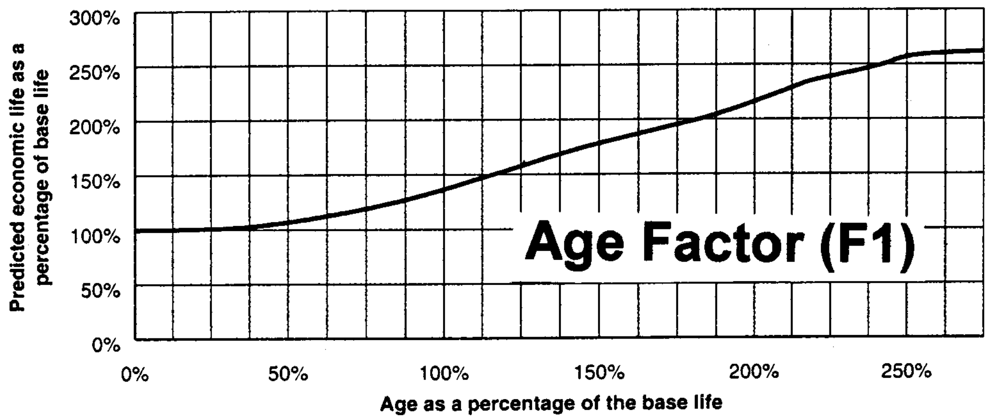


Figure 7.8. Prediction of asset economic life using Age Factor (F1)

Example: Asset useful (base) life = 40 years
Asset current age = 25 years

Therefore the age of the asset as a percentage of the estimated service life = 25/40 = 62.5%
Therefore the economic life of the asset as a percentage base life = 110% (from graph)
Therefore the economic life of the asset = 40 x 1.10 (F1) = 44 years
Therefore the remaining economic life of the asset = 44 – 25 = 19 years

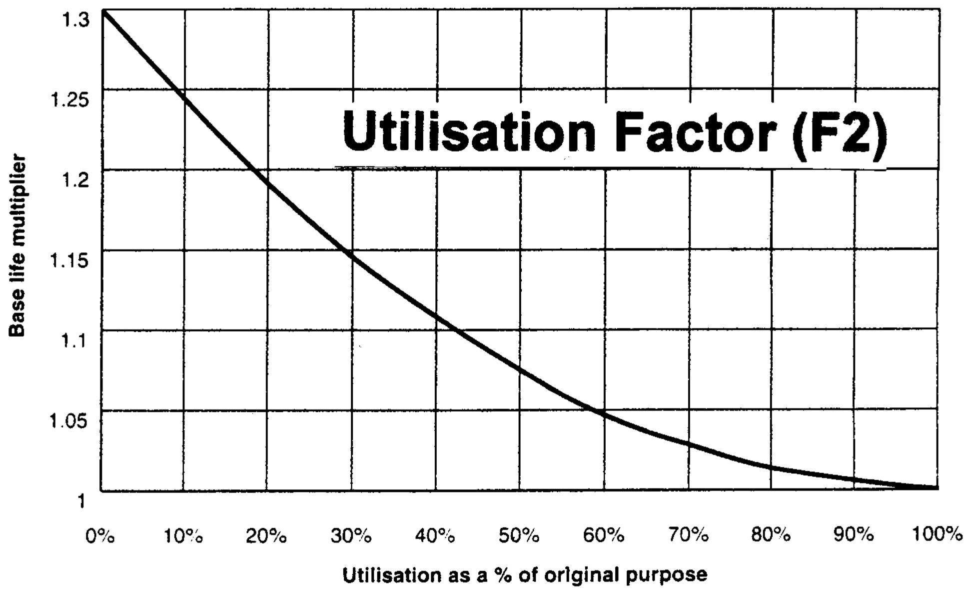


Figure 7.9. Prediction of asset economic life using Utilisation Factor F2

Example: Asset useful (base) life = 40 years
Asset current age = 25 years

An asset has been utilised 30% of the original expectation
Therefore the baseline multiplier is 1.15 (F2)
The asset is predicted to have an economic life factor of 1.15 (F2)
Therefore the economic life =40 x 1.15 (F1) = 46 years
Therefore the remaining economic life is = 46 – 25 = 21 years

GAAP (1998) postulates that the useful life of an asset may be shorter than its economic life, considering that utilities management policy may entail disposing of assets after a specified time. It is recommended that the useful life should be estimated after considering the following:

- expected physical wear and tear,
- obsolescence, and
- legal or other limits on the use of the asset

Depreciation is commonly based on the useful life allowing for a high residual value.

CHAPTER 8. MANAGEMENT ALTERNATIVES TO CONTROL EXTRANEOUS FLOWS

8.1 Decision making on replacement/rehabilitation

8.1.1 Typical alternative methods for rehabilitation or replacement

(i) Choice of suitable pipe material

The choice of suitable pipe material and construction techniques could reduce future rehabilitation requirements as the wastewater collection system ages. It is now known that concrete sewer pipes corrode from sulphuric acid formation. Vitrified clay pipes (VCP) have historically had problems due to leaking joints, short segment lengths and brittleness. These conventional materials are gradually being replaced by plastic materials such as:

- *High density polyethylene (HDPC)*
- *Polyvinyl chloride (PVC)*
- *Reinforced plastic mortar (RPM)*
- *Centrifugally cast fibreglass reinforced plastic mortar (CCFRPM)*
- *Polymer concrete, and*
- *Acrylomitrile-but-adiene-styrene (ABS)*

It should be noted that although plastic materials resist chemical corrosion and provide a root free service, they are not rigid and tend to creep over time. Problems of damage by rodents and crushing from heavy loads are also rather common.

Pipeline rehabilitation methods use the existing pipe either to form part of the new pipeline or to support a new lining. Rehabilitation is preceded by cleaning the pipe to remove scale, tuberculation, corrosion and other foreign matter. Linings, to be effective, must make intimate contact with the pipe surface. Proper surface preparation significantly affects the strength and bonding of lining. These methods can be divided into two categories: non-structural and structural.

(ii) Rehabilitation by non-structural lining

Non-structural lining involves placing a thin coating of corrosion-resistant material on the inner surface of the pipe. The coating is applied to prevent leaks and increase the service life. However, coating does not increase the structural integrity of the pipe.

- *Cement mortar lining*. Cement mortar linings are unique because they are porous. Corrosion protection is achieved by the development of a highly alkaline environment within the pores, which is a result of the production of calcium hydroxide during cement hydration. Cement mortar is applied using a variety of equipment, depending on pipe size and overall project length. Access to the pipeline is accomplished by excavation and removal of a length of pipe.
- *Epoxy lining*. Epoxy resin lining of water mains is an alternative to cement mortar lining. It has not been widely used in the United States. However, it has been practiced in several other countries, including the United Kingdom and Japan. Epoxy lining is envisaged to increase the estimated life in excess of 75 years.

(iii) Rehabilitation by structural lining

Structural lining involves placing a watertight structure in immediate contact with the inner surface of a cleaned pipe. A variety of technologies are available, including sliplining, cured-in-place pipe, fold and form pipe, and closed-fit pipe lining. These rehabilitation techniques improve the structural integrity of a pipe.

- *Sliplining*. Sliplining is the oldest rehabilitation method. In this process a new pipeline of a diameter smaller than the pipe being repaired is inserted into the defective pipe and the annulus grouted. It has the merit of simplicity and is relatively inexpensive, but there is a reduction in flow capacity (35 to 60%), depending upon pipe size. Excavation is required for insertion and receiving pits. All service connections, valves, bends and appurtenances must be individually excavated and connected to the new main.
- *Cured-in-place pipe*. Cured-in-place pipe (CIPP) involves placing a fabric tube, impregnated with a thermosetting resin that hardens into a structurally sound jointless pipe when exposed to hot circulating water or steam into a cleaned host pipe, using the inversion process described below. Access to the pipeline is accomplished by excavation and removal of a length of pipe. There is no reduction in flow capacity. However, the flow must be completely stopped or by-passed during installation and curing. All service connections, valves, bends and appurtenances must be individually excavated and connected to the new main.
- *Fold and form pipe*. Fold and form pipe (FFP) utilizes thermoplastic materials polyvinylchloride (PVC) or polyethylene (PE) that are heated and deformed at the factory from a circular to a U-shape to produce a net cross section than can be easily fed into the pipe to be rehabilitated. This method requires that all service connections, valves, bends and appurtenances must be individually excavated and connected to the new main.
- *Close-fit pipe*. Close-fit pipe lining involves pulling a continuous lining pipe that has been deformed temporarily so that its profile is smaller than the inner diameter of the host pipe. This lining method is often referred to as the modified sliplining approach. Close-fit pipe lining makes use of the properties of PE or PVC to allow temporary reduction in diameter and change in shape prior to insertion in the defective pipe. As with sliplining, excavation is required for insertion and receiving pits. All service connections, valves, bends and appurtenances must be individually excavated and connected to the new main. Close-fit pipe has a design life of greater than 50 years.

8.1.2 Trench replacement techniques

Replacement of pipelines can be accomplished by using either trenchless or open-trench techniques.

(i) Trenchless replacement

Replacement of pipelines means installing a new pipeline without incorporating the existing pipelines by either open-cut or trenchless replacement. Trenchless replacement involves inserting a new pipe along or near the existing pipe without requiring extensive excavation of soil. Trenchless replacement can be done with minimal disruption to surface traffic, business and other activities, in contrast to open trenching.

There is a significant reduction of the social costs associated with construction. The best-known trenchless replacement techniques are pipe bursting, microtunneling and horizontal directional drilling.

- *Pipe bursting*. Pipe bursting is a method for replacing pipe by bursting from within while simultaneously pulling in a new pipe. The method involves the use of a static, pneumatic or hydraulic pipe-bursting tool drawn through the inside of the pipe by a winched cable, with the new pipe attached behind the tool. The bursting tool breaks the old pipe by applying radical force against the pipe and then pushes pipe fragments into the surrounding soil. The liner pipe can be the same size or as much as two pipe sizes larger than the existing pipe. Excavation is required for insertion and receiving pits.
- *Microtunneling*. Microtunneling involves the use of a remotely-controlled, laser-guided, pipe-jacking system that forces a new pipe horizontally through the ground. This trenchless method is used for construction pipelines to close (250mm) tolerances for line and grade. This method can be cost-effective compared to open-cut construction when pipelines are to be installed in congested urban or environmentally sensitive areas, at depths greater than 0,6m in unstable ground, or below the water table. Microtuneling can be used in a variety of soil conditions from soft clay to rock, or even when there are boulders to deal with, and can be used at depths of up to 30m below the water table without dewatering.
- *Horizontal directional drilling*. Horizontal directional drilling (HDD) consists of a rig that makes a pilot bore by pushing a curing or drilling head that is steered and guided from the surface. Drilling fluid is pumped through the drill/push rods and displaces the cut soil. When the pilot bore is completed, pulling back a reamer enlarges the hole. Progressively larger back-reamers are used until the hole is large enough to pull in the pipe. HDD is suitable for installing pipes under waterways, major highways and other obstacles.

(ii) Open trench replacement

Open-trench replacement is the most commonly used method for replacement of water mains and sewers. This technique involves placing new pipe in a trench cut along or near the path of the existing pipe. This approach is cost intensive and problems of working within developed areas where pipes may be beneath streets, sidewalks, customer landscapes, utility poles are inevitable. There are two basic types of open trench replacement: (a) conventional, and (b) narrow. The conventional open-trench method uses the same approach as that used to place new pipe. In using the narrow-trench replacement method, the trench width is kept to the absolute minimum excavation width possible. It is primarily used for installing polyethylene pipes.

Table 8.1. Summary of typical rehabilitation/replacement methods

Method	**Suitable pipe size**	**Common materials used in rehabilitation or replacement**
Cement mortar lining	100-1500	Cement-sand
Epoxy lining[a]	100-300	Epoxy resin
Sliplining	100-2500	HDPE, PVC, fibreglass reinforced polyester
Cured-in-place pipe	150-1300	Polyester resins
Fold and form pipe	200-450	HDPE, PVC
Close-fit pipe	50-1000	PE, PVC
Pipe bursting	100-1000	HDPE, PVC. ductile iron
Microtunneling	300-3600	HDPE, PVC, concrete, steel, fibreglass
Horizontal directional drilling	50-1500	HDPE, PVC. steel, copper, ductile and cast iron

Note: HDPE = high density polyethylene; PVC = polyvinyl chloride; PE = polyethylene
Source: Adapted from Selvakumar et al. (2002)

8.2 Approach to costing of various options of rehabilitation and alternative strategies

8.2.1 General background

When considering the price of a wastewater system development, upgrading or replacement project, required expenditure is broadly divided into three cost groups:

- *Capital cost* – initial cost of constructing the scheme
- *Operation and maintenance (or revenue costs)* – representing the costs incurred in running the scheme (e.g. power, labour, materials and routine maintenance)
- *Refurbishment costs* – representing costs involved in a major programme renovation (major refurbishment after 15-20 years is rather common)

According to contemporary trends in the economics of various options for the development of municipal wastewater systems, infrastructural asset management principles should apply.

Table 8.2. Levels of service versus cost of service

Requirements for level of service	**Required inputs on infrastructure asset**	**Cost of service and asset management**
• Reliability • Quality • Quantity • Safety • Low risk • Security	• Ways of creation • Procedures in operation • Means of maintenance • Performance monitoring • Risk assessment methods • Audit frequency • Renewal strategy	• Original costs • Cost of operations • Cost of maintenance • Cost of administration management • Cost of exposure to risk • Cost of replacement / rehabilitation / disposal

8.2.2 Costing of sewer flow monitoring and analysis

As stated previously, the first steps in determining a maintenance programme are to check on the accuracy and completeness of existing records of the system, and then initiate a survey on the parts of the system which are assumed to be most affected. To determine the extent of infiltration in waterborne sewers, field inspections and flow monitoring are essential in decision-making on the sewer repair, renovation or replacement.

(i) Common objectives for flow monitoring

There are several reasons for monitoring the flows in a wastewater collection system, such as determining total systems flows, customer billing, identification of capacity problems, monitoring system performance for operation and maintenance, detection and quantification of bypasses or overflows, measurement of the PWWF, inflow/infiltration events, exfiltration and to calibrate flow models.

(ii) Flow monitoring programme

A well prepared and executed field flow monitoring programme will enable a WSA/WSP to isolate areas or specific reaches of a wastewater collection system which has excessive inflows and infiltration and/or exfiltration.

(iii) Field flow metering equipment

There are three major methods where gravity flow metering can be deployed:

- *Critical-depth metering,* which may include flumes and weirs
- *Area-velocity metering,* which offers a choice of different technologies for depth (e.g. air bubblers, pressure transducers, etc.) and velocity measurements (e.g. electromagnetic sensors and acoustic devices).
- *Combination of flumes and electromagnetic sensors*

Modern flow monitoring equipment has a data logging function, which allows the operator to collect "real time" flow records over an extended period of time. Such data can then be correlated with rainfall events to determine the inflows into a system.

The accuracy and reliability of different monitoring devices together with data transmission and energy provision are criteria for choosing suitable monitoring technology.

(iv) Close-circuit television (CCTV)

CCTV inspections for waterborne sewers are popular as they can be carried out quickly with minimal disruption and less of a hazard for people to enter a sewer. This way of inspection is used to locate and define the cause of a known condition or defect and enables inspectors to prepare a plan of action. Known rates of inspection are between 400 to 800 m/day in pipes from 100 to 1400 mm in diameter. The usual method is a propulsion camera winched between sewer manholes.

8.2.3 Additional storage treatment optimization

If additional WWTP storage is considered, it is important to optimize the storage volume in conjunction with the treatment rate in order to obtain the least-cost storage treatment system. The optimization strategy to adopt is recommended as follows:

- *make operational (i.e. low cost) in line improvements to the collection system before enlarging the treatment plant*
- *consider the treatment by settling in overflowing storage tanks*
- *reassess design capacity of the WWTP by choosing the point of diminishing returns in vie of pollution control versus operating and maintenance costs*
- *size the storage /treatment system according to the break-even point between the amount of storage versus the treatment capacity*

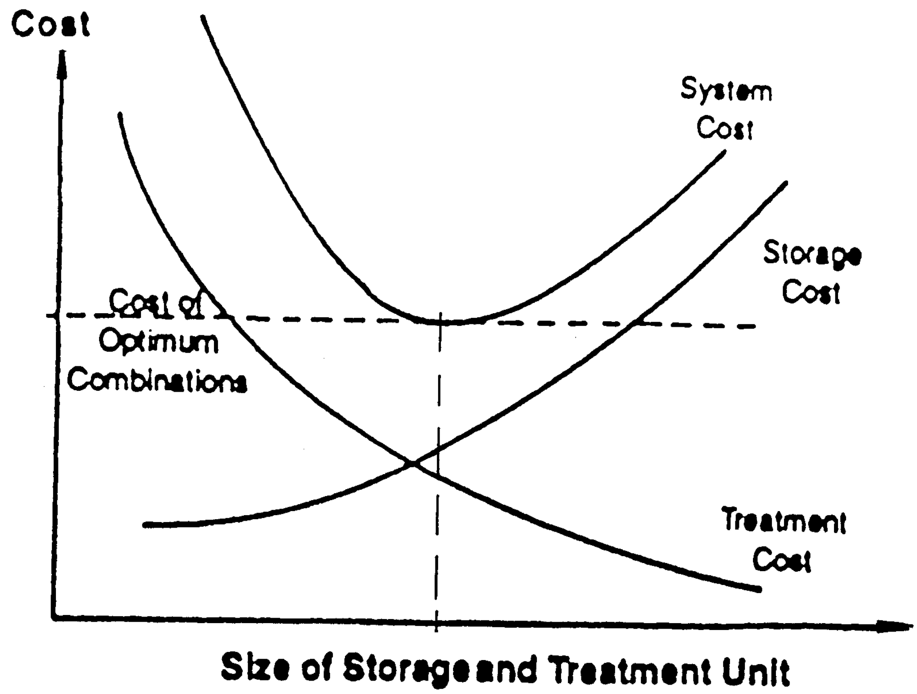


Figure 8.1. Optimising size of storage and treatment unit in a wastewater system

8.3 Costing procedures in rehabilitation/replacement of waterborne sewer assets

8.3.1 Determining relative costs of rehabilitation

To make a decision on rehabilitation of a waterborne sewer, comparisons of the cost of rehabilitation now with the costs of absorbing the consequences of a sewer collapse in the future should be made. Predicting what will happen if rehabilitation is not carried out may be applied as follows:

$$\text{Cost of Rehabilitation } (C_{rehab}) < \frac{C_{collapse}}{\left(1+\frac{r}{100}\right)^{t}} \tag{8.1}$$

Where: $C_{collapse}$ = estimated cost of collapse (i.e. disruption and sewer replacement
r = discount rate (say 6% p.a.)
t = number of years before collapse is predicted to take place

It should be noted that although all inputs to formula (8.1) can be estimated, the exact time of failure and cost of total collapse will be difficult to predict.

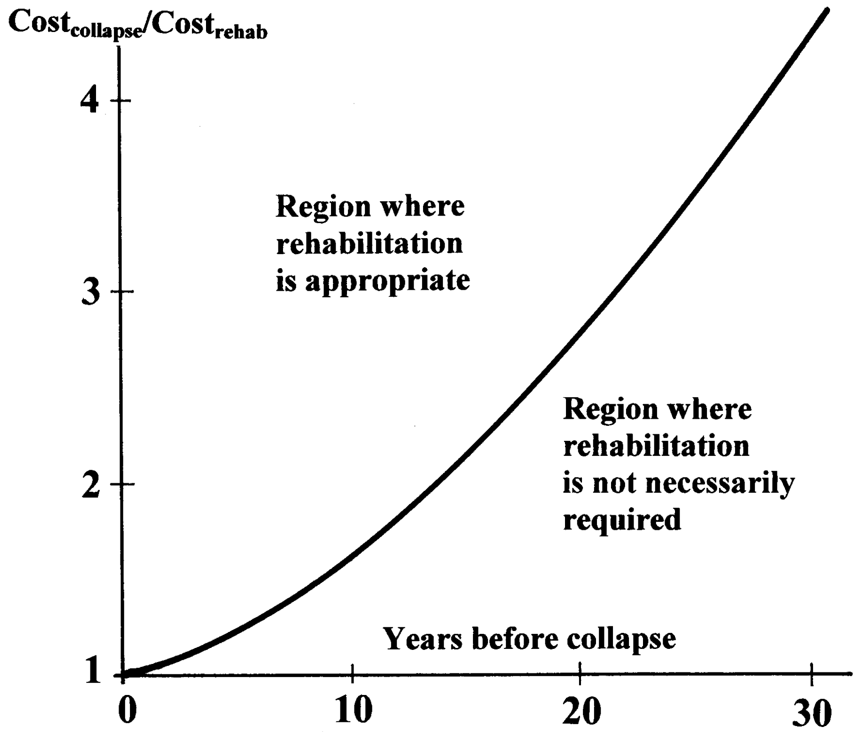


Figure 8.2. Relative costs of rehabilitation and collapse at 5% discount rate (after Butler and Davis, 2000)

8.3.2 Benefit-cost analysis

(i) Ratio of net benefits to cost (B/C ratio)

The assumption that costs generated and benefits derived from rehabilitation/ replacement of a sewer pipe can be assigned a monetary value allows for a benefit-cost analysis to be conducted.

The technique commonly applied is the ratio of net benefits to costs (B/C ratio). This technique measures the ratio of the present value of future benefits (i.e. at a given discount rate) to the present value of future costs (i.e. discounted at the same rate). The B/C ratio measures the economic efficiency of maximum contribution to the proposed project.

$$\frac{B}{C} = \left[\sum_{t=o}^{T} \frac{B_t}{(1+r)^t} \right] = \left[\sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \right] \tag{8.2}$$

Where:
- t = an index of time (usually in years)
- T = time horizon, the last period for planning
- B_t = Total benefits accruing in period t (Rand)
- C_t = Total costs accruing in period t (Rand)
- r = selected discount rate

If the B/C ratio is >1, then the alternative (project) is economically justified. If B/C $\leq$1 then the alternative (project) should be rejected or revised.

(ii) Quantification of economic costs and benefits

The benefits derived from any given alternative (project) are usually more difficult and complicated to value than the costs. All costs and benefits must be valued at economic rates. The gross benefits of new or refurbished water services projects are commonly derived from two key components:

- *increased knowledge about infrastructural assets, and*
- *gradual introduction and upgrading of formal and advanced practices, procedures and systems*

Some of the costs and benefits related to project development will be immediately apparent, others will be more or less unavoidable trade-offs (or externalities). Externalities are costs of benefits generally considered external to direct economic evaluation as they do not benefit or cost the investor directly. They are not easy to quantify in monetary terms and are not commonly included in the calculation of present worth.

The key areas from where benefits for projects can be derived and quantified are related primarily to well-managed infrastructural assets.

(a) *Asset life extension* – based on required levels of service, life span horizon and associated costs.
(b) *Optimized rehabilitation decisions* – knowing rates of decay, current conditions and replacement value of assets.
(c) *Reduced risk control* – knowing impacts of failure and associated risks.
(d) *Appropriate resources management* – cost effective (optimal) maintenance, operations and rehabilitation programmes leading to reduced capital and recurrent costs.
(e) *Improved managerial decision-making and planning* – based on the lowest life cycle costs when considering combinations of conventional and advanced technology.
(f) *Planned preventative maintenance* – introducing a culture of long-term planning and developing strategic plans for rehabilitation, renewal and/or replacement.
(g) *Improved customer service* – reducing exposure to litigation, driving condition programmes, improving customer relations, greater administrative efficiency.

In addition to the above-listed key benefit areas, it is beneficial for WSPs/WSAs to know their external benefits which might amount to the following:

- *minimizing a service gap between supply and demand*
- *minimizing costs*
- *minimizing negative environmental consequences, and*
- *minimizing the economic effect on the regional economy from investment*

The short and long term benefits resulting from implementation of new or rehabilitated projects should be recognized. WSA/WSPs with mature (or old) infrastructural assets can gain some 50 percent in value of medium to long-term benefits if they implement the benefit gain approaches listed in (b), (c) and (e) above.

8.3.3 Benefit-cost-risk analysis

(i) Purpose of benefit-cost-risk analysis

The purpose of benefit-cost-risk analysis is to quantify the costs and benefits of various rehabilitation alternatives with regard to rehabilitation (or sustained integrity) of the overall system. The key objective is to determine the losses associated with various failures and the optimal level of system reliability.

(ii) System engineering and economic reliability

- *Engineering reliability* – Engineering reliability of a wastewater system is defined as the reliability specified implicitly or explicitly through standards to which the system must be designed to meet those standards at minimum cost.
- Economic reliability – Economic reliability is defined according to the standards where the system reliability is selected for the system to minimise the total socio-environmental cost. The optimal level of reliability is the value of benefits minus costs (B – C) and depends on the unique situation of each system and the alternative selected.

8.3.4 Quantitative risk analysis

(i) Risk assessment for a sewer system

A risk can be defined as the product of the likelihood of an event and the consequences of that event. The consequences are rather difficult to define and are most commonly attached to the risk to human life. However, to determine the risk associated with the poor condition and performance of a wastewater system would be fare more complex. To assess the risk for such a system, it is necessary to determine individual components of risk and combine them together to obtain the overall risk situation. A risk assessment is either carried out as a quantitative or qualitative analysis.

(ii) Estimation of probability and frequency in qualitative risk analysis

It is essential to consider the combination of events in the assessment of the risk of a system or its vital components. The events are represented by a combination of probabilities and frequencies. This is commonly explained as follows: Two possible events A and B are considered causing C and generating the following probabilities and frequencies:

Probabilities: $P_{A \text{ or } B} = P_A + P_B - P_A P_B$ (8.3)

$P_{A \text{ or } B} = P_A + P_B$, if P_A and P_B are smaller than $P_{A \text{ and } B} = P_A * P_B$ (8.4)

Frequencies: $F_{A \text{ or } B} = F_A + F_B$ (8.5)

$F_{A \text{ or } B} = F_A * F_B * (\tau_A + \tau_B)$ (8.6)

Where: τ_A and τ_B are the duration of the events A and B

Units of frequency are expressed as occasion/year (occ/yr). Frequencies can be multiplied by probabilities.

(iii) Quantifying the risk costs of sewer pipe failure

The cost of risk to the WSA or WSP needs to be assessed for all failures ranging from those needing minor maintenance to major catastrophic structural failures. The reduction or avoidance of risk needs to be quantified as a benefit to the WSA/WSP.

Current Risk Cost (Benefit) = Probability of Failure within next 12 months * cost of the consequence of Failure (8.7)

This approach can assist in the identification of the components that might have a high probability of failure (or highest risk to the WSA/WSP). typical consequences of failure can be listed as follows:

- *Effect on public health (potential loss of life*
- *Damage to private property*
- *Effect on business capacity*
- *Effect on essential services*
- *Disruption to traffic or public transport*
- *Inconvenience to residents (ratepayers) due to repair cost (e.g. digging, access, etc.)*
- *Availability of spare materials*
- *Cost of providing the service during failure*
- *Damage to the environment*
- *Actual cost of the repair*
- *Public image/public relations loss*

(iv) Single/multi-failure state and costs

It should be noted that in some systems, a multitude of failures can take place. The timing of a failure is likely to affect the cost. The cost of failure excludes both the cost of the lost product and the cost of repair (replacement). For fully repairable single failure state system, the following applies:

$$C_{failure} = [(CR + CLP) * u] * SOT \quad (8.8)$$

Where: CR = repair cost per hour
CLP = cost of lost production per hour
u = system unavailability (probability of failure)
SOT = system operating hours per annum (i.e. typically 8760 hours)

The following applies for a fully reparable, multi-failure state system:

$$C_{failure} = [(CR_1 + CLP_1) * U_1 + (CR_2 + CLP_2) * U_2] * SOT \quad (8.9)$$

Where: CR_1, CR_2 = repair cost per hour for failure state 1 and state 2 respectively
CLP_1, CLP_2 = cost of lost production per hour for failure state 1 and state 2 resp.
U_1, U_2 = system unavailability for failure state 1 and state 2 respectively
SOT = system operating hours per annum (full year = 8760 hours)

The total operating cost (C_{OP}) is the sum of the failure costs, the engineering charge costs (C_{EC}), the fixed maintenance costs (C_{FM}) and the consumable costs (C_{CC}). Since the cost of failure calculation accounts for operating hours in one year, that cost must be multiplied by the anticipated lifetime of the system:

$C_{OP} = [(C_{EC} + C_{FM} + C_{CC} * C_{failure}) * \text{years of life}]$ (8.10)

8.4 Requirements and assumptions applied in economic evaluation

8.4.1 Life-cycle costing in economic analysis

(i) Life-cycle cost components

Life-cycle costing is a dynamic approach which deals with changing economic factors by accommodating year-by-year-changes in price inflation, price changes, regulatory requirements and variations in replacement and O & M costs. The method allows for a conversion of changing future costs and benefits to a common time basis by means of a lump sum present worth method. In this way, the total cost of rehabilitation or replacement over the full life span can be determined.

Table 8.3. Life-cycle cost components

LCC component	**Procedure**	**Incremental annual costs**
Interest/opportunity cost	Total capital cost at annual interest rate (%)	C_1
Depreciation of system component	Depreciate costs of components over estimated life span	C_2
Operating costs	Based on labour, plant, materials and energy requirements at unit life	C_3
Maintenance costs	Estimated at proportional percentage of the total capital cost	C_4
Rehabilitation costs	Assumed to be funded from depreciation provision	As per C_2
Decommission/demolition	Estimated value at the end of the life span	C_5
Annual Life Cycle Cost (LCC)		$\mathbf{C_{LCC}}$

(ii) Planned and unplanned life-cycle costs

The estimate of planned and unplanned life-cycle costs provides economic insight into the various cost components of a system and identifies the specific information required to make such estimates into the future.

- *Planned life-cycle costs* – include expenditures and user costs related to the procurement and maintenance phases with regard to the life-span of a system (i.e. capital costs and maintenance.
- *Unplanned life-cycle costs* – cost related to damages which might occur to a system's component(s) primarily due to natural or man-caused hazards.

The total life-cycle cost (TLCC) including planned and unplanned costs is as follows:

$TLCC = CPO + CPU + CUO + CUC$ (8.11)

Where: CPO = planned costs incurred to the system owner/developer
CUO = unplanned costs induced upon the owner/developer of a system
CPU and CUC = costs associated with planned and unplanned costs respectively

(iii) Example of life-cycle costing for sewer pumping plant

The following procedure is a recommended guideline in life cycle costing of a sewer pumping plant.

$LCC = [C_{ic} + C_{in} + C_e + C_o + C_m + C_s + C_{env} + C_d]$ (8.12)

Where: C_{ic} = initial costs, purchase price (pump, system, pipe, auxiliary services)
C_{in} = installation and commissioning cost (including training)
C_e = energy costs (predicted cost for system operation, including pump driver, control and any auxiliary services)
C_o = operating cost (labour cost of normal system supervisor)
C_m = maintenance and repair cost (routine and predicted repairs)
C_s = down time and lost of production costs
C_{env} = environmental cost (contamination from pumped liquid and auxiliary equipment)
C_d = decommissioning and disposal cost (including restoration of the local environment and disposal of auxiliary services)

Refer to Appendix E1 and E2 illustrate the relative costs of various sewage pumping plans and how the distribution costs may vary with pump size and utilization.

(iv) Assumptions for a comparative economic evaluation

Illustrative values required for a comparative economic evaluation are listed below:

- *Cost of capital* — *12% per annum*
- *Rate of inflation* — *7% per annum*
- *Capital repayment period*
    - *Civil work* — *30 years*
    - *Electrical/mechanical* — *15 years*
- *Composition of cost for new works*
    - *Civil work* — *60%*
    - *Mechanical* — *32,5%*
    - *Electrical and instrumentation* — *7,5%*
- *Planning horizon (illustrative)* — *21 years*
- *Economy of scale functions for* — *1994*
- *Construction cost escalation* — *8% per annum*
- *Repayment period for capital project of less than R2m* — *5 years*
- *Repayment period for capital project of less than R10m* — *10 years*

This procedure enables the WSAs/WSPs to select the rehabilitation/replacement alternative which will be affordable and suitable to their circumstances.

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

9.1 Key conclusions manifested from the research

The wastewater generated within urban areas is water that has been supplied and used to support life, maintain a standard of living and sustain industrial and commercial activities. After use, if not drained and treated properly, it will most certainly cause pollution and create health risks (e.g. cholera) as well as serious degradation of the natural environment.

Extraneous flows as stormwater inflow and groundwater infiltration, which take place mainly due to ageing in municipal waterborne sewers, are problems which increase the likelihood of the above-mentioned risks. The elimination/mitigation of extraneous flows in sewers are obvious engineering and managerial tasks. The decision to upgrade, rehabilitate or replace urban waterborne sewers which are subjected to excessive extraneous flows depends on adequate and continuously updated databases based nowadays on GIS representation which will lead to effective decision-making in mitigating/eliminating extraneous flows.

The key conclusion this project identified was low awareness about I/I problems and remedial/rehabilitation techniques by most South African WSAs/WSPs. Due to the magnitude and complexities inherent to municipal waterborne sewer systems, only a few WSAs/WSPs can make an educated decision on developing a new or upgrading/rehabilitating an existing system (or its key components). They lack mainly field and modeled data, particularly on inflow/infiltration/ exfiltration events and their consequences. Typical values generated by this research from literature and particularly from field monitoring of sewer flows in urban areas around South Africa are illustrated in Table 9.1.

Table 9.1. Typical values of municipal sewer flow components

Residential wastewater outflow (ℓ/min/household)	**Leakage from households (ℓ/min/household)**	**Groundwater infiltration (all pipe types) (ℓ/min/m-dia/m-pipe)**
0,01 to 1,20	0,06 to 0,20	0,01 to 0,50
SABS 1200: Permissible wastewater loss from new sewers (all types)		0,01
Red Book (2003): Design criteria for all sewer pipes		Allowance of 15% of ADWF to add

With regard to the range of values on groundwater infiltration as illustrated above, it is recommended to consider that groundwater infiltration exceeding 0,10 ℓ/min/m-dia/m-pipe is excessive for all sewer pipe materials.

Another major observation from this project is that the maintenance strategy of most WSA/WSPs in South Africa is essentially reactive maintenance, where problems are dealt with on a corrective basis as they arise. Consequently, municipal wastewater system maintenance budgets are commonly low and are based on the previous year's financial expenditure mainly from clogging and collapsing sewers. It has been established from a survey that stoppages and clogging of sewers in South Africa per are about ten times higher than the international average, averaging to 3,3 blockages/km/sewer pipe/per annum.

It was also established by this project that the costs associated with maintaining or expanding existing and/or developing new urban wastewater infrastructure appear to be large, but well invested if allocated on a regular basis. Because there is not yet enough pressure applied from the wastewater services end users to municipal managers about the economics of alternative solutions, conventional methods therefore prevail and benefits are not highlighted in the cost analysis.

Water infiltration in sewer pipelines is common and should be included in the peak design flow. A norm of 15% of the dry weather flow allowance for extraneous flows is a generally acceptable standard. Flows exceeding this norm will result in pipe capacity problems and an unnecessary increase in sewer discharge volumes and treatment costs. A reduction in infiltration/inflow rates will not only save on sewerage treatment costs, but may defer capital expenditure for the upsizing of collection sewer pipelines and wastewater treatment plant. The decision to solve or ignore an infiltration problem should therefore be based on a benefit-cost analysis.

The municipalities surveyed under this project represent a population of 4,593 million urbanized people, or about 15% of the total population. The per capita daily water use ranges between 100 and 675ℓ/capita/day within a given sample. The average ratio of volume of wastewater treated to volume of water supplied in surveyed WSAs amounted to 0,62. This contrasts with the general impression that the return flows from large urban areas are much larger. This ratio reflects the overall composition and diversity in urban water supply and sanitation in South Africa.

It may be further derived from the information available that the average maintenance budget per kilometre of municipal sewers installed amounted to R10390 in the financial year 2001/02.

9.2 Choice of relevant rehabilitation/replacement methods

Based on the identified and rated problem(s) present in waterborne sewer collection systems, there are several rehabilitation/replacement methods available for WSAs/WSPs to chose from.

Table 9.2. Summary of rehabilitation/replacement methods

Method	Types of problem
Excavation and replacement or duplication	Misaligned pipes, additional capacity needed, avoiding reduction in capacity, damaged pipes
Grouting, lining, sliplining, rejointing	Leaking joints, high infiltration/exfiltration, circumferential or radial cracks, roots, corrosion
Cured-in-place lining	Non-circular pipe, mild deterioration, misaligned pipes and bends, corrosion by waste
Insertion, speciality concrete	Specific structural problems

It is recommended that all possible solutions to problems are identified and costs of the most feasible alternatives should be determined and compared. The selection of the relevant method and materials depends on an understanding of the specific problem being prevented or corrected.

9.3 Recommended overall rehabilitation/replacement programme

Next to regular maintenance procedures, Water Services Authorities/Providers should adopt and develop an overall waterborne sanitation system rehabilitation/replacement programme as summarized in Table 9.3 below.

Table 9.3: Summary on sewer system rehabilitation/replacement programme

Assessment phase	Key objectives	Key activities
Phase 1: Planning and investigation, and data gathering	Preliminary determination of extent and severity of problem	• Assemble relevant data sources and establish GIS database • Outline networks (or zones) with problems • Identify locations of monitoring points • Propose field monitoring methods • Determine staff training requirements
Phase 2: System assessment/ inspection programmes	Establish all necessary inspection programmes and costs resulting from monitoring	• Inspect and determine extraneous flows • Re-evaluate hydraulic performance and parameters • Inspect and determine structural condition of relevant components
Phase 3: Action plan for rehabilitation/ replacement	Propose, design and cost relevant rehabilitation/ replacement and preventative maintenance measures	• Propose remedial measures • Propose rehabilitation measures • Propose replacement measures • Tender for procurement
Phase 4: Implementation of Action Plan	Organise and supervise procurement and assurance of quality control	• Determine implementation schedule • Process and evaluate all factors which might influence procurement • Establish adequate quality testing • Test and monitor post procurement performance

9.4 Evaluation of operation integrity of a wastewater system

To evaluate the operational integrity of a wastewater system is not an easy task and it is necessary to compile and analyse hydraulic data, asset condition and costing information in order for the correct decision to be made. It is recommended that information on a wastewater system be scrutinized according to four sets of key criteria:

- compliance with legal/environmental safety standards,
- current loadings versus designated capacities,
- risk of potential failure (history of breaks and outages),
- status of expenditure on revenue costs.

The layout of a programme illustrated in Appendix E will enable WSAs/WSPs to evaluate the operational integrity of a waterborne sewer system

9.5 Guidelines compiled from this research

This research project investigated, evaluated and selected techniques and methodology in estimating the impacts of I/I/E events on municipal sanitation services. Approaches and methods on how to eliminate/mitigate I/I/E problems are proposed in this report and the Guidelines WRC TT 239/05 which emanated from this research. Both this report and the Guidelines have been drawn up to help WSAs/WSPs in the planning, design, construction, operation and rehabilitation of municipal waterborne sanitation systems.

Due to the magnitude and complexity of the attention required to research, design, construction and management of wastewater sanitation systems, all relevant stakeholders must share responsibility for development and management of these systems. The application of new local and international technologies must be promoted by the WSAs and WSPs through adopting the Guidelines in capacity building programmes.

9.6 Improving upon traditional methods

To sustain reliable municipal wastewater infrastructure and required services to customers, new and improved solutions to existing and emerging problems will have to be researched. Further research needs that have manifested from this study related to the following key areas as illustrated as follows:

Table 9.4. Further research needs for waterborne sewer systems

Assessment objective / sphere	**Area of possible research**
Flow monitoring	• Evaluate new flow monitoring techniques • Investigate miniaturized sensors and wireless data transmission
Structural integrity	• Investigate remote sensing and monitoring systems • Standardize inspection procedures and techniques
O & M	• Assess effectiveness of O & M programmes • Develop predictive methods for assessment of sewer system
Rehabilitation	• Evaluate improved repair and replacement techniques • Evaluate performance trade-offs between rehabilitation and replacement

As a final conclusion from this investigation, it is recommended that formal methods are developed to determine how much maintenance is needed to achieve a specific level of performance of a wastewater sewer system in order to enable WSAs/WSPs to justify the necessary maintenance investments.

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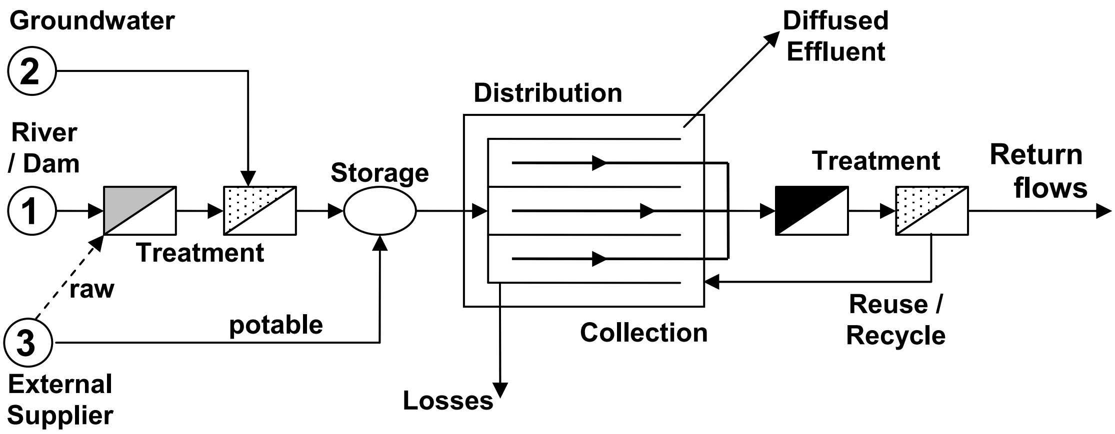
APPENDICES

APPENDIX A. Questionnaire on water services system return flows

RESEARCH PROJECT: WATER RESEARCH COMMISSION PROJECT NO. K5/1386
"Impact of Stormwater and Groundwater Ingress on municipal sanitation services"

A. <u>Questionnaire on water services system return flows</u>
(Please fill in either estimated or measured values)

1. For the benefit of adequate water resources management, it is of great importance that the return flows are accurately monitored with regard to water quantity and quality, sustained availability and potential for direct or indirect reuse. The return flows from a water services system can be evaluated according to the conceptual model illustrated below:



2. Total water supply from all sources: ________________________ (m^3/a or million m^3/a)
3. Proportional breakdown: Ground water __________ (%); Surface water: _________ (%)
   External supply: ____________ (%); Other: __________________________________(%)
4. Water exports from a system: ______________________ m^3/a, or _______ (% of TWS)
5. Stormwater ingress: ____ m^3/a
6. Groundwater ingress: ____ m^3/a
7. Exfiltration from sewers, if any: ____ m^3/a
8. Bulk water losses: _______________________________ m^3/a; or ______ (% of TWS)
9. Unaccounted-for water: ___________________________ m^3/a; or _______ (% of TWS)
10. Effluent diffused locally: __________________________ m^3/a; or _______ (% of TWS)
11. Wastewater influent: ______________________________ m^3/a, or _______ (% of TWS)
12. Wastewater treatment losses: _____________________ m^3/a; or _______ (% of TWS)
13. Reused/recycled treated wastewater: _______________ m^3/a; or _______ (% of TWS)
14. Measured releases of treated wastewater: ___________ m^3/a; or _______ (% of TWS)
15. Quantity and means of sludge disposal: _______________________________________

Net return flows = Total water supply (TWS) less consumptive water use and water exports and reuse
TWS = Total water supply from all available sources

APPENDIX B. Questionnaire on stormwater and groundwater (S & G) ingress

RESEARCH PROJECT: WATER RESEARCH COMMISSION PROJECT NO. K5/1386
"Impact of Stormwater and Groundwater Ingress on municipal sanitation services"

B. Questionnaire on stormwater and groundwater (S & G) ingress
(Please fill or tick relevant answers)

1. Name and address of Water Services Authority/Provider administering sewer system:
____
____
2. Rainfall region:
☐ Summer ☐ Late summer ☐ Very late summer ☐ Winter ☐ All year round
3. Average rainfall (mm) ______________________________
4. Sewer system details:
Gravity mains: ____________ (km); Raising mains ____________ (km);
Number of pumpstations: ____________; Pumping capacity ____________ (kW);
Reticulated area: ____________ (ha, km^2); Maximum flow ____________ (m^3/s);
Diameter(s) ______________________; Typical slope ______________________ ;
Pipe materials ______________________________
5. Status of sewer network:
☐ Excellent; ☐ Good; ☐ Satisfactory; ☐ Unsatisfactory.
6. Predominant land use of area drained (zone type): ______________________
7. Records of infiltration /inflow of groundwater/stormwater into the sewer system:
☐ None; ☐ Groundwater; ☐ Stormwater; ☐ Both.
8. Extent of infiltration/exfiltration and inflow (surges in sewers) problems:
Av, dry period flow: __________ (m^3/s); Av. wet period flow: ____________ (m^3/s);
Infiltration rate: ______________________ (ℓ/min/m dia/m length)
Exfiltration rate: ______________________ (ℓ/min/m dia/m length)
Stormwater ingress: __________________________ (mm/h)
9. Design criteria adopted:
☐ Peak Dry Weather Flow (PDWF): ______________________________
☐ Pipe reserve for SI as flow area (_____________ %)
10. Specific arrangements to accommodate SI: ______________________________
____
11. Ways and means of monitoring sewer flows: ______________________________
____
12. Most common causes of S & G ingress or exfiltration in your area: ______________
____
____
13. Blockage/km pipe/year average: ______________________________
14. Sewer annual maintenance budget: ______________________________
15. Preferred methods in rehabilitation of sewers: ______________________________
____
16. Is there a reticulation maintenance and management plan?
☐ Yes; ☐ No. If No, why?______________________________
____
____
17. Use of modelling and models for sewer management: ______________________
____
____

18. Cost of sewer surcharging, treatment processes: ________________________________
19. Cost of alternative strategies (e.g. stormwater diversion): _________________________
20. Comments on guidelines for decision and management of sewer networks: _________
    ____
    ____

APPENDIX C: Daily rainfall data for Station no. 0476403-2 at Boksburg, East Rand, Gauteng Province, (latitude S26 21’70”, longitude E28 23’30”, altitude 1631 masl)

Day of month	2002						2003					
	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
01												
02												
03												
04			5		22							1
05									12			
06												9
07							38					3
08												
09												
10								28				
11						22						
12												
13												
14								6				
15												
16								3				
17						13	11	4				
18						16						
19						5	17			3		
20							1	48	38			
21												
22				30			22		1			
23				7		16	13					
24						1			17			
25						6		5	3			
26						13						
27							12					
28								5				
28				4	13							
30				5								
31												
Monthly total	Nil	Nil	5	46	35	92	138	99	71	3	Nil	13
Annual total	1221 mm Jan – Dec 2002						529 mm Jan – Dec 2003					

Source: SA Weather Bureau (www.weathersa.co.za)

Note: The shaded areas indicate time periods of measurements taken on the Boksburg sewer outfall

APPENDIX D: Application of URFA model in Ekurhuleni MM

Evaluation of impact of extraneous flows in the Ekurhuleni MM

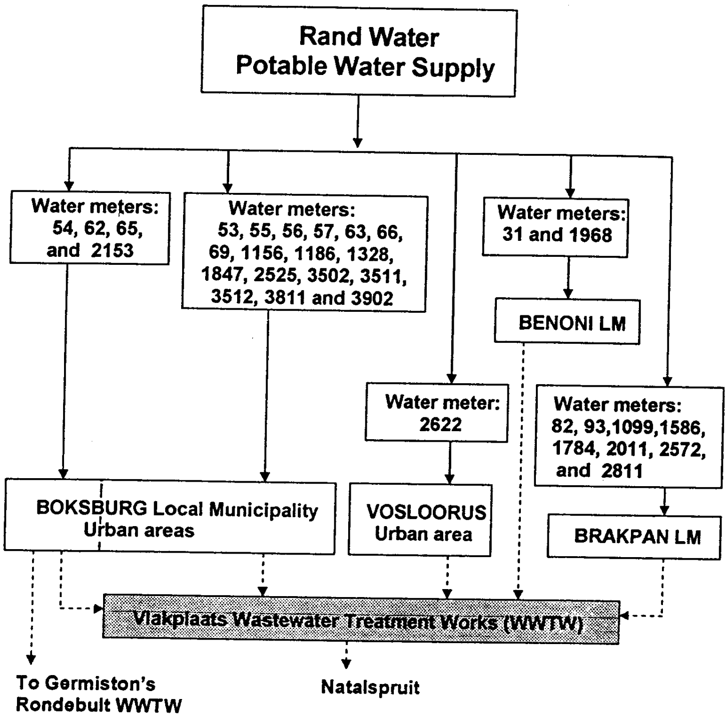
(i) Southern drainage basin master plan

Since 1991, developments in urban sanitation south of the Witwatersrand Ridge in Gauteng have been guided by the strategic plan prepared for the Southern Ridge Basin. A consortium of consulting engineers evaluated the needs for new outfall sewers and necessary extensions to existing WWTWs. The first phase of the plan was implemented a few years ago, primarily in the Johannesburg Metropolitan Municipality juridical area. The responsibility of meeting the required quality of return flows to the Vaal River generated in the Gauteng MC south of the Witwatersrand Ridge is shared between Johannesburg Water (Pty) Ltd and ERWAT, releasing on average 930 and 610 Mℓ of treated effluent per day respectively.

(ii) Water service provision in the Ekurhuleni Metropolitan Municipality

Ekurhuleni MM is situated in the eastern area of the Gauteng Province. Wastewater district DD6 (Natalspruit/Rietspruit drainage) straddles the urban areas of the Ekurhuleni MM namely Alberton, Boksburg, Brakpan and Germiston and the Lesedi Municipality (formerly Heidelberg). All the urban areas in the DD6 district are supplied with potable water from the Rand Water. Other water sources (e.g. boreholes) have a negligible share of the total water supply. There are four WWTPs situated in this district, namely Dekema (30 Mℓ/d), Rondebult (30 Mℓ/d), Vlakplaats (83 Mℓ/d) and Waterval (100 Mℓ/d) with a total current processing capacity of 243 Mℓ/d. The urban wastewater generated in. Boksburg, Brakpan and a small portion of Benoni drains into the Vlakplaats WWTP. The treated effluent is released into the Natalspruit/Rietspruit ecosystems.

Figure D.1. Model of potable water input to urban areas drained to the Vlakplaats WWTP



(iii) Vlakplaats WWTP sewer reticulation network

- *Sewer network layout.* The wastewater generated primarily from approximately 52 000 formerly developed stands with full waterborne sanitation in the Boksburg and Brakpan urban areas is collected by two main gravity outfalls and treated at the Vlakplaats WWTP situated at Vosloorus. The responsibility for water services delivery including water supply and sanitation is the Ekurhuleni Metropolitan Council. However, the licensing and quality control of treated effluent discharges lies with ERWAT.
- *Boksburg branch of Vlakplaats sewer network.* This branch has about 690 km of sewers of various sizes, the largest being 1050 mm in diameter. There are 19 pumpstations and about 11 000 manholes situated in the subsystem. About 30 000 residential properties are serviced by this branch. Ten informal settlements are serviced by chemical toilets and VIPs as well as 23 septic and conservancy tanks which indirectly contribute a few times per year to the Vlakplaats WWTP by means of vacuum tank trucks loads.

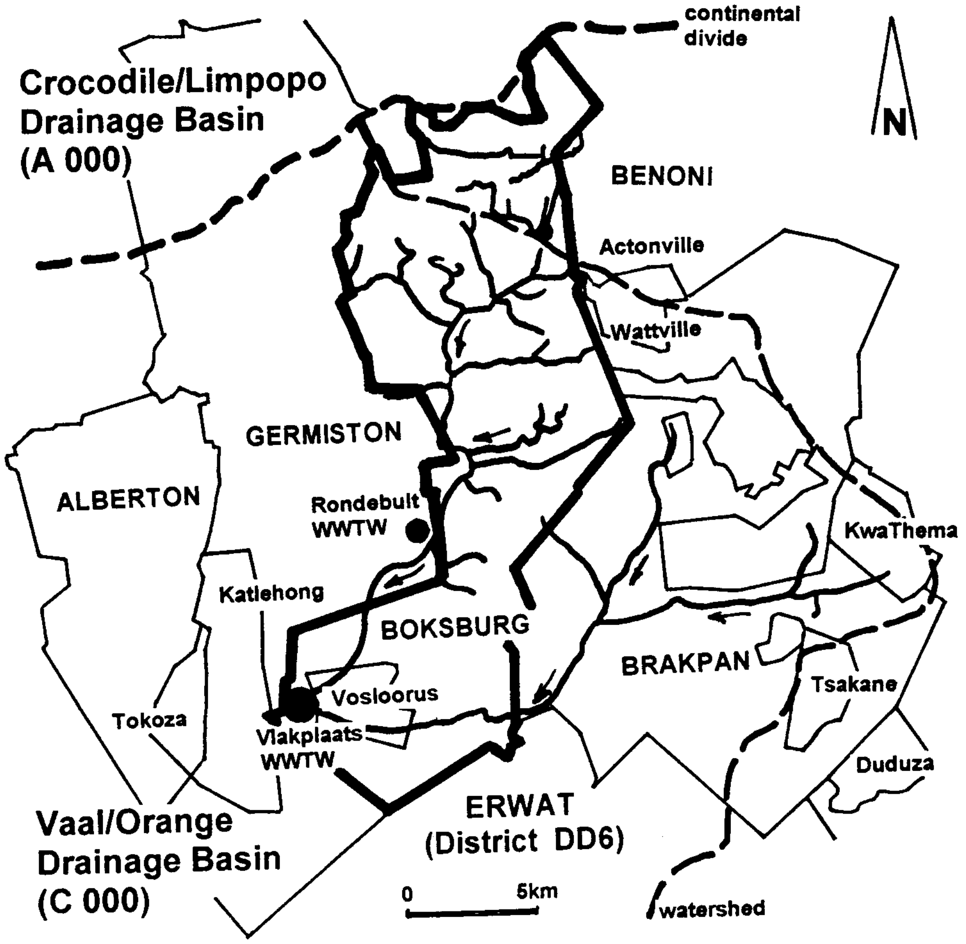


Figure D.2. Layout of Vlakplaats WWTP outfall subsystem

About 15 percent of sewer piping is older than the assumed economic design life of 50 years. Some 750 metres of sewers is repaired or replaced annually under a sewer rehabilitation programme. GMKS (1999) established that the reticulation subsystem is rather sensitive to the operating rules set out for the 17 wet well pumpstations interconnected with the gravity sewers. A major problem identified in the Boksburg branch is groundwater infiltration into the sewer system, especially in areas close to existing dams and the sewers situated in low lying areas (i.e. below the groundwater table). It has been established from previous studies that on average the outfall experiences an inflow/infiltration rate of 400 ℓ/s in the wet season and about 230 ℓ/s during the dry season.

When frequent heavy rain occurs in the catchment in the summer season, stormwater inflows will manifest in visible surges at numerous manholes. Both storm and groundwater I/I events can reach dramatic peak flows of up to 3 times of ADWF, thereby disturbing the sewage treatment processes at the Vlakplaats WWTP.

(iv) Volumetric water balance at Vlakplaats WWTP

- *The method adopted*. A bulk volumetric water balance for a dedicated municipal water services system can be performed by the full cycle water balance methodology.

The Vlakplaats WWTP sewer reticulation network is a dedicated subsystem comprising both supply and collection and disposal subsystems and are interconnected within the reticulated urban areas of Boksburg and Brakpan. The sewer reticulation network separates residential and industrial effluent from stormwater.

The analysed system is assumed to be a typical municipal water services system where the rehabilitated water is not extensively reused or recharged to the ground, but discharged directly into the receiving river ecosystem. However, water is moderately recycled in-house primarily by industries situated in the Boksburg urban areas. The following equation can be used to determine the bulk water balance for the Vlakplaats WWTP if not all records are available.

$$TWS + (SWI + GWI) – EXF = WWI + CWU + WEX \qquad (D1)$$

Where: TWS = total Rand Water supplies into investigated area
SWI = stormwater inflows (ingress)
GWI = groundwater infiltration
EXF = exfiltration from sewers
WWI = wastewater influent from waterborne sewerage
WEX = water exported from a system
CWU = consumptive water use = BWL + UAW + EDL + WWL
BWL = bulk water losses (typically between 2 and 8 percent of total bulk supply)
UAW = unaccounted for water including unmetred water use
EDL = effluent diffused locally (i.e. wastewater not reaching a WWTP)
WWL = wastewater treatment losses (typically between 2 and 5 percent of total influent)

- *Total water supply***.** As mentioned previously, all water users situated within the investigated area are supplied from the Rand Water system. Table D.1 illustrates the build up of water supplies into the area.

Table D.1. Metered water supply to Vlakplaats sewer reticulation area

Urban area supplied	**Rand Water potable water supplies (Mℓ/d)**							
	1993	**1994**	**1995**	**1996**	**1997**	**1998**	**1999**	**2000**
Boksburg (ptn)	28,4	28,7	31,9	26,4	32,1	37,2	36,8	37,5
Benoni (ptn)	6,6	6,8	7,2	7,4	7,5	7,8	7,6	7,7
Brakpan (ptn)	11,6	11,1	12,0	11,0	10,7	11,5	11,5	11,5
KwaThema (ptn)	13,8	16,2	17,6	17,0	17,4	17,6	17,4	17,6
Tsakane (ptn)	2,1	2,4	2,7	2,9	3,3	3,3	3,2	3,2
Vosloorus	25,5	24,1	25,9	25,6	25,9	26,1	26,0	26,1
Total (Mℓ/day)	**88,0**	**89,3**	**97,3**	**90,3**	**96,9**	**103,5**	**102,5**	**103,7**

Denotation: ptn = portion only

All other water sources that might be used within the urban areas of the study area are considered marginal in the overall analysis.

- *Wastewater returns from urban areas.* Typical unit domestic and industrial wastewater flows from urbanized areas in South Africa are given in Table D.1. These values are commonly applied in the design and development of municipal wastewater reticulation and treatment facilities.

A useful method is based on consumptive water use coefficients of the eight urban water use categories as illustrated in Table D.2. This approach is used in determining return flows draining into the Vlakplaats WWTP from the residential and commercial areas of Boksburg and Brakpan as well as adjacent townships.

Table D.2. Return flows from urban areas draining into Vlakplaats WWTP (Mℓ/d)

Urban area	Rand Water supply	Urban return flows (URF)	Overall return ratio
Benoni (ptn)	7,0	4,4	0,629
Boksburg (ptn)	30,3	19,0	0,627
Brakpan (ptn)	11,6	7,3	0,629
KwaThema (ptn)	16,9	10,1	0,598
Tsakane (ptn)	2,6	1,6	0,615
Vosloorus	25,0	15,0	0,600
TOTAL	**93,4**	**57,4**	**0,615**

Notes: ptn = portion, 1995 the basis, $RFU_i = UWU_i * (1 – PL_i)$

(v) Determination of extraneous flows (I/I events) for a system

- *Approach if metered flows are available*. In principle, the amount of infiltration/inflow reaching a sewer conduit depends on its length and age, the construction material and workmanship during installing, number of illegal gulley connections, the relative level of the groundwater table to the sewer location, type of soil and ground vegetation cover as well as the general topographic conditions.

When dealing with a specific portion of a sewer network, all or most of the above listed factors should be taken into consideration to determine I/I events. In evaluating the whole subsystem or a major branch of a subsystem, the infiltration/inflow can be determined from available wastewater influent records measured in most instances at the WWTP. It should be noted that the average wastewater flow may vary from 60 to 130 percent of water used in an urban area.

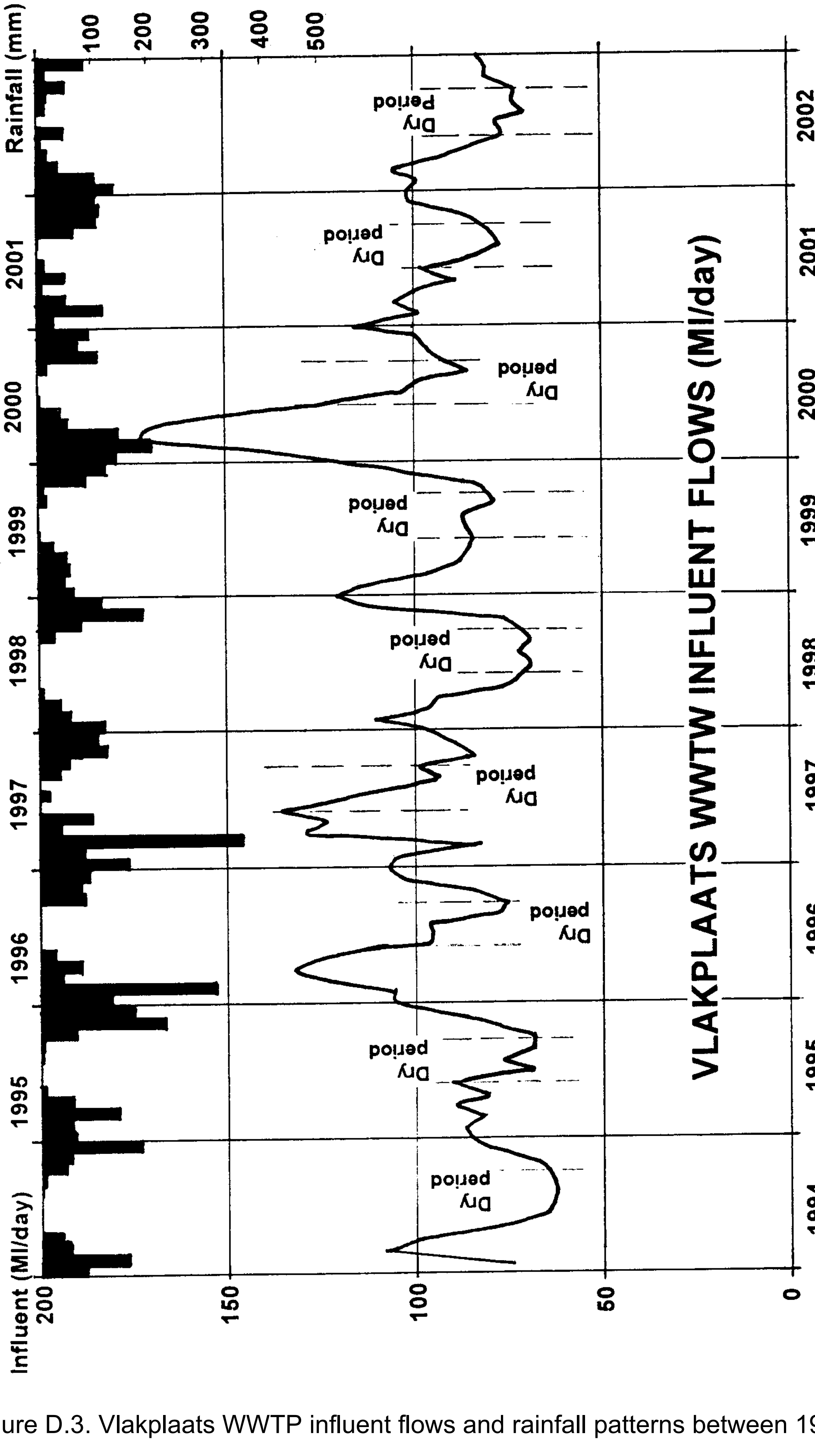


Figure D.3. Vlakplaats WWTP influent flows and rainfall patterns between 1994 and 2002

- *Driest period of the year*. Using 10 year average rainfall, the driest period for the study area is determined between month June and month September of each year. The correlation pattern between actual rainfall and sewer e\influent at the Vlakplaats WWTP is illustrated in Figure D.3 and Table D.3. The driest period of four months also defines the lowest influent flows to Vlakplaats WWTP. According to Table D.3 influent flows are averaged to some 61,2 Mℓ/day.

Table D.3. Methodology in determining extraneous flows (if records are available)

Month	10 year av. rainfall (mm)	Vlakplaats influent (Mℓ/day)	Wastewater from users (Mℓ/day)	Difference (Mℓ/day)	Infiltration and inflow (ℓ/s)
Jan	104	72,4	57,9	14,5	168
Feb	131	87,3	69,6	17,7	205
Mar	104	86,8	68,9	17,9	207
Apr	36	72,9	57,9	15,0	174
May	34	72,4	57,9	14,5	168
Jun	4	59,9	47,5	12,4	144
Jul	1	63,7	51,0	12,7	147
Aug	3	60,4	48,2	12,2	141
Sep	17	60,8	48,2	12,6	146
Oct	77	66,6	53,0	13,6	157
Nov	99	74,9	59,9	15,0	174
Dec	118	86,1	68,8	17,3	200
Total	**728**	**864,2**	**688,8**	**175,4**	**2030**

- *Urban wastewater flows generated*. Based on metered water supplies into the study area as illustrated in Table D.3 and urban wastewater return flows by means of monthly factors, the average residential, commercial and industrial wastewater generated in the study area amounted to some 48,7 Mℓ/day for the driest period of the year.
- *Estimation of groundwater infiltration rates*. The difference between metered influent flows and urban wastewater flow generated by the users in the driest period of the year amounted to 12,5 Mℓ/day (or 145 ℓ/s). This amount represents the most probably rate of groundwater infiltration including leakage losses (e.g. leaking toilets, etc.). The amount of domestic and industrial leakages for the study area is assumed at 6 ℓ/s based on studies conducted in Johannesburg in urban areas with similar character and dynamics.
- *Impact of infiltration rate on WWTP capacity*. The present capacity of Vlakplaats WWTP is 83 Mℓ/day. Taking into consideration that about 12,5 Mℓ/day of treatment capacity has to be allocated to groundwater infiltration, some 15 percent of design capability has been lost due to infiltration between 1968 and 1995 (i.e. over the 27 years of the WWTP's existence).
- *Rate of infiltration along outfalls*. The rate of groundwater infiltration is 12,5 Mℓ/day for the whole system. The outfall is 100 km long and the sewer diameter is 1050 mm. The unit infiltration determined from given parameters is determined as:

0,114 kℓ/day/mm-dia/km-length (or 0,08ℓ/min/m-dia/m pipe)

To enable benchmark comparison on the severity of infiltration, a nominal rule by SABS 1200 is used.

0,01ℓ/min/m-dia/m-pipe length (D2)

Based on this comparison, the Vlakplaats outfall groundwater infiltration is moderately excessive. This means that this problem needs urgent attention.

- *Rate of infiltration by monitoring programme.* By all practical terms, I/I/E problems can only really be addressed by field measurements before any rehabilitation programme can be considered. However, it would be expensive to measure an entire wastewater reticulation network to search for I/I/E problems. For this reason, it is necessary to monitor critical areas in order to prioritize remedial activities. The rate of most severe infiltration rate determined from a monitoring programme conducted by GMKS (1999) for the specific sewer portion at Boksburg Stadium amounted to 0,535 m^3/ day/mm-dia/km-pipe (or 0,372 ℓ/min/m-dia/m-pipe). Applying the nominal rule from Equation D2 above, the infiltration taking place at that specific section of the system is highly excessive for that specific portion of the urban sewer system.

APPENDIX E: Programme for predicting future reliability of a system

E.1. Prediction of future reliability of a system

To evaluation and predict future reliability of a water services system (or it subsystems) requires investigating the system complexity, management practices, maintenance programme and costs. A comprehensive programme layout proposed for predicting the reliability of a system is illustrated in Figure E.1. The broad criteria in this assessment approach are compliance, safety, capacity and costs.

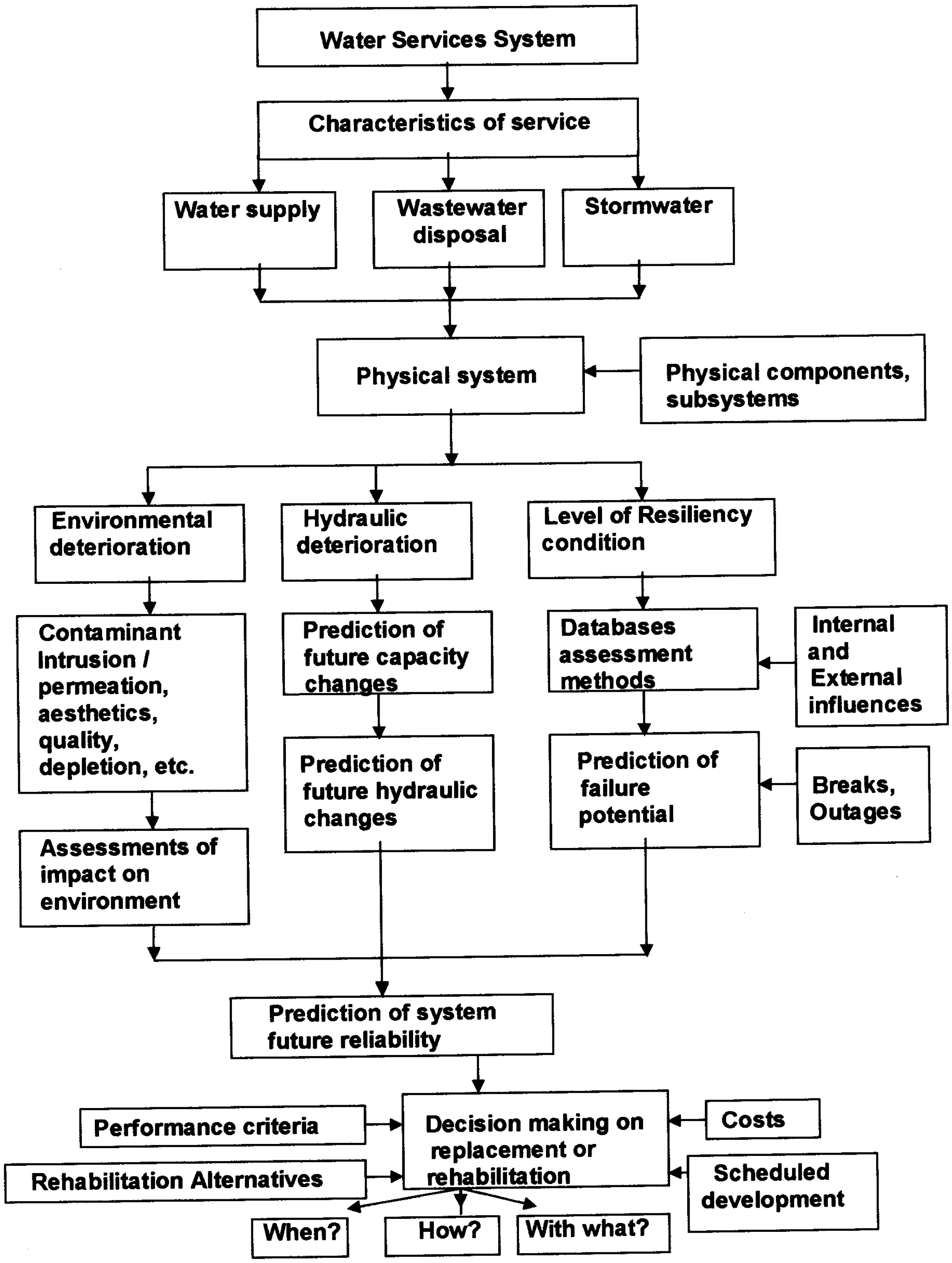


Figure E.1. Programme for predicting future reliability of a system

E.2. Quantitative risk analysis

E.2.1 Risk assessment for a sewer system

A risk can be defined as the product of the likelihood of an event and the consequences of that event. The consequences are rather difficult to define and are most commonly attached to the risk to human life. However, to determine the risk associated with the poor condition and performance of a wastewater system would be fare more complex. To assess the risk for such a system, it is necessary to determine individual components of risk and combine them together to obtain the overall risk situation. A risk assessment is either carried out as a quantitative or qualitative analysis.

E.2.2 Probability and frequency in qualitative risk analysis

It is essential to consider the combination of events in the assessment of the risk of a system or its vital components. The events are represented by a combination of probabilities and frequencies. This is commonly explained as follows: Two possible events A and B are considered causing C and generating the following probabilities and frequencies:

$$P_{A\text{ or }B} = P_A + P_B - P_A P_B \qquad \text{(E1)}$$
$$P_{A\text{ or }B} = P_A + P_B\text{, if } P_A \text{ and } P_B \text{ are smaller than } P_{A\text{ and }B} = P_A * P_B \qquad \text{(E2)}$$
$$F_{A\text{ or }B} = F_A + F_B \qquad \text{(E3)}$$
$$F_{A\text{ or }B} = F_A * F_B * (\tau_A + \tau_B) \qquad \text{(E4)}$$

Where: τ_A and τ_B are the duration of the events A and B

Units of frequency are expressed as occasion/year (occ/yr). Frequencies can be multiplied by probabilities.

E.2.3 Quantifying the risk costs

The cost of risk to the WSA or WSP needs to be assessed for all failures ranging from those needing minor maintenance to major catastrophic structural failures. The reduction or avoidance of risk needs to be quantified as a benefit to the WSA/WSP.

Current Risk Cost (Benefit) = Probability of Failure within next 12 months * cost of the consequence of Failure
(E5)

This approach can assist in the identification of the components that might have a high probability of failure (or highest risk to the WSA/WSP). typical consequences of failure can be listed as follows:

- *Effect on public health (potential loss of life*
- *Damage to private property*
- *Effect on business capacity*
- *Effect on essential services*
- *Disruption to traffic or public transport*
- *Inconvenience to residents (ratepayers) due to repair cost (e.g. digging, access, etc.)*
- *Availability of spare materials*
- *Cost of providing the service during failure*
- *Damage to the environment*
- *Actual cost of the repair*
- *Public image/public relations loss*

It should be noted that in some systems, a multitude of failures can take place. The timing of a failure is likely to affect the cost. The cost of failure excludes both the cost of the lost product and the cost of repair (replacement). For fully repairable single failure state system, the following applies:

$$C_{failure} = [(CR + CLP) * u] * SOT \qquad \text{(E6)}$$

Where: CR = repair cost per hour
CLP = cost of lost production per hour
U = system availability (probability of failure)
SOT = system operating hours per annum (i.e. typically 8760 hours)

The following applies for a fully reparable, multi-failure state system:

$$C_{failure} = [(CR_1 + CLP_1) * U_1 + (CR_2 + CLP_2) * U_2] * SOT \qquad \text{(E7)}$$

Where: CR_1, CR_2 = repair cost per hour for failure state 1 and state 2 respectively
CLP_1, CLP_2 = cost of lost production per hour for failure state 1 and state 2 resp.
U_1, U_2 = system unavailability for failure state 1 and state 2 respectively
SOT = system operating hours per annum (full year = 8760 hours)

The total operating cost (C_{OP}) is the sum of the failure costs, the engineering charge costs (C_{EC}), the fixed maintenance costs (C_{FM}) and the consumable costs (C_{CC}). Since the cost of failure calculation accounts for operating hours in one year, that cost must be multiplied by the anticipated lifetime of the system:

$$C_{OP} = [(C_{EC} + C_{FM} + C_{CC} * C_{failure}) * \text{years of life}] \qquad \text{(E8)}$$

APPENDIX F: Examples on costing procedures to control of extraneous flows

F.1 Costing of sewer flow metering and analysis

Table F.1 illustrates costing of field flow metering as experienced by the researchers during metering of flows for this project.

Table F.1. Costing of field flow metering (cost base 2003)

Item	Activity	Unit cost
1.	Client to define flow metering positions	R200/hr
2.	Contractor to establish flow measurement stations (a) Flume (b) Secure manholes	 Sum Sum
3.	Clean-up upstream sewer line from contingencies	Proportion of contingency
4.	Installation of measuring equipment (a) To purchase (b) To hire	 R12 500/device R3 000/week/site
5.	Cost for setting up metering device (including calibration of equipment)	R350/device
6.	Intermediate readings (including battery charges, etc.)	R200/device
7.	Removal of metering units and data downloading Add software package	R280/device R4 000
8.	Interpreting readings and reporting	R300/device

Note: VAT is excluded

F.2. Costing of preventative measures

There is a growing recognition of the problems of groundwater and stormwater inflow into sewers. The methods that most WSAs/WSPs should use to control infiltration are currently remedial and not preventative in nature. Remedial solutions are undesirable because they are merely temporary arrangements and do not necessarily solve the problem in the long-term. Some of these methods involve:

Table F.2. Summary of costing preventative measures (cost base 2003)

Preventative measure	Description of problem	Estimated cost
Sealing of sewers	Remedial measures by rehabilitation methods	Depending on choice of measure
Replacing missing or broken manhole components	Covers only Covers and frames Covers and cover slabs Reconstruct manhole (e.g. raising manholes above flooding)	R450/unit R900/unit R1200/unit R4000/unit
Training of maintenance staff	Qualification and experience	LGWSETA rates based
Regulatory measures	Policing, etc.	Budget sum

Notation: LGWSETA = Local Government and Water SETA

F.3. Costing of increased capacity of WWTP

The design of WWTP to accommodate extraneous flows appears as one of the most expensive options but the one done most commonly. It may cost a WWTP on average R3 million per ℓ/d or R300 000 per ℓ/s of inflow. The cost is related to the hydraulic capacity and BOD loading. The hydraulic related components are the ones to consider, since inflow/infiltration are more hydraulic than quality problems. The components related to peak flow rate are:

- *Pipework* (cost proportional to flow to the power of 0.5)
- *Inlet works: screens, grit channels* (cost proportional to flow rate)
- *Settling tanks* (cost proportional to flow rate)

It must be noted that the costs of sludge handling, digesters, drying beds, aeration or filters and tertiary treatment depend primarily on the pollution load of the WWTP's influent.

The cost of the hydraulic related components is approximately 30% of the cost of the works (i.e. R100 000 per ℓ/s of inflow). This type of expenditure could be used to rehabilitate or upgrade a considerable length of sewer. Peak flows could be reduced 10% by upgrading. At a rehabilitation cost of R1 000/m/m dia, and assuming 500 mm dia sewers with a capacity of 300 ℓ/s, then if flow could be reduced by 10% (i.e. by 30 ℓ/s), R3 million could be spent (i.e. R3 000 000 / R500 = 6 000 m could be renovated instead of extending the WWTP capacity).

If more flow reduction than 10% were possible, a greater length of sewer could be rehabilitated.

F.4. Costing of new sewer development

Costs are based on activities including: P & G, site clearance, excavation and backfill, rock excavation, bedding, supply, lay and test pipes, supply and construct manholes, house connection, contingencies (20%) and engineering fees (14%).

Table F.3. Illustrative sewer pipe development material costs

Pipe material	Diameter (mm)	New development area (R/m)	Development in existing area (R/m)
Clay pipes	150	390	511
	200	383	531
	250	434	610
	300	542	749
Concrete pipes	375	700	802
	450	984	1053
	525	1187	1257
	600	1391	1515
	750	1870	1944
	900	2335	2567
	1050	2944	3103
	1200	3618	3792
	1350	4226	4568
	1500	4767	5455
	1650	6052	6200
	1800	6828	7310
	2000	9511	9402
	2250	11592	11592
	2500	13856	13670

Source: JHB Metro (2001)

Notes:
(i) Sewers up to 400mm are assumed to be on average 2m deep
(ii) Sewer outfalls are assumed to be on average 3m up to 1500mm dia and 4m deep > 1500 mm dia.
(iii) Unit cost exclude VAT
(iv) Cost escalations to date, add 10%

F.5. Costing approach to sewer rehabilitation

Wessels (2002) illustrated a costing approach based on a WSA administering between 5000 and 8500 km of sewer pipe network Capital-intensive schemes to reduce flooding should not be conducted in isolation. They should be assessed on a catchment-wide basis, taking the opportunity to investigate the potential to improve receiving water quality by the reduction in the number and frequency of storm discharges. Similarly, any other operational shortcomings of the system should be addressed at the same time. This approach could improve the cost-benefit ratio of a capital-intensive scheme and thereby turn it into a "low-cost" option. All costs are illustrative only.

(i) Annual budget for infrastructure maintenance:

Sum: R20 to R50 million/annum

(ii) Maintenance and rehabilitation based on the following assumptions:

- *1,17 blockages/km pipe/year average,*
- *R400 per blockage to unblock,*
- *Cost of unblocking the blockage:*
  *R400 * 1,17 = R484/km pipe/year that should be replaced/year*
- *Prevention of extra loading on wastewater treatment works due to infiltration into the sewer:*
  Assume: 1,7 ℓ/s/km pipe infiltration during rainy season
- *Water purification cost @ R0,50/m^3.*
  Assume: 6 months for a season
- *Estimate of cost for the pipe which should be replaced*:
  *1,7 * 6 * 2592 * 0,50 = R13 219/km/year*

(iii) Capital cost of trenchless replacement:

Assume replacement cost:	*R230/m*
Useful life:	*60 yrs*
Interest rate:	*10%*

Then cost of replacement:

*R230/m * 1000 * 0,10033 = R23 075/km pipe replaced/year*

- *If sewer already collapsed:*
  Assume: open excavation rate: R1000/m
- *Cost of replacement by open excavation:*
  *(1000 – 230) * 1000 * 0,10033 = R77 254/km pipe/year (discounted)*

(iv) Cost if trenchless technology is adopted in a sewer/maintenance programme:

Assume: R468 + R13 219 + R77 254 = R90 941/km of pipe replaced per year

Compared with estimated cost of trenchless technology:

Potential savings: R90 951 – R23 075 = R67 866/km/year

Note: The assumption of 1,7 ℓ/s/km pipe infiltration is arbitrary and has to be verified from survey results in the field for a specific area.

APPENDIX G:. Life-cycle costing of sewage pumps

$$\text{Life-Cycle Costs (LCC)} = [C_{ic} + C_{in} + C_e + C_o + C_m + C_s + C_{env} + C_d] \qquad \text{(G1)}$$

Where:
- C_{ic} = initial costs, purchase price (pump, system, pipe, auxiliary services)
- C_{in} = installation and commissioning cost (including training)
- C_e = energy costs (predicted cost for system operation, including pump driver, control and any auxiliary services)
- C_o = operating cost (labour cost of normal system supervisor)
- C_m = maintenance and repair cost (routine and predicted repairs)
- C_s = down time and lost of production costs
- C_{env} = environmental cost (contamination from pumped liquid and auxiliary equipment)
- C_d = decommissioning and disposal cost (including restoration of the local environment and disposal of auxiliary services)

Table G.1. Distribution of costs over 20 years – 30% utilisation

Pump (Head in m)	**A10**	**B10**	**C10**
Q (ℓ/s)	20	150	500
H (m)	10	10	10
Power (kW)	2.6	19.1	61.3
Purchase cost (%)	22	17	16
Energy cost[1] (%)	60	73	73
Maintenance[2] (%)	18	10	11
	100%	100%	100%
Pump (Head in m)	**A20**	**B20**	**C20**
Q (ℓ/s)	20	150	500
H (m)	20	20	20
Power (kW)	6.0	37.0	124.1
Purchase cost (%)	14	12	10
Energy cost[1] (%)	76	81	86
Maintenance[2] (%)	10	7	4
	100%	100%	100%
Pump (Head in m)	**A40**	**B40**	**C40**
Q (ℓ/s)	20	150	500
H (m)	40	40	40
Power (kW)	12.4	77.4	245.1
Purchase cost (%)	13	`0	8
Energy cost[1] (%)	80	84	88
Maintenance[2] (%)	7	6	4
	100%	100%	100%

[1] Energy. Costs based on present prices, no allowance for inflation or loss of efficiency due to blockages or wear during the life time of the pump.
[2] Maintenance costs are for routine maintenance and repairs including spare parts. Excludes unscheduled maintenance such as unblocking pumps.

Table G.2. Effects of pump utilisation on costs

Pump	A10			B10			C10		
Utilisation	5%	30%	60%	5%	30%	60%	5%	30%	60%
Purchase cost %	60	225	12	57	17	9	56	16	8
Energy %	28	60	66	40	73	78	42	73	78
Maintenance %	12	18	22	3	10	24	2	11	14
	100%	100%	100%	100%	100%	100%	100%	100%	100%
Pump	**A20**			**B20**			**C20**		
Utilisation	5%	30%	60%	5%	30%	60%	5%	30%	60%
Purchase cost %	48	14	8	47	12	7	41	10	6
Energy %	43	76	81	51	81	85	58	86	89
Maintenance %	9	10	11	2	7	8	1	4	5
	100%	100%	100%	100%	100%	100%	100%	100%	100%
Pump	**A40**			**B40**			**C40**		
Utilisation	5%	30%	60%	5%	30%	60%	5%	30%	60%
Purchase cost %	47	13	7	41	10	5	36	8	4
Energy %	48	80	85	58	84	87	63	88	91
Maintenance %	5	7	8	1	6	8	1	4	5
	100%	100%	100%	100%	100%	100%	100%	100%	100%

Source: Jackson (2002)

A pump with 5% utilisation could be a typical storm pump
A pump with 30% utilisation could be a typical network pumping station
A pump with 60% utilisation could be an inlet or Return Activated Sludge pump

Note: It should also be remembered that the percentage of costs attributable to energy will be higher than those shown in Tables G.1 and G.2, if rising energy costs due to inflation and taxes are taken into account. Plus additional energy consumption arising from lower actual efficiency during the life time of the pump and additional energy from running partially blocked sewage pumps