DEVELOPMENT OF A PROCEDURE FOR SEPARATELY ALLOCATING WATER LEAKAGE AND OTHER TYPES OF NON-METERED WATER TO NODES IN THE HYDRAULIC MODEL

by

Gerrit van Wageningen

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Supervisor: Prof. H.E. Jacobs

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ABSTRACT

The correct allocation of water loss to nodes in hydraulic models of water distribution systems is an important consideration for the purposes of designing such systems. Some components of total water demand (e.g. metered-consumption) are relatively simple to determine, for example, by analysing recorded meter consumption data. However, the extent and spatial distribution of non-metered water (including water losses) is often much more challenging to determine. Designers of water distribution system infrastructure and analysts need to be able to distinguish between the water that is lost from a water distribution system due to leakage (real loss) and that which is not accounted for as result of non-metered consumption (e.g. non-metered authorised consumption and apparent loss).

A possible shortcoming has been identified regarding the current assumptions for water loss modelling. The customary practice employed by consultants, whereby water loss is distributed among nodes in proportion to the metered consumption at those nodes, is often unrealistic. This research project focused on the evaluation and further development of an already existing technique for incorporating water losses in hydraulic models by segregating leakage from other types of non-metered water, as well as accounting for selected factors that influence water loss spatially.

The literature reviewed indicated that limited research had been conducted on techniques for distinguishing between different types of water loss when performing hydraulic analyses. Most earlier research studies focussed on the pressure-leakage relationship and methods for improving the modelling of leakage from distributions systems. Furthermore, not much work could be found on the potential impact that different approaches to estimating leakage would have on the ultimate results obtained from hydraulic models.

A computer-based modelling procedure titled *SEGLEAK* was developed as part of this research study, after which it was implemented and tested on a hydraulic model of a real water distribution system in South Africa, as part of a case study problem. The *SEGLEAK* procedure provided an effective and practical technique for distinguishing between leakage and non-metered consumption when making use of hydraulic modelling.

OPSOMMING

Die korrekte toewysing van waterverlies aan nodusse in hidrouliese modelle van waterverspreidingstelsels is 'n belangrike oorweging vir die ontwerp van sulke stelsels. Sommige komponente van die totale water aanvraag (bv. gemeetde verbruik) is relatief maklik om te bepaal, byvoorbeeld deur die opname van aangetekende meterverbruiksdata te analiseer. Die omvang en ruimtelike verspreiding van nie-gemeterde water (insluitende waterverliese) is egter dikwels meer uitdagend om te bepaal. Ontwerpers van waterdistribusiestelsel-infrastruktuur en ontleders moet kan onderskei tussen die water wat verlore gaan van 'n waterverspreidingstelsel as gevolg van lekkasie (werklike verlies) en wat nie verantwoord word as gevolg van nie-gemete verbruik (bv. nie- gemete gemagtigde verbruik en oënskynlike verlies).

'n Moontlike tekortkoming is geïdentifiseer met betrekking tot die huidige aannames vir waterverliesmodellering. Die gewone praktyk in diens van konsultante, waardeur waterverlies onder nodusse verdeel word in verhouding tot die gemete verbruik by daardie nodusse, is dikwels onrealisties. Hierdie navorsingsprojek het gefokus op die evaluering en verdere ontwikkeling van 'n reeds bestaande tegniek vir die inkorporering van waterverliese in hidrouliese modelle deur lekkasie van ander soorte nie-gemeterde water af te skei, asook om rekening te hou met geselekteerde faktore wat ruimtelike verlies aan waterverlies beïnvloed.

Die literatuur wat ondersoek is, het aangedui dat daar beperkte navorsing gedoen is oor tegnieke om te onderskei tussen verskillende tipes waterverlies by die uitvoer van hidrouliese ontledings. Die meeste vroeëre navorsingstudies het gefokus op die druklekkasieverhouding en metodes om die modellering van lekkasie uit verspreidingsisteme te verbeter. Verder kan nie veel werk gevind word oor die moontlike impak wat verskillende benaderings tot skatting van lekkasie op die uiteindelike resultate van hidrouliese modelle sal hê nie.

'n Rekenaargebaseerde modelleringsprosedure met die titel *SEGLEAK* is ontwikkel as deel van hierdie navorsingsstudie, waarna dit geïmplementeer en getoets is op 'n hidrouliese model van 'n ware waterverspreidingstelsel in Suid-Afrika, as deel van 'n gevallestudieprobleem. Die *SEGLEAK*-prosedure verskaf 'n effektiewe en praktiese tegniek om onderskeid te tref tussen lekkasie en nie-gemete verbruik wanneer gebruik gemaak word van hidrouliese modellering.

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LIST OF SYMBOLS

The following is a list of symbols that have been used in this document and the definitions of these symbols are as stated in the list, except where specifically indicated otherwise. For cases where the same symbol is used for parameters of different meaning, the context in which the symbol appears should provide sufficient clarity on its intended meaning.

A	-	Altered leak area
A	-	Orifice area
A_0	-	Initial leak area
C	-	Leakage coefficient
C_d	-	Discharge coefficient
C_j	-	Leakage coefficient for node j
f_{nrl}	-	Real losses fraction of non-metered water
g	-	Gravitational acceleration constant
h	-	Pressure head
h_j	-	Pressure head at node j
K_l	-	Average leakage constant
K_{lj}	-	Leakage constant for node j
$K_l(f_{nrl})$	-	Average leakage constant
k	-	Ratio of non-metered consumption to metered consumption
k_1	-	Constant for openings of fixed area
k_2	-	Constant for openings of variable area
L_j	-	50% of total length of mains connected to node j
L_m	-	Length of mains
L_p	-	Length of unmetered underground pipe
L_T	-	Total length of mains
$\bar{L}_j$	-	Length weighting factor for node j
m	-	Head-area slope
$N1$	-	Leakage exponent
N_c	-	Number of service connections
n	-	Number of nodes
P_{ave}	-	Average operating pressure at average zone point
Q	-	Discharge
Q_d	-	Total demand flow rate

$(Q_d)_j$	-	Demand flow rate at node j
Q_i	-	Total input flow rate
Q_{mc}	-	Total metered consumption flow rate
$(Q_{mc})_j$	-	Metered consumption flow rate at node j
$(\bar{Q}_{mc})_j$	-	Daily average metered consumption flow rate at node j
Q_n	-	Total non-metered flow rate
$(Q_{nac})_j$	-	Non-metered authorised consumption flow rate at node j
$(Q_{nal})_j$	-	Non-metered apparent loss flow rate at node j
Q_{nc}	-	Total non-metered consumption flow rate
$(Q_{nc})_j$	-	Non-metered consumption flow rate at node j
Q_{nrl}	-	Total non-metered real loss flow rate
$(Q_{nrl})_j$	-	Non-metered real loss flow rate at node j
$Q_i^m(t)$	-	Measured total input flow rate
$Q_i^m(t)_j$	-	Measured total input flow rate at hour j
$Q_i^s(t, f_{nrl})$	-	Simulated total input flow rate
$Q_i^s(t, f_{nrl})_j$	-	Simulated total input flow rate at hour j
t	-	Time
V_{mc}	-	Total daily volume of metered consumption
V_i^m	-	Measured total daily input volume
V_i^s	-	Simulated total daily input volume
α_j	-	Leakage constant weight for node j
ℓ	-	Litre
$\bar{\eta}_s$	-	Daily average system efficiency
µ	-	Average
Σ	-	Sum
σ	-	Standard deviation
σ^m	-	Standard deviation of $Q_i^m(t)$ values over a single day period
$\sigma^s(f_{nrl})$	-	Standard deviation of $Q_i^s(t, f_{nrl})$ values over a single day period

ABBREVIATIONS AND ACRONYMS

a.m.	-	Ante meridiem (before midday)
AADD	-	Annual average daily demand
BABE	-	Burst and Background Estimate
CARL	-	Current annual real losses
c	-	Capita
d	-	Day
Eq.	-	Equation
e.g.	-	Exempli gratia (for example)
FAVAD	-	Fixed and Variable Area Discharges
h	-	Hour
ILI	-	Infrastructure Leakage Index
IWA	-	International Water Association
i.e.	-	Id est (that is)
km	-	Kilometre
Ltd.	-	Limited
m	-	Metre
No.	-	Number
no.	-	Number
Pty.	-	Propriety
s	-	Second
UARL	-	Unavoidable annual real losses
UK	-	United Kingdom
yr	-	Year

1. INTRODUCTION

1.1. Background

Water loss in the form of leakage from water distribution systems is a major challenge faced globally by service providers. According to Winarni (2009), leakage usually forms the primary component of water loss in developed countries, whereas illegal connections, metering error, or other accounting errors are often more significant in developing countries. McKenzie and Seago (2005b) stated that some of the most common forms of leakage include: (1) leakage on transmission and distribution mains; (2) leakage and overflows at storage facilities; and (3) leakage on service connections up to the point of customer meters. Van Zyl (2014) furthermore stated that there will always be some measure of leakage from any water distribution system, and that it is practically impossible to eliminate all forms of leakage.

Water suppliers could minimize the amount of water lost through leakage by implementing several types of leakage management (e.g. proper pressure management). Consistent maintenance procedures and regular physical inspections of a water distribution system also greatly facilitate the reduction of leakage. An economical balance must, however, be established between the undertaking of certain maintenance endeavours and the mere acceptance of certain levels of leakage (real loss) from such a system. Water suppliers therefore need to calculate whether the amount of water saved through proposed mitigation strategies would be worth the overall cost of implementing the strategies themselves.

According to McKenzie and Langenhoven (2001), many varied factors influence the volume of water lost through leakage from potable water distribution systems. These authors identified the following factors as among some of the most significant: (1) average operating pressures; (2) length of mains; (3) number of service connections; (4) pipe material and surrounding soil conditions; (5) quality of workmanship during system installation; and (6) levels of added protection on pipe materials. Giustolisi *et al.* (2008) further recognised pipe age, pipe diameter, and pipe material as some of the primary variables influencing the process of pipe degradation, which ultimately leads to pipe failure, and thus causes additional leakage from a water distribution system.

To properly model leakage in a water distribution system, an understanding is needed of the most notable factors influencing the occurrence of leakage, the volume of leakage (real loss) in relation to the total volume of water loss, as well as the spatial distribution of leakage in a system. Some contributing factors, such as average operating pressures, are directly proportional to the demand being placed on a system, whereas other factors, such as length of mains or number of service connections, remain relatively constant during normal operation of the system. A further set of factors may change gradually over extended periods of time, such as pipe material and surrounding soil conditions. The modelling of the effects that the above-mentioned factors have on the system leakage volume and the location of leaks in the system is further complicated by the interdependency of such factors. The primary factors responsible for leakage, the extent of their impact, and their spatial distribution therefore need to be accounted for, to be able to take appropriate measures of action.

1.2. Terminology

1.2.1. Definitions of Terms and Concepts

The definitions provided for the terms and concepts in the terminology section are specifically intended and valid for the purposes of this research study, and relate mostly to various distinct components of a water balance. A comprehensive discussion of the standard International Water Association (IWA) water balance is provided in Chapter 2 as part of the literature review, whereas an alternative simplified water balance, which was developed as part of this research study, is both introduced and explained in Chapter 4. All terms and concepts defined in this terminology section relate precisely to those used in the simplified water balance, and the method of classification between these terms and concepts is illustrated diagrammatically in Figure 1.1.

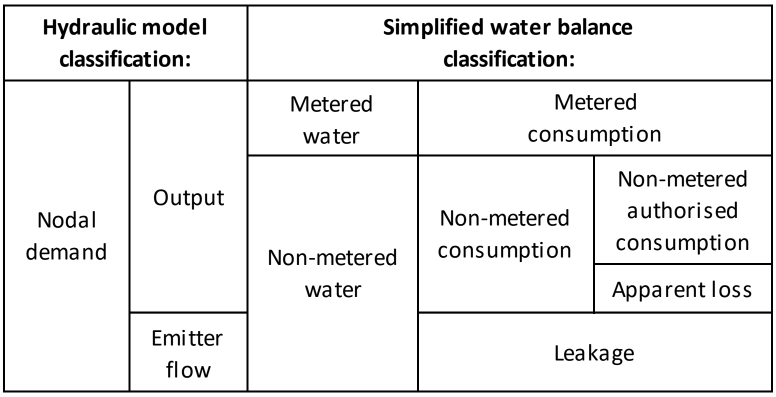

Figure 1.1: Components of nodal demand

1.2.2. Metered Consumption

Metered consumption is defined as the proportion of the total water use that is recorded by consumer water meters, which are generally located either at, or relatively close to, the property boundaries of consumers. All types of water loss upstream of consumer water meters are, therefore, excluded by the definition of metered consumption. On-site leakage (e.g. leaking toilet cisterns, dripping outside taps) is, however, included in the definition of metered consumption, since this volume of water will already have been recorded by consumer water meters by the time it is lost on the consumers' properties.

1.2.3. Non-Metered Consumption

Non-metered consumption comprises two separate components: (1) apparent loss; and (2) non-metered authorised consumption. First, apparent loss refers to the volume of water that is lost resulting from water simply seeming to disappear somewhere within a water distribution system, without being physically lost as leakage. Incorrect measurements by consumer water meters and unauthorised consumption of water (i.e. theft) are two good examples of apparent loss.

Secondly, non-metered authorised consumption refers to the volume of water that is either billed at a fixed rate, or not billed at all (e.g. public taps, schools, hospitals, irrigation of public parks, water used for system flushing purposes).

1.2.4. Leakage

Leakage (real loss) is defined as the difference between the total volume of water supplied (input to the system) and the volume of water that is attributed to consumption by users, whether recorded by consumer water meters or not. However, this definition of the term *leakage* excludes all types of on-site leakage at the properties of consumers. Leakage, in this sense, therefore refers to the physical loss of water from a water distribution system, upstream of consumer water meters.

1.2.5. Non-Metered Water

Non-metered water is defined as the difference between the total volume of water supplied (input to the water distribution system) and the volume of accounted-for water (i.e. metered consumption). An alternative definition of non-metered water would be the sum of non-metered consumption and leakage in a water distribution system.

1.2.6. Output

Output describes the flow rate that is allocated to a node in the hydraulic model of a water distribution system, and includes for both metered and non-metered consumption in the procedure that was evaluated and further developed as part of this research study. As stated by the definition of non-metered consumption, the components of non-metered consumption generally include both apparent losses and non-metered authorised consumption, which consequently means that both components are assigned to any node as part of the output value at that node. Furthermore, the output assigned to a node excludes the potential emitter flow from that node.

1.2.7. Emitter Flow

For the purposes of this study, the total emitter flow in a hydraulic model represents the total leakage volume (real loss) from the related water distribution system, which occurs upstream of consumer water meters. The emitter flow from a node is a function of the various contributing factors to leakage at that node, of which pressure is presumed to be a dominant contributing factor. This assumption forms part of the procedure that was evaluated and further developed through this research study.

1.2.8. Nodal Demand

For the purposes of this research, the nodal demand at a node refers to the sum of the output assigned to, and the emitter flow from, that node. This means that nodal demand includes metered consumption, non-metered consumption, and the leakage that occurs upstream of consumer water meters. The sum of all the individual nodal demand components must, therefore, be equal to the total supply (input) to the system.

1.3. Rationale

Water distribution systems are generally difficult to analyse because of their many components, non-linear hydraulics, and complex demand patterns, which makes the use of computer network models essential for calculating flow rates and pressures in such systems (Van Zyl, 2014). In recent years, the use of computer software for the analysis, design, and management of water distribution systems has become increasingly popular. Because of the advances made in information technology and geographical information systems, the water industry is now able to obtain all necessary information regarding water topology (Liu & Yu, 2014). During the process of using computer modelling for the purposes of designing a water distribution system, the designer would typically be interested in the total water demand that is expected to be imposed on the system. Since the total water demand of a water distribution system directly impacts the selection of certain system specific infrastructure (e.g. pipe sizes, pumping capacities, and the volumes required for storage facilities), the total water demand needs to be predicted as accurately as possible, during the initial process of designing the system.

Tools are available to allocate metered water consumption to hydraulic model nodes, based on spatial information of water meters and pipe topology (Jacobs & Fair, 2012). A problem that often arises, however, is that a significant part of the overall demand that is imposed on a water distribution system is attributable to water loss that occurs within the system. This water loss component, however, is not always as easy to estimate or predict, because of it being influenced by many uncertain factors. During the process of designing a water distribution system, it is often assumed by the relevant system designer that the volume of water loss at each node of the hydraulic model is merely proportionate to the metered consumption at each node.

Practicing engineers are known to base model results on crude assumptions of leak flow distribution (e.g. leaks could be uniformly distributed over all model nodes), despite the availability of more advanced methods. The reasons centre around the relative complexity of including the latest advancements of leakage modelling in the hydraulic models. In other words, water loss is thereby assumed to be independent of local contributing factors (e.g. pressure) within such a system. This assumption is considered inaccurate, since there are indeed many contributing factors influencing water loss in real-world distribution systems.

The above-mentioned problem can be defined as a modelling anomaly regarding the spatial distribution of water loss within water distribution systems. As mentioned in the previous paragraph, a proportional distribution of water loss among nodes is often assumed during the design of a water distribution system. However, areas with higher average network pressures, areas with larger densities of service connections, as well as areas with older components of system infrastructure, for example, are all more likely to experience higher levels of water loss, due to the greater volumes of leakage to be expected from such areas.

1.4. Problem Statement

Given available monthly water use from consumer water meters and total system input volume (or input volume per district metered area), how could hydraulic model nodes be populated with leakage flow rates in a more realistic, yet relatively uncomplicated way?

1.5. Approach

Designers and modellers of water distribution systems are often faced with the difficult challenge of making realistic assumptions regarding aspects of water loss from such systems. There are several different techniques used in practice to model water loss from water distribution systems, some of which are investigated in Chapter 2 as part of the literature review. A possible shortcoming has been identified regarding some contemporary assumptions for water loss modelling, with specific reference to the spatial distribution of water loss in hydraulic models.

A customary practice of using a distribution for which water loss is assumed to merely be proportional to metered consumption at each node of a hydraulic model, for example, is regarded as simplistic and often unrealistic. System designers can therefore greatly benefit from a more realistic methodology for the estimation of water loss within water distribution systems. An improved methodology could possibly improve the accuracy of future modelling processes by: (1) accounting for the most significant contributing factors that influence water loss flow rates; and (2) possessing reliable information regarding classification of the total water loss volume into its various separate components.

An alternative approach to the customary practice of distributing water loss proportional to metered consumption in a hydraulic model of a water distribution system is proposed as part of this research study, by correlating the extent of water loss with various contributing factors, as well as by segregating leakage volumes from other components of water loss. Since pressure has a prevalent impact on the leakage rate from a leak in a pipe, the focus of this research study was largely directed towards pressure-leakage relationships. The segregation and spatial distribution of leakage, as well as some other components of water loss (as presented in Figure 1.1), were considered and investigated as part of this study.

1.6. Research Objectives

The following key research objectives were set for this study:

- An extensive literature review of important concepts relating to water loss from water distribution systems, as well as the estimation and modelling of water loss from such systems.
- Evaluation and further development of an already existing procedure for the segregation of leakage from other components of water loss, and the allocation thereof to hydraulic model nodes.
- Practical application of the already existing procedure that was evaluated and further developed as part of this study to a case study problem involving a real water distribution system in South Africa.
- Investigation of the results obtained from analyses performed, the drawing of conclusions from the outcomes of this research study, and the provision of recommendations for future work to be done.

1.7. Delineation and Limitations

The delineation and limitations of this research study are as follows:

- An exclusive focus on the aspects of segregation and, subsequent, modelling of leakage from water distribution systems, although the modelling of other components of non-metered water (i.e. apparent loss, non-metered authorised consumption) is also incorporated to some extent.
- Although the quantity and spatial distribution of leakage from a water distribution system generally depends on many different contributing factors, the length of mains was selected as the dominant contributing factor for the procedure that was evaluated and further developed.
- Sufficient provision was made for the accounting of various other types of contributing factor as well, but the potential impact of such factors was not tested as part of this study. The reason for not including other types of contributing factor was that they were not anticipated to have measures of impact that were significant enough to be worth investigating, in comparison to that of pressure (Van Zyl & Clayton, 2007).
- The leakage exponent, $N1$, was not adjustable between separate nodes in the hydraulic model of the real water distribution system that was used as part of the case study problem. The reason for this was that the most recent version of *Wadiso*, which was used for the purposes of analyses, caters for only a single $N1$ value, which is valid for an entire hydraulic model. This meant that accurate estimation of the $N1$ value was even more important.
- The nature of the procedure that was followed as part of implementing extended period simulation on the hydraulic model of the case study problem was a definite limitation to this research study. The aforesaid procedure involved a rigorous and labour-intensive process of repeatedly adjusting certain parameters in the hydraulic model, to achieve some required balances, before the results could be used.

2. LITERATURE REVIEW

2.1. Overview

Water is becoming a critical issue of the twenty-first century (McKenzie & Seago, 2005a). Seago *et al.* (2005) suggested that water lost from potable water distribution systems remained one of the major concerns, particularly in developing countries. Van Zyl and Clayton (2007) also expressed that losses from water distribution systems were reaching alarming levels in many towns and cities throughout the world, primarily because such water distribution systems were ageing and deteriorating over time, while the demands on such systems (and thus on natural resources) were ever increasing.

The literature review for this research study starts off with a discussion of the various techniques that are used in practice for quantifying water loss. Some of the fundamental principles regarding leakage management are then presented, which is followed by a discussion relating to some of the commonly used methodologies for estimating leakage from water distribution systems. An interrelated suite of software models that had been developed specifically for the performing of calculations involving aspects of leakage from water distribution systems were also reviewed. Some existing approaches available for the allocation of water loss to the nodes in hydraulic models were furthermore investigated. An existing technique for segregating leakage from other types of non-metered water was introduced next. This was followed by a discussion of some of the useful findings that had been obtained from literature reviewed as part of this research.

2.2. Water Loss Quantification

2.2.1. General Practice

Water that is lost from a distribution system can be quantified through implementation of a water balance, which can be performed either on a system-wide basis or at the district metering area level, as expressed by Mutikanga *et al.* (2012). These authors also proposed that a water balance is an effective tool for the systematic accounting for water supply and consumption. Two main water balance methodologies are generally used for quantifying the volume of water losses: (1) the IWA (or American Water Works Association) standardised water balance (Lambert & Hirner, 2000); and (2) the United Kingdom (UK) water balance (Lambert, 1994). Mutikanga *et al.* (2012) further stated that the two above-mentioned water balance methodologies evolved from earlier works by Male *et al.* (1985) and the American Water Works Association (Wallace, 1987). The IWA standard water balance is the most widely implemented methodology worldwide. A more detailed discussion on this water balance is provided in the next section.

An additional customary practice for the quantification of water loss from water distribution systems is to make use of certain performance indicators. In general, such performance indicators indicate not only the quantity or volume of water loss from a water distribution system, but also provide valuable measures relating to the operational efficiency of such a system. A comprehensive discussion on the numerous performance indicators that are available for use is provided in a subsequent section.

2.2.2. Standard Water Balance

According to McKenzie and Seago (2005a), a clearly defined water balance is the first essential step in assessing the volumes of non-revenue water and the management of water losses from potable water distribution systems. Winarni (2009) stated that the *water balance* concept is based on measurements or estimations of: (1) water produced; (2) water imported and exported; (3) water consumed; and (4) water lost. In 1996, the IWA formed a water losses task force with the objective of developing international best practices in the field of water loss management (McKenzie & Lambert, 2004). A standardised water balance, as presented in Figure 2.1, was published by Lambert and Hirner (2000) as part of the best practices developed by the water losses task force.

<table>
<tr><td rowspan="10">System input volume</td><td rowspan="5">Authorised consumption</td><td rowspan="3">Billed authorised consumption</td><td>Billed water exported</td><td rowspan="3">Revenue water</td></tr>
<tr><td>Billed metered consumption</td></tr>
<tr><td>Billed unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled authorised consumption</td><td>Unbilled metered consumption</td><td rowspan="7">Non-revenue water</td></tr>
<tr><td>Unbilled unmetered consumption</td></tr>
<tr><td rowspan="5">Water losses</td><td rowspan="2">Apparent losses</td><td>Unauthorised consumption</td></tr>
<tr><td>Customer meter inaccuracies</td></tr>
<tr><td rowspan="3">Real losses</td><td>Leakage on transmission and distribution mains</td></tr>
<tr><td>Leakage and overflows at storage tanks</td></tr>
<tr><td>Leakage on service connections up to point of customer meter</td></tr>
</table>

Figure 2.1: The standardised IWA water balance (Lambert & Hirner, 2000)

From Figure 2.1, the system input volume is simply categorised into different components that comprises the total water balance. McKenzie and Seago (2005a) stated that the standard water balance proposed by the water losses task force (Lambert & Hirner, 2000) had since been widely adopted and recognised as international best practice by an increasing number of water utilities in various countries worldwide.

2.2.3. Simplified Water Balance

A research study by Almandoz *et al.* (2005) involved some water balance calculations that were more of a technical nature, as opposed to the managerial approach of the standard water balance that was introduced in the previous section. These authors proposed the use of a simplified water balance, which focusses more on whether the ultimate destination of water that is input to a distribution system is known, rather than on whether there is revenue associated with the distinct components of the water balance. An adapted illustration of the simplified water balance by Almandoz *et al.* (2005) is presented in Figure 2.2.

<table>
<tr><td rowspan="8">Flow rate entering the system, Q</td><td colspan="2" rowspan="4">Flow rate measured by customer meters, Q_m</td><td>Domestic consumption</td></tr>
<tr><td>Commercial consumption</td></tr>
<tr><td>Industrial consumption</td></tr>
<tr><td>Official consumption</td></tr>
<tr><td rowspan="4">Uncontrolled flow rate, Q_u</td><td rowspan="3">Flow rate consumed, but not measured by meters (apparent losses), Q_{uc}</td><td>Not measured on customer meters (metering errors), Q_{uce}</td></tr>
<tr><td>Billed by fixed quota users (non-metered water)</td></tr>
<tr><td>Fire hydrants, system flushing, illegal use (non-metered water)</td></tr>
<tr><td>Physical leakage in mains and service connections, Q_{ul}</td><td>Real losses</td></tr>
</table>

Figure 2.2: Simplified water balance (adapted from Almandoz *et al.* (2005))

2.2.4. Performance Indicators

2.2.4.1. General Overview

As mentioned before, the use of performance indicators is another common practice for quantifying water losses (and real losses in particular). Such performance indicators are generally used by various groups of water utilities for making decisions regarding whether the real losses from water distribution systems are within acceptable limits. Measurements on the operational efficiency of any distribution system, or processes of comparison with other such systems, are also made possible through the application of various performance indicators. Since many different factors potentially affect the volumes of real loss from a given water distribution system, a combination of performance indicators is generally required to properly account for the numerous contributing factors.

Lambert *et al.* (1999) presented the following basic traditional performance indicators for real losses, which are considered the most widely used for effectively comparing the annual volume of real losses between separate water distribution systems:

- Percentage (%) of system input volume;

- Volume lost per length of mains per unit time;
- Volume lost per property per unit time;
- Volume lost per service connection per unit time;
- Volume lost per length of system per unit time (length of system = length of mains + length of service connections up to point of customer metering).

Lambert *et al.* (1999) furthermore suggested that traditional performance indicators for real losses appeared to be selected based on either: (1) simplicity of calculation; (2) a country's tradition; (3) availability of data for calculation; or (4) the performance indicator that produced the best impression of a water distribution system's performance. These authors consequently advised that the basis of selection should be the performance indicator that provides the most rational technical basis for comparisons.

2.2.4.2. Problems with Using Percentages

Water loss in general is often expressed as a percentage of the system input volume. A percentage value is relatively easy to grasp and understand, but also has some problems relating to its use. Winarni (2009) explained that water loss as a percentage of system input volume is: (1) strongly influenced by consumption (and changes in consumption); (2) influenced by high pressure (above average pressure); (3) difficult to interpret for intermittent supply situations; and (4) not distinguishable between apparent and real losses. Winarni (2009) concluded that the use of percentages had therefore been unsuitable for assessing the efficiency of water distribution system management and often proved to be misleading.

McKenzie, Bhagwan, *et al.* (2002) used the following example (Lambert *et al.*, 1998) to demonstrate the problems associated with using percentage values alone to express real losses: A particular water distribution system experiences a total leakage flow rate of 10 000 m^3/d. An analysis was conducted on this system for a range of separate consumption related scenarios, which involved consumers from different countries making use of the same water distribution system. A summary of the analysis is presented in Table 2.1.

Table 2.1: Problems with using percentages example (adapted from Lambert *et al.* (1998))

Per capita consumption [ℓ/c/d]	Consumption volume [m^3/d]	Real loss volume [m^3/d]	Total input volume [m^3/d]	Percentage real losses [%]
25 (Standpipe)	6,250	10,000	16,250	61.5
50 (Jordan)	12,500	10,000	22,500	44.4
100 (Czech Republic)	25,000	10,000	35,000	28.6
150 (UK, France)	37,500	10,000	47,500	21.1
300 (Japan)	75,000	10,000	85,000	11.8
400 (USA)	100,000	10,000	110,000	9.1

It should be clear that even though these consumers all experienced the same amount of leakage (real loss), the percentage of real loss differs very significantly between the analyses. It may for this reason not be very useful to compare the percentage real loss between two separate water distribution systems, since the water use of one system might be very different to that of the other, which clearly influences the results significantly. If, for example, a single large consumer is present in a water distribution system, the percentage of real loss would consequently be lower as a result. If this user should, however, decide to relocate to some other area, the percentage of real loss would effectively increase, even though the volumes of real loss might not have changed at all. Similarly, if water utilities can persuade all users to use more water, the percentage of real loss would effectively decrease.

The need developed for an indicator that provided meaningful results and which would enable useful comparison of performance between separate water utilities. This problem was addressed by Lambert *et al.* (1999) through the introduction of the *Infrastructure Leakage Index* (ILI), which is based on the ratio of the actual level of real loss to the theoretical unavoidable level of real loss. These authors furthermore proposed that the customary practice of expressing real losses as a percentage of volume input needed to be rejected as a technical performance indicator, because of the problems related to its use.

2.2.4.3. Infrastructure Leakage Index

One of the most widely used performance indicators for evaluating the extent of leakage from water distribution systems is the ILI. McKenzie and Seago (2005a) proposed that the ILI measures how effectively a water utility is managing real losses under its current operating pressure regime. Seago *et al.* (2007) furthermore explained that this indicator provides an indication of how serious the leakage occurring in a water distribution system or district metering area is compared to the theoretical minimum level of leakage that could be achieved. The ILI is defined as the ratio of the *current annual real losses* (CARL) to the *unavoidable annual real losses* (UARL), as presented by Eq. (1) (Lambert *et al.*, 1999).

$$ILI = \frac{CARL}{UARL} \tag{1}$$

where:

ILI	-	Infrastructure Leakage Index
$CARL$	-	Current annual real losses [m^3/yr]
$UARL$	-	Unavoidable annual real losses [m^3/yr].

McKenzie and Seago (2005a) highlighted the importance of understanding that the ILI calculation does not imply that pressure management is being optimally implemented in the system under consideration. The reasoning behind this statement is that it is usually possible to further reduce the volume of real losses (but not the ILI) through improved active pressure management. Since the ILI is simply a ratio (i.e. it has no units), it is regarded as a non-dimensional performance indicator for the current overall management of system infrastructure regarding leakage.

Thus, this indicator can be used for comparison between separate countries with different units of measurement. The higher the ILI, the greater the potential for further management of real losses. If a water distribution system has an ILI value of 3.0, for example, it means that the CARL is estimated as being three times as high as the expected minimum volume of leakage from the same system. Van Zyl (2014) stated that the definition of the ILI implies that an ILI value of 1.0 is the lowest that any water distribution system can practically achieve. The expected minimum volume of leakage is valid for the case where the relevant system is properly managed and well maintained.

Figure 2.3 illustrates the ILI values for 27 water utilities in South Africa. The ILI values for these South African utilities ranged from 2.1 to 15.6, with an average value of 6.3. McKenzie, Bhagwan, *et al.* (2002) proposed that an ILI value of below 2.0 would rarely be achieved for water utilities in South Africa. These authors added that values in the order of 5.0 would be relatively common and regarded as being representative of systems in a reasonable condition.

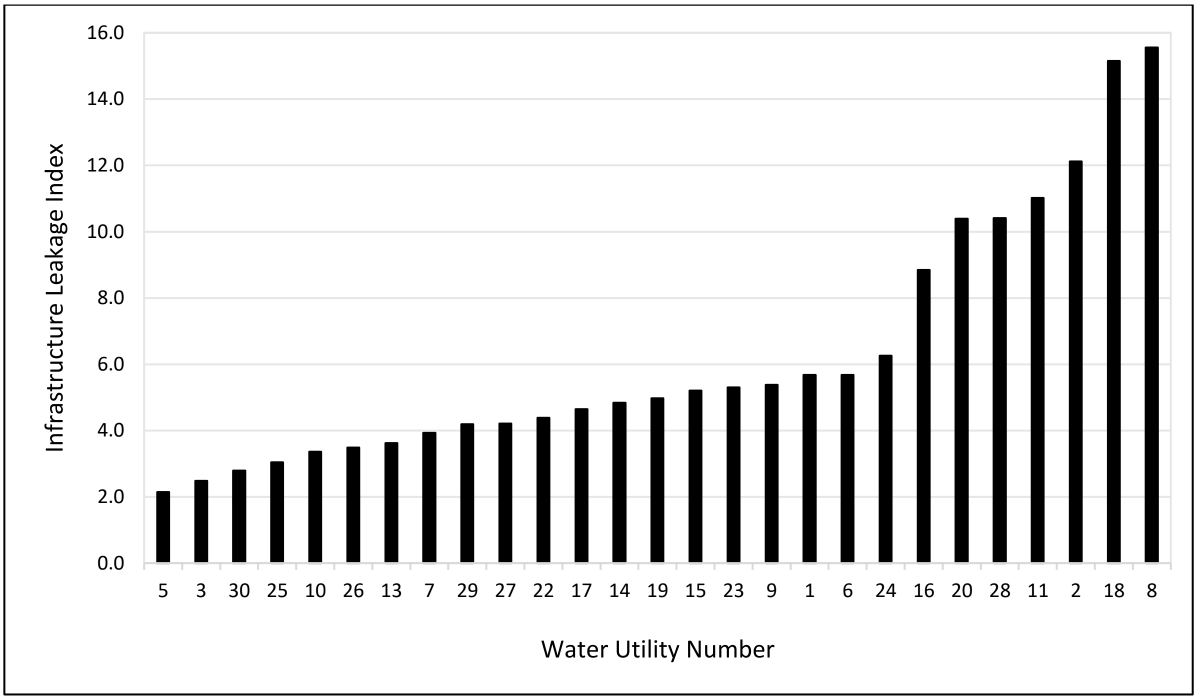

Figure 2.3: ILI results for 27 water utilities in South Africa (Seago *et al.*, 2007)

According to Lambert *et al.* (1999), South Africa had been one of the leading proponents in the use of the ILI as the main indicator for comparison of leakage levels between water utilities, since the year 1987. Although there had apparently been a strong sense of agreement between specialists on the usefulness of the ILI for the assessment of leakage, McKenzie *et al.* (2012) suggested that some water loss specialists in the South African municipal sector did, however, consider this indicator to be somewhat misleading at times. According to Seago *et al.* (2005), the basic simplicity of the ILI indicator had apparently often been criticised, as well as the fact that it did not incorporate some of the key factors that influenced leakage from water distribution systems.

2.2.4.4. Unavoidable Annual Real Losses

The minimum level of leakage that can theoretically be achieved for any water distribution system is defined as the *unavoidable annual real losses* (UARL). In theory, this level of leakage can be achieved if a system is in top physical condition; all reported leaks are repaired quickly and effectively; active leakage control is practised to reduce losses from unreported bursts; and there are no financial or economic constraints. The concept of UARL is one of the key developments that originated from the *Burst and Background Estimate* (BABE) methodology and the procedure to estimate the UARL was developed by Lambert *et al.* (1999). A more detailed discussion on the BABE methodology is provided in a subsequent section. Most of the BABE concepts are based on auditable assumptions, which were used by Lambert *et al.* (1999) to derive a formula for the UARL, as illustrated by Eq. (2).

$$UARL = \left(18\,L_m + 0.8\,N_c + 25\,L_p\right) \times P_{ave} \tag{2}$$

where:

$UARL$	-	Unavoidable annual real losses [ℓ/d]
L_m	-	Length of mains [km]
N_c	-	Number of service connections [connection]
L_p	-	Length of unmetered underground pipe [km]
P_{ave}	-	Average operating pressure at average zone point [m].

Upon inspection, it should be clear from Eq. (2) that unavoidable real losses are estimated for three separate components of infrastructure:

- Transmission and distribution mains (excluding service connections);
- Service connections - mains to street/property boundary;
- Private underground pipes between street/property boundaries and customer meters.

Seago *et al.* (2007) proposed that the third component could usually be ignored in the South African context, since customer meters in South Africa are generally located close to street edges.

2.3. Water Loss Management

2.3.1. Basic Leakage Management Activities

The broad concept of water loss management typically involves several basic leakage management activities that need to be implemented, to successfully prevent or reduce leakage rates from a water distribution system. It was concluded, from work undertaken by the IWA (Lambert *et al.*, 1999), that the following four leakage management activities (as presented in Figure 2.4) are the most important for constraining the increase in the annual volume of real loss:

- Pressure management;

- Active leakage control;
- Pipeline and assets management: selection, installation, maintenance, renewal, replacement;
- Speed and quality of repairs.

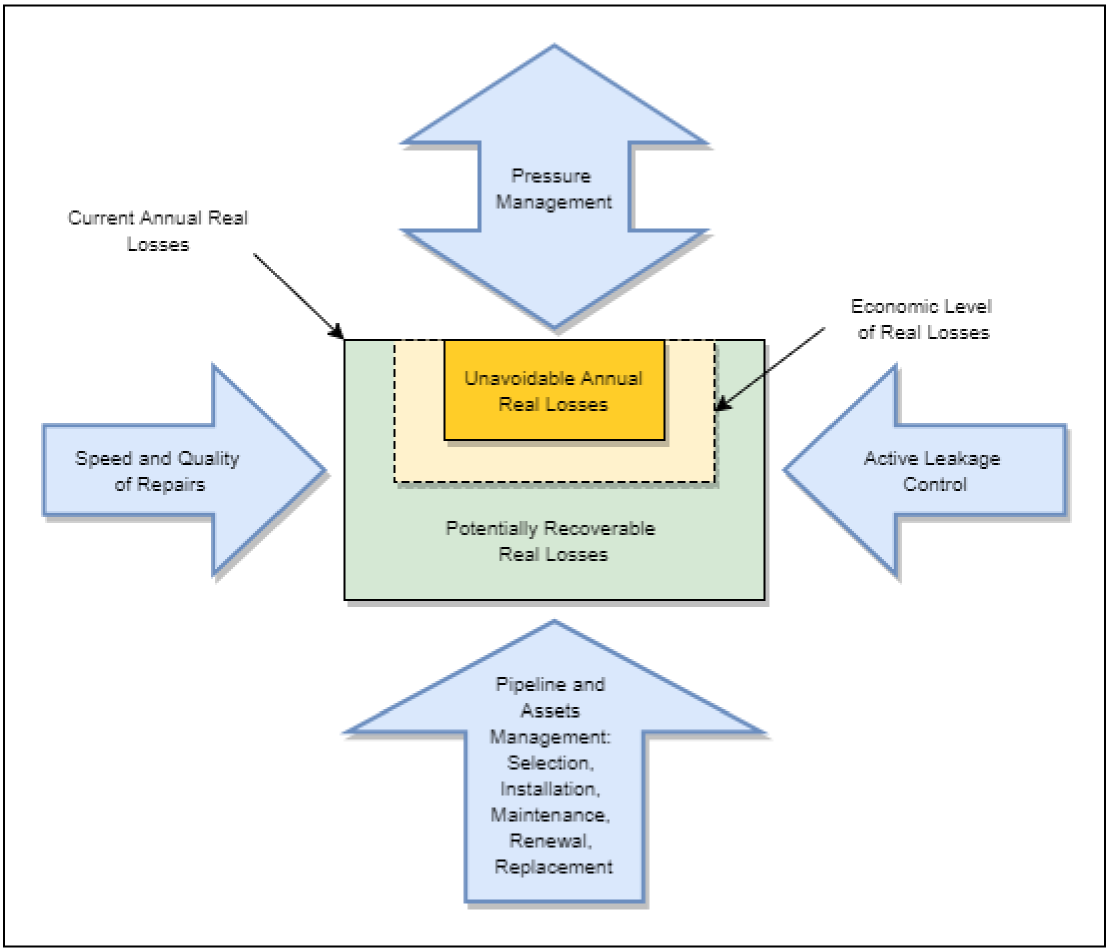

Figure 2.4: Four basic leakage management activities (McKenzie & Lambert, 2002)

2.3.2. Pressure Management and Assets Management

According to Thornton *et al.* (2008), pressure management and assets management (main and service line replacements) were the only procedures known to be available for reducing background leakage at the time. All unreported leakage and that which was undetectable using acoustic equipment was accordingly referred to as background leakage. Since assets management is usually very costly and often remains beyond the means of many water utilities, pressure management is typically considered to be the only practical and cost-effective method for reducing background leakage, once system infrastructure has already been installed (Mutikanga *et al.*, 2012).

As part of pressure management, a pressure-reducing valve is typically installed at the inlet of an isolated and metered pressure zone, which is generally referred to as a *pressure managed area* or *pressure managed zone* (Schwaller & Van Zyl, 2015). These authors further explained that a pressure-reducing valve is used to reduce excessive pressures in a water distribution system, particularly during the early morning hours when demand is at a minimum, and thus average system pressure is at a maximum.

Several real case studies (McKenzie *et al.*, 2004; Girard & Stewart, 2007; Babel *et al.*, 2009) have reported significant leakage reduction resulting from active pressure management. Although reduction in pressure through active pressure management cannot improve the condition of a distribution network (all leaks remain), it does, however, significantly reduce the rate of occurrence of new failures (Greyvenstein & Van Zyl, 2007). Lambert and Fantozzi (2010) furthermore stated that pressure management not only reduces leakage, but also: (1) extends the useful life of infrastructure; (2) decreases operation and maintenance costs through reduced frequency of main breaks and energy consumption; (3) improves customer service because of less water supply interruptions; and (4) is a demand management tool.

Mutikanga *et al.* (2012) suggested that although pressure management provided numerous benefits, the fact that it had not been generally implemented as a leakage control mechanism for most developing countries at the time, was due to two main reasons: (1) it was difficult to accurately predict the benefits associated with pressure management, which made the justification of certain investment decisions rather challenging; and (2) water distribution systems were typically not very well configured for effective pressure management.

2.3.3. Leakage Monitoring

According to Mutikanga *et al.* (2012), leakage monitoring basically involves the measurement of flow rates (and often pressures) into discrete zones or district metering areas. These authors also explained that the purpose of a leakage monitoring system is to continuously, or regularly, monitor flow rates into a district metering area, as well as to monitor and then analyse the minimum night flow into the district metering area. Leakage in the district metering area can then be identified as the excess flow beyond the legitimate customer use at the time of minimum night flow.

During the period of minimum night flow, which typically occurs between 2 a.m. and 4 a.m., the legitimate customer use is generally at a minimum, network pressures are relatively high, and leakage is typically at its maximum percentage of total inflow to the district metering area (Mutikanga *et al.*, 2012). These authors also suggested that the analysis of minimum night flow was the most widely used method in practice for the assessment of leakage.

Statistical analysis on flow rates is a further means used to assess leakage, and has been reported on by various separate researchers (Buchberger & Nadimpalli, 2004; Jankovic-Nišic *et al.*, 2004; Palau *et al.*, 2011). Although leakage monitoring methods and tools are widely implemented and very useful for prioritising zones with high leakage rates, they generally do not provide comprehensive information on how leakage is distributed spatially in a water distribution system (Mutikanga *et al.*, 2012).

2.3.4. Leakage Detection and Location

For field crews to be able to repair occurrences of leakage in a timely manner and thereby reduce water losses, application of some leak detection, or leak location techniques is necessary. The process of leak detection can be described as the narrowing down of a leak to some section of a pipe network, whereas the process of leak location refers to pinpointing the exact position of a leak.

Hartley (2009) explained that acoustic equipment such as listening devices, noise loggers, and leak noise correlators are typically used in leak detection surveys to determine the exact location of leaks. Clark (2012) and Hamilton (2012) showed that recent (at the time) advancements in technology and communication facilities had led to the development of more modern acoustic equipment, which was more efficient and less dependent on user experience.

For large-diameter pipes, it is furthermore possible to find leaks by tethered-in-pipe inspection and also through a number of types of wireless technology, which include video cameras, microphones, acoustic sensors, and smart balls (Stringer *et al.*, 2007; Wu *et al.*, 2011; Ong & Rodil, 2012). Some additional non-acoustic techniques such as tracer gas, infrared imaging, and ground penetrating radar were also proposed by Fanner *et al.* (2007) for locating leaks in water distribution systems. Fanner *et al.* (2007) properly documented the advantages and disadvantages associated with various leak detection and location equipment and technologies.

Network hydraulic modelling is another procedure that has been widely implemented, both in practice and by research institutions, for the prediction of leak sizes and location (Mutikanga *et al.*, 2012). According to these authors, hydraulic modelling can be used for various purposes relating to leakage management, which includes: (1) network zoning (Sempewo *et al.*, 2008; Awad *et al.*, 2009); (2) leakage modelling as a pressure-dependent demand (Almandoz *et al.*, 2005; Giustolisi *et al.*, 2008; Wu *et al.*, 2010); and (3) pressure management planning for leakage control (Ulanicki *et al.*, 2000; Tabesh *et al.*, 2009). However, although hydraulic modelling was regarded as an effective tool for leakage hydraulic analysis, (Savic *et al.*, 2009) pointed out that several model calibration challenges remained in practice.

2.4. Leakage Estimation Methodologies

2.4.1. The Effect of Pressure

Van Zyl and Clayton (2007) stated that pressure is regarded as one of the most significant factors influencing leakage from water distribution systems. The conventional view in the past has been that the leakage flow rate (discharge) from a pipe is a function of both the pressure head within the pipe and the area of the leak opening (orifice), as defined by the Torricelli orifice equation, Eq. (3). This equation is derived from the principle of conservation of energy and mathematically describes the conversion of potential energy to kinetic energy (Finnemore & Franzini, 2009).

$$Q = C_d A\sqrt{2gh} \tag{3}$$

where:

Q	-	Discharge [m^3/s]
C_d	-	Discharge coefficient
A	-	Orifice area [m^2]
g	-	Gravitational acceleration constant [m/s^2]
h	-	Pressure head [m].

According to Schwaller and Van Zyl (2015), some field tests found Eq. (3) to be unsuitable for describing the pressure-leakage response of pressure managed areas and district metered areas in general, which consequently led to the adoption of a more general equation, Eq. (4).

$$Q = Ch^{N1} \tag{4}$$

where:

Q	-	Discharge [m^3/s]
C	-	Leakage coefficient
h	-	Pressure head [m]
$N1$	-	Leakage exponent.

Several field studies that have been undertaken in the past by various researchers indicated that the value for $N1$ can be much higher than 0.5, as proposed by the Torricelli orifice equation (Wu *et al.*, 2011). Some laboratory and modelling studies (Walski *et al.*, 2006; Greyvenstein & Zyl, 2007; Cassa *et al.*, 2010) discovered leakage exponents ranging between 0.36 and 2.3. This wide range of exponents indicated that leakage is much more sensitive to pressure than was conventionally believed. A further study by Van Zyl and Cassa (2011) revealed that the $N1$ leakage exponent does not provide a good characterization of the pressure response of a leak, since different leakage exponents resulted for the same leak when measured at different pressures.

Van Zyl and Clayton (2007) specifically focused on the effects of pressure through four separate factors, which included: (1) leak hydraulics; (2) pipe material behaviour; (3) soil hydraulics; and (4) water demand. These authors concluded that a considerable proportion of leakage can consist of transitional flow, regarding leakage hydraulics, and typically has leakage exponents of between 0.5 and 1.0. Pipe material behaviour was also identified, through several experimental and theoretical investigations, as a significant contributing factor regarding the observed range of leakage exponents.

The study by Van Zyl and Clayton (2007) furthermore concluded that the interaction between a leaking pipe and its surrounding soil is extremely complex, and that it is influenced by many different conditions, which differ for individual leak occurrences. As a final deduction, the above-mentioned study established that the leakage exponent is most probably underestimated whenever water demand is present in minimum night flows.

2.4.2. Burst and Background Estimate

The BABE methodology was first developed in the mid-1990s by a task team comprising specialists from several privatised water companies in England and Wales (McKenzie & Seago, 2005a). According to these authors, the BABE techniques have been properly documented (UK Water Industry, 1994) and has ever since been widely accepted and adopted in many places throughout the world. A report by McKenzie and Seago (2005b) stated that many international water associations even recommended this approach to leakage management as the most systematic and pragmatic solution (at the time), because it had been so successful. Unfortunately, not much peer-reviewed (or published) literature could be found on the BABE methodology however.

The following were identified by McKenzie and Seago (2005a) as being some of the key issues that are covered by the BABE methodology: (1) breakdown of total losses into real and apparent losses; (2) influence of pressure on leakage and the $N1$ exponent; and (3) the use of component analysis to determine unexplained leakage from minimum night flow measurements.

Several South African water suppliers have accepted the BABE methodology and its concepts, according to the report by McKenzie and Seago (2005b), and these authors further suggested that South Africa had been regarded as one of the key players in this field worldwide at the time, through the efforts and initiatives of the Water Research Commission of South Africa. The user guide for a software tool, known as *SANFLOW* (McKenzie, 1999), stated that the BABE water balance approach was incorporated in much of the South African water legislation instituted at that time.

Four principal issues regarding leakage management were identified by the UK Water Industry (1994) during the development of the BABE techniques: (1) logging and analysis of minimum night flows; (2) pressure management; (3) water auditing and benchmarking of leakage; and (4) economics of leakage. The user guide for a further software tool, known as *ECONOLEAK* (McKenzie & Lambert, 2002), explained that each of these four issues had been addressed through the development of four self-contained computer models, of which both *SANFLOW* and *ECONOLEAK* form part. A more in-depth discussion on these four models is provided in a subsequent section.

According to McKenzie (2014), the BABE methodology is based on the theoretical concept that leakage in a water reticulation system can be classified into three separate categories: (1) background leakage; (2) reported bursts; and (3) unreported bursts.

Larger detectable events are referred to as bursts, while those too small to be detected or located are referred to as background leaks. The reported bursts are those with larger flow rates, which tend to cause problems and are therefore reported to the relevant water supplier. Unreported bursts, however, are defined as noteworthy events that do not necessarily lead to problems and that can only be found by means of active leakage control. Small undetectable leaks at joints and fittings are referred to as background leakage (McKenzie & Seago, 2005b).

It was suggested that a threshold figure of approximately 250 ℓ/h would be appropriate in South Africa to differentiate between distinct events being classified as either bursts or background leaks. Events that have flow rates of more than 250 ℓ/h are consequently defined as bursts, whereas events with flow rates lower than 250 ℓ/h are defined as background leaks. It is therefore possible to calculate the components that make up the annual volume of real losses by exclusively focusing on these three categories (McKenzie & Seago, 2005b).

2.4.3. Fixed and Variable Area Discharges

Application of the Torricelli orifice equation implies that a leak is assumed to have a fixed orifice area. Several laboratory and modelling studies (May, 1994; Greyvenstein & Van Zyl, 2007; Van Zyl & Clayton, 2007; Cassa *et al.*, 2010; Ferrante *et al.*, 2011; Massari *et al.*, 2012; De Marchis *et al.*, 2016; Fox *et al.*, 2016, Fox *et al.*, 2017) have however shown that the areas of real leak openings are generally not fixed, but rather varies with residual pressure head in most cases. Van Zyl *et al.* (2017) explained that changes in leak orifice area with pressure means that the conventional Torricelli orifice equation cannot accurately describe the flow through leak openings in real pipes.

The research conducted by May (1994) involved investigation into the effects of operating at different pressure levels, which ultimately led to the development of the *Fixed and Variable Area Discharges* (FAVAD) concept and FAVAD modified leakage equations (Cassa *et al.*, 2010; Van Zyl & Cassa, 2014). The FAVAD concept is particularly focused on the specifics regarding the hydraulics of leaks. Development of the FAVAD concept was undertaken because of the proposal made by May (1994) to make use of a combined leakage equation in the form of Eq. (5).

$$Q = k_1 h^{0.5} + k_2 h^{1.5} \tag{5}$$

where:

Q	-	Discharge [m^3/s]
k_1	-	Constant for openings of fixed area [m$^{2.5}$/s]
h	-	Pressure head [m]
k_2	-	Constant for openings of variable area [m$^{1.5}$/s].

Eq. (5) combines the theory of the well-known orifice equation presented as Eq. (3), which is particularly applicable to openings of fixed area, with the suggestion by May (1994), which states that leaks from flexible materials tend to have leakage exponents of 1.5.

May (1994) was therefore of the opinion that overall leakage from water distribution systems can be estimated by combining the theory for leaks of fixed area with the theory for leaks with variable area. Cassa *et al.* (2010) discovered that whenever linear elastic behaviour is assumed, the areas of distinct types of leak openings (round holes, longitudinal, circumferential, and spiral cracks) vary linearly with pressure, irrespective of the pipe dimensions, pipe material, or loading conditions. These authors proposed that the area of any leak undergoing elastic deformation can consequently be described as a function of pressure head, as presented by Eq. (6).

$$A = A_0 + mh \tag{6}$$

where:

A	-	Altered leak area [m^2]
A_0	-	Initial leak area [m^2]
m	-	Head-area slope [m^2/h]
h	-	Pressure head [h].

The FAVAD equation, as presented by Eq. (7), is obtained through substitution of Eq. (6) into Eq. (3). It should be quite clear that Eq. (7) has the same form as that of Eq. (5).

$$Q = C_d\sqrt{2g}(A_0h^{0.5} + mh^{1.5}) \tag{7}$$

where:

Q	-	Discharge [m^3/s]
C_d	-	Discharge coefficient
g	-	Gravitational acceleration constant [m/s^2]
A_0	-	Initial leak area [m^2]
h	-	Pressure head [m]
m	-	Head-area slope [m^2/h].

Cassa *et al.* (2010) however noted that, despite the similarity in form, there is an inherent difference between the two equations. This difference arises from the fact that, according to the FAVAD equation, all leaks are considered variable, whereas the combined leakage equation by May (1994) proposes that leaks can be considered as either fixed or variable. In other words, the FAVAD equation simply proposes that all leaks will increase in area with increasing pressure.

2.5. Software Models Available for Estimating Leakage

2.5.1. Background Night Flow Analysis Model (SANFLOW)

SANFLOW is one of the various computer programs that was developed through the Water Research Commission of South Africa and had officially been released in August 1999. According to the *SANFLOW* user guide (McKenzie, 1999), this computer model was originally developed with the specific objective of assisting water suppliers in determining the extent of leakage for discrete zone-metered areas, through the analysis of recorded minimum night flow data. The *SANFLOW* user guide also explains that measurements on the minimum night flow into a zone-metered area is a simple and effective technique for determining whether a water supplier has a serious leakage problem. Minimum night flow can be identified from the normal inflow to a zone-metered area as the lowest flow entering the zone at any specific moment and typically occurs between midnight and 4 a.m. for most zones (McKenzie, 1999). An example of inflow to a zone-metered area is presented in Figure 2.5, which also indicates the level of minimum night flow.

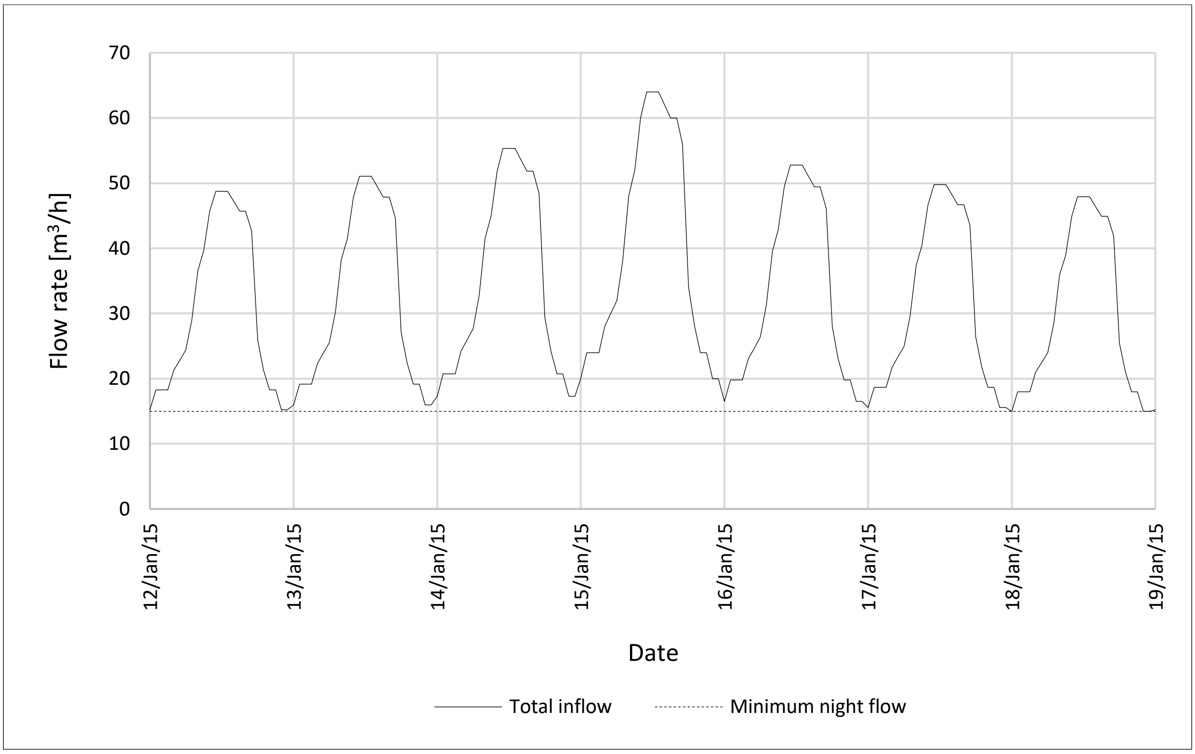

Figure 2.5: Example of inflow to a zone-metered area that illustrates minimum night flow

The *SANFLOW* user guide furthermore suggests that by making use of general BABE principles, minimum night flow can be split into various components, which are calculated separately by the *SANFLOW* model and include: (1) normal night use, (2) background leakage, and (3) pipe bursts. Normal night use is further subdivided into normal domestic night use, small non-domestic night use, and larger non-domestic night use.

Background leakage is, similarly, subdivided into background leaks from mains, background leaks from connections, and background leaks from installations. The difference between the total night flow and the sum of the normal night use and background leakage is assumed to make up the component corresponding to burst pipes.

The methodology used in the *SANFLOW* model can be defined as an empirical approach, which is based on numerous test results, from both the United Kingdom, and some other parts of the world. This methodology has, however, been implemented with remarkable success in many different countries worldwide. Further research has nevertheless been recommended by (McKenzie, 1999) in the *SANFLOW* user guide, in order specifically to establish parameter values for South African conditions.

2.5.2. Pressure Management Model (PRESMAC)

In 1999, a project was initiated by the Water Research Commission of South Africa to develop a South African pressure management model as part of the greater strategy to promote water conservation (McKenzie & Langenhoven, 2001). This model is referred to as *PRESMAC* and it was created primarily for assessing the likely savings (in monetary terms) that could be achieved regarding leakage, by implementation of certain pressure reduction options in a zone-metered area. Such pressure reduction options included fixed-outlet and time-modulated pressure reducing valves. Users of *PRESMAC* are thereby enabled to efficiently evaluate the potential for pressure management, without having to perform a complete and detailed pipe network analysis.

According to the *PRESMAC* user guide (McKenzie & Langenhoven, 2001), some basic information regarding a water distribution system is required from users for them to make use of the *PRESMAC* model. This information specifically includes system parameters such as: number of service connections; length of mains; number of properties; population; expected leakage rates from service connections, properties and mains; and the pressure exponent for the system. Three 24-hour pressure profiles, as well as the 24-hour zone inflow profile, are furthermore required, in addition to the above-mentioned basic information. More specifically, the average hourly values for the pressures at the inlet point, average zone point, and critical point are needed, as well as the average hourly values for total inflow to the zone. Flow and pressure loggers are generally used to obtain these four sets of hourly values (McKenzie & Langenhoven, 2001).

The *PRESMAC* user guide explains that the program allows users first to analyse the existing situation of any specific pressure management area. *PRESMAC* then provides an additional option for assessing the savings that are likely to be achieved through the installation of new pressure reducing valves, or by resetting existing pressure reducing valves to lower pressures. Furthermore, the above-mentioned user guide states that the potential savings that could be achieved using a time-modulated controller can also be assessed when making use of *PRESMAC*. According to the *PRESMAC* user guide, time-modulated controllers, which were introduced to South Africa in 1999, had been considered the simplest, least expensive, and most widely used controllers available on the market at the time.

2.5.3. Benchmarking of Leakage Model (BENCHLEAK)

The *BENCHLEAK* model is a software tool that was developed through the Water Research Commission of South Africa, to facilitate water utilities with the evaluation of leakage and non-revenue water in their water distribution systems (McKenzie, Lambert, *et al.*, 2002). These authors also explained that *BENCHLEAK* is a simple user-friendly model that is operated in a *Microsoft Office Excel* spreadsheet environment, which delivers a variety of performance indicators regarding non-revenue water and real losses. The *BENCHLEAK* program also provides clear definitions of all components of the standard water balance and thereby facilitates annual water balance calculations. According to McKenzie, Bhagwan, *et al.* (2002), several water utilities have developed their own versions of *BENCHLEAK* since its release, to obtain first order estimates of real losses and non-revenue water in their water distribution systems.

BENCHLEAK requires a certain number of input parameters, which would typically be provided by the user of the software. Seago *et al.* (2007) explained that some of the necessary input parameters for the *BENCHLEAK* program include: length of mains; number of service connections; average operating pressure; population; system input volume; components of authorised consumption; and monetary values for real- and apparent losses. These authors furthermore explained that the model then performs some calculations, after which it provides the user with several useful output parameters, which include: UARL; apparent losses; CARL; and ILI.

2.5.4. Economics of Leakage Model (ECONOLEAK)

ECONOLEAK was released in 2001 and concluded the line-up of computer models that have been developed through the Water Research Commission of South Africa to assist water suppliers in evaluating and managing leakage from their water distribution systems. The *ECONOLEAK* user guide (McKenzie & Lambert, 2002) explains that this model was created with the specific purpose of aiding water suppliers in identifying how often active leakage control measures should be implemented for a specific zone-metered area. Using *ECONOLEAK*, the volume of water lost through leakage from a water distribution system can effectively be estimated for three separate scenarios, which include the undertaking of appropriate measures either every 6, 12, or 24 months (McKenzie & Lambert, 2002). These three scenarios are the proposed time intervals for undertaking an active leakage control intervention, which is essentially a full-scale leak detection and repair programme.

A basic framework and explanation of the methodology for assessing the economic factors associated with leakage and leakage control is also provided by *ECONOLEAK*, according to the *ECONOLEAK* user guide. Through the above-mentioned framework, water suppliers are enabled to develop their own financial models to address specific issues, by using their own data and individual circumstances. The above-mentioned user guide further states that the financial implications of various possible water demand management strategies can be assessed easily once the *ECONOLEAK* model is set up for a specific water supply area. The benefits of reducing the repair times of reported bursts or increasing the number of leak detection teams, for example, can be evaluated by users. *ECONOLEAK* therefore enables water suppliers to identify the most cost-effective methods of reducing leakage from their systems (McKenzie & Lambert, 2002).

The *ECONOLEAK* user guide states that to effectively assess the economics of leakage in a water supply system, a considerable amount of factual system information is required. Since very few water suppliers in South Africa could provide the necessary data at the time, the *ECONOLEAK* model was further regarded as useful for generating awareness of some key data aspects that needed to be captured and monitored by water suppliers on a continuous basis (McKenzie & Lambert, 2002). Most of the information required by *ECONOLEAK* is the same as that used for a minimum night flow analysis or pressure management analysis. Some additional forms of data are, however, necessary and these include the marginal cost of water, as well as costs associated with leakage detection and repair (McKenzie & Lambert, 2002).

The *ECONOLEAK* model complements the *SANFLOW*-, *PRESMAC*-, and *BENCHLEAK* models. Together, these four models provide water suppliers with four key tools that can facilitate a standard approach to water leakage for use in South Africa. The four models are independent of one another, although they often require the same input data.

2.6. Allocating Water Loss to Hydraulic Models

2.6.1. Introduction to Existing Techniques

People working in the water supply industry (e.g. consulting engineers) are often faced with the complex problem of correctly allocating water loss in hydraulic models. Depending on the personal preference of the relevant user, and the software options available, there are several different techniques for effectively modelling the effects of water loss in water distribution systems. For many of the existing computer programs, one of the most commonly implemented techniques involves the allocation of water losses to the nodes of the relevant hydraulic model. Since standard consumption by consumers is usually assigned as the output value to each of the nodes of a hydraulic model, it simply seems to make sense to allocate water loss to the nodes as well.

The spatial distribution of water loss within a hydraulic model is a further key aspect in respect of which individual entities have their own methods of practice. Water loss can be distributed spatially in a variety of separate ways, among some of which include: (1) consumption-dependent distribution; (2) weighted distribution; and (3) pressure-dependent distribution.

2.6.2. Consumption-Dependent Distribution to Nodes

For a consumption-dependent distribution it is assumed that water loss can be estimated as a certain percentage of the consumption at each node (Compion, 2013). This percentage value is first calculated as the total volume of losses divided by the total volume of consumption in the system, after which it is applied to the corresponding volume of consumption at each individual node, to ultimately obtain the appropriate volume of water loss at each node. In other words, the water loss volume allocated to each node is exclusively dependent on the level of consumption at the node itself, and thereby assumed to be unrelated to any other possible contributing factors.

Loubser (2015) stated that the general practice at one consulting firm in South Africa, *AECOM*, for allocating losses to nodes was to increase the output of each node by an appropriate percentage. However, Loubser (2015) also added that water loss is not often modelled at all at *AECOM*. The reason for this is that a large measure of uncertainty is usually associated with the values for annual average daily demand and peak factors, which means that attempts at modelling water loss specifically, is in many cases not believed to be feasible. According to Compion (2013), the engineers employed at *GLS Consulting* generally also made use of a consumption-dependent distribution for allocating water loss to hydraulic model nodes.

The consumption-dependent distribution, for which water loss at a node is assumed as a certain percentage of the consumption at that node, is possibly the most common and widely implemented technique for allocating water loss to hydraulic models.

2.6.3. Weighted Distribution to Nodes

Implementation of a weighted distribution is another possible technique for spatially allocating water loss to nodes in a hydraulic model of a water distribution system, and this technique has also been used in the past by practising engineers (Compion, 2013). The above-mentioned concept involves the use of certain weighting factors for assigning relative measures of weight to the various nodes in the model, which in effect describes the level of the contribution of each node to the total loss of water in the system. In theory, the weighting factors could be linked to any type of contributing factor that is expected to influence water loss in the system. A few examples of such contributing factors include: pressure, pipe diameter, pipe length, pipe material, and the spatial density of service connections.

An example of using weighting factors is introduced as follows: Suppose a range of residual pressure heads (applicable at system nodes during peak flow) were obtained as part of the results of steady state analysis that had been performed on a hydraulic model. A decision was then made that 75% of the total volume of water loss was to be distributed equally among nodes that exhibited pressure values above that of the average system pressure. The other 25% of total water loss was to be distributed equally among the remaining nodes, which exhibited pressure values below that of the average system pressure.

This very basic example successfully illustrates the principle of using a weighted distribution for spatially allocating water loss within a water distribution system. The use of weighting factors can, however, be more complex in cases for which several types of contributing factor are required to be accounted for.

2.6.4. Pressure-Dependent Distribution to Nodes

A pressure-dependent distribution is a third technique for subdividing the total volume of water loss between relevant system nodes. Whenever this technique of distribution is used, it is, in effect, being assumed that leakage is the primary component of the total volume of water lost, since the extent of leakage is very closely related to pressure.

The most common approach for implementing this type of distribution is through application of Eq. (4), as presented in a previous section on the effect of pressure, at each relevant node of the hydraulic model (Lambert & Fantozzi, 2010). Selection of appropriate values for the leakage coefficient, C, and leakage exponent, $N1$, is the responsibility of the analyst.

Du Plessis (2015) stated that *Civil Designer* (Knowledge Base Software, 2017) is used, amongst others, by Aurecon, South Africa, to design and analyse water distribution systems. According to Du Plessis (2015), the water related module of *Civil Designer* provides the option for specifying unit loss (volume of loss per unit length of pipe) for an entire network, sections of a network, or for individual pipes. Unit losses are then multiplied by the associated pipe lengths, before finally being concentrated (lumped) at system nodes in the form of loss factors (comparable to leakage coefficients). A water loss equation, which is theoretically identical to the one presented as Eq. (4), is subsequently implemented at each node of the hydraulic model during hydraulic analyses. Furthermore, when making use of *Civil Designer*, the leakage exponent, $N1$, as used in Eq. (4), can be altered from its default value of 0.8 (as per the software) if deemed appropriate by the analyst.

2.7. Existing Segregation Technique

This section merely provides an overview of the existing segregation technique by Almandoz *et al.* (2005), since a large portion of their work forms the basis for this research study and is, therefore, greatly incorporated in the content of this research study. This section also serves as a formal introduction to the work by Almandoz *et al.* (2005), since many references have been made to their study in the subsequent sections of this research study.

The earlier study by Almandoz *et al.* (2005) introduced a technique whereby leakage (real loss) in a water distribution system can be segregated from all other types/components of non-metered water. This technique is based on the comparison of temporal profile patterns of the total input flow rate curves to discrete water supply zones / district metering areas. Almandoz *et al.* (2005) proposed a technique by which all volumes of *uncontrolled water* in a distribution system are recognised as either: (1) leakage from mains and service connections; or (2) non-metered consumption. An estimate of the fractional split between leakage and non-metered consumption is thereby obtained by implementing the technique of these authors.

The term *uncontrolled water*, as referred to by Almandoz *et al.* (2005), is defined as the difference between the total input volume to a water distribution system / district metering area, and the total volume of metered consumption in the system / metering area. Unfortunately, the IWA standard water balance does not provide a single and definite alternative term for *uncontrolled water*. This term is, however, synonymous with the total non-metered water in a water distribution system.

2.8. Discussion of Useful Findings

Several useful findings have been discovered as part of the literature review for this research study and are briefly discussed in this section. The implications of these findings for the relevance of this research study are also explained in short.

The first piece of useful information regards the structure of the standard IWA water balance, and the way in which the different components that make up the overall balance are classified. This information was considered useful, since an alternative simplified water balance has been developed as part of this study, which required some form of theoretical platform. Secondly, an improved understanding was achieved of the performance indicators that are generally used to measure the significance of water loss. The problems associated with the use of percentages became clear, which provided further support for the procedure that was evaluated and further developed as part of this study, since this procedure involves a technique that specifically does not make use of percentages for aspects of leakage.

Some useful leakage estimation methodologies also proved enlightening, as a third useful aspect of the literature reviewed, with specific reference to the effects of pressure on leakage flow rates. Since an emitter type of flow has been incorporated as part of the procedure developed in this study, it was deemed important to fully understand the hydraulics involved in such a type of flow, which is why the theory associated with the leakage estimation methodologies was regarded valuable.

A few software models that are currently available for the estimation of leakage from water distribution systems were reviewed briefly, which ensured a proper awareness of existing techniques and tools for estimating leakage using computer software. Another aspect of the literature review that was considered valuable, was that some techniques that are currently used in practice for allocating water loss to hydraulic models were investigated. This process of investigation proved to be rather useful, since the technique that has been developed as part of this study involves a combination of the aforesaid series of techniques.

The final, and probably most useful, aspect of the literature reviewed was the existing technique by Almandoz *et al.* (2005), which introduced an approach by which leakage (real loss) may be segregated from all other types of non-metered water in a hydraulic model of a distribution system. Since the focus of this research study was also very much on the segregation of leakage from other non-metered water components, in advance to the process of performing hydraulic analyses, the author of this study considered the work by Almandoz *et al.* (2005) to be valuable.

3. RESEARCH METHODOLOGY

3.1. Overview

This chapter presents the methodology of this research study, which involved: (1) the conducting of an extensive literature review on the topic of losses from water distribution systems; (2) investigation and further development of an already existing methodology regarding the estimation of water loss in hydraulic models; (3) identification and selection of a possible real-world case study scenario for application and evaluation purposes; (4) data acquisition and software selection for the purposes of performing analyses and obtaining results; (5) application of the further developed water loss estimation methodology through software simulations; (6) analysis of results and drawing conclusions.

3.2. Theoretical Foundation

As explained before, an existing procedure was investigated and slightly modified through further development as part of this research study. Almandoz *et al.* (2005) proposed a technique by which all volumes of *uncontrolled water* in a distribution system are recognised as either: (1) leakage from mains and service connections; or (2) non-metered consumption. An estimate of the fractional split between leakage and non-metered consumption is thereby obtained by implementing the Almandoz *et al.* (2005) technique.

It should be noted that the technique is, however, applicable only when making use of extended period simulation on models of water distribution systems, and that accurate flow logging data is a requisite. The approach of the procedure that was followed as part of this research study is somewhat different from the one proposed by Almandoz *et al.* (2005), in the sense that the water balance technique of the former corresponds more closely to that of the IWA water balance.

3.3. Case Study Application

The technique that was investigated and further developed as part of this research study was applied to and tested on a real-world case study. Application of the procedure to the well-known hypothetical *Anytown* problem (Walski *et al.*, 1987) was initially considered for the purposes of demonstrating the practical application of the procedure. Fortunately, some flow logging data from a real-world case study was acquired and used instead. The data quality was excellent and, therefore, proved to be useful for application purposes. During the process of identification and selection of an appropriate case study scenario, certain eligibility criteria were set:

- A comprehensive set of continuous and uninterrupted flow logging data for a discrete reservoir supply zone or district metering area was needed.
- The flow data had to be logged at a frequency rate of at least one hour or greater, since hourly peak factors were to be generated from this data as part of the overall procedure.

- Metered consumption records for all consumers imposing demands on the reservoir supply zone or district metering area were required, to successfully establish a basis of comparison between the measured consumption within the distribution system and the logged supply to the system.
- An accurate hydraulic model of the real-world water distribution system was necessary, as well as the corresponding software package by which hydraulic simulations and analyses could be performed.

3.4. Software Requirements

3.4.1. Wadiso

The first computer software package that was necessary for successful implementation of this methodology is titled *Wadiso*, and is both introduced and discussed briefly. *Wadiso* is described by the relevant developer company (GLS Software (Pty.) Ltd., 2017a) as a comprehensive computer program for the analysis and optimal design of water distribution networks. Since its initial development, *Wadiso* has been extended and improved substantially regarding its user-friendliness, speed, and interfacing with other sorts of application software, according to GLS Software (Pty.) Ltd. (2017a). Further development of *Wadiso* was undertaken by a team of computer software developers, employed by a local South African firm, *GLS Engineering Software (Pty.) Ltd.*

Wadiso effectively integrates four separate modules, which have different purposes of analysis, with graphical displays of data. These four modules include: (1) steady state analysis; (2) extended period simulation; (3) optimisation; and (4) water quality analysis. The combination of the above-mentioned modules with the graphical displays of data effectively results in a single, most valuable tool for water engineers (GLS Software (Pty.) Ltd., 2017a). Since *Wadiso* is a relatively well-known computer program among many practicing engineers in the water engineering industry of South Africa, the validity of the results obtained was anticipated to be reliable.

Wadiso includes a seamless interface to the public domain *EPANET* program module for all modelling aspects, according to GLS Software (Pty.) Ltd. (2017a). The *EPANET* engine includes the option to model emitter types of flow at hydraulic model nodes (particularly useful for simulating sprinklers). The flow rates of these emitters are governed by the residual pressure head at the relevant nodes, the emitter coefficients allocated to the nodes, and the single global emitter exponent that is valid for the entire system, according to Eq. (4), which has been presented in Chapter 2. The global emitter exponent is unfortunately not adjustable among separate nodes in the current versions of *EPANET* and *Wadiso*. This emitter functionality is available for use to model leaks in a water distribution system as a function of residual pressure head at model nodes.

3.4.2. Microsoft Office Excel

The second computer software package that was necessary for successful implementation of this methodology is rather well-known as *Microsoft Office Excel*, and is also introduced and discussed briefly. *Microsoft Office Excel* is a recognized and popular computer program, which is used by countless individuals around the world in separate fields of expertise. This program provides numerous powerful features and tools in a spreadsheet type of environment, which enables a user to analyse and manage large sets of data, as well as to perform advanced calculations on such data, in a timely manner.

Microsoft Office Excel was selected for the purposes of preparing, analysing, and storing of the necessary input data for this research study, as well as to store and manage the results that were obtained from hydraulic analyses performed. The spreadsheet environment of *Microsoft Office Excel*, furthermore, proved to be very useful for performing tedious and repetitive calculations in a time-efficient manner, which was very much necessary for this methodology to be practically viable.

3.5. Data Acquisition

3.5.1. Flow Logging

A large set of flow logging data was used for this research study to evaluate and implement the modelling procedure that was investigated and further developed during the study. This data set was specifically used as part of the input parameters to a hydraulic model of a real-world water distribution system. A web-based data acquisition and display system, known as *Zednet*, was used to acquire all the necessary flow logging data. *Zednet* was developed by *WRP Consulting Engineers* to effectively manage infrastructure, as well as to identify and reduce losses from water distribution systems (WRP (Pty.) Ltd., 2017a). *WRP* has installed numerous GSM and GPRS loggers across the *City of Tshwane Metropolitan Municipality*, which automatically transmit logging data directly to several computer servers, after which it is published immediately on the web-based *Zednet* system (WRP (Pty.) Ltd., 2017b).

During the time of this study, the author was employed full-time as an engineering graduate at *GLS Consulting*, which is a civil engineering firm in Stellenbosch, South Africa. The author was therefore fortunate enough to have complete access to the *Zednet* database, from which all the temporal flow logging data that was used in this research study was acquired. Selection of an appropriate data set was done based on the reliability of the logged data, regarding the following categories of selection criteria: (1) continuity (i.e. prevalence of gaps and/or spikes); (2) record length; (3) consistency (in terms of the temporal profile pattern); (4) size of the distribution zone / district metering area; and (5) number of different land-use categories represented within the distribution zone / district metering area.

The measures/limits of acceptability that were set for each one of the above-mentioned categories of selection criteria are listed in Table 3.1.

Table 3.1: Measures/limits of acceptability for selection criteria

No.	Selection criteria	Measure/limit of acceptability
1	Continuity	Very few gaps or spikes in the data set / preferably zero
2	Record length	Greater or equal to 1 month
3	Consistancy	Consistent temporal profile pattern from one day to the next
4	Size of distribution zone / district metering area	As small as possible / AADD of less or equal to 1 000 kℓ/d
5	Number of different land-use categories	Lowest number of different land-use categories possible / preferably only one

3.5.2. Hydraulic Model

For the purposes of evaluation of the methodology that was developed during this research study, a hydraulic model of the same real-world water distribution system, for which flow logging data was obtained, was sought after. Since *GLS Consulting* particularly specialises in the use of hydraulic models of real-world water distribution and sewer systems, as part of their consulting services, the company has created and maintained many such models for numerous regions throughout South Africa up to the time of this study. These models have also already been populated with most of the necessary variables that could possibly have an influence on the results obtained from hydraulic analyses performed on the models, for example: loads applied at nodes/manholes; hydraulic properties of existing pipes and other related system infrastructure; and topographical information (i.e. elevations of system infrastructure).

The author was once again fortunate enough to have complete access to and use of the exact hydraulic model that corresponds to the flow logging data that was acquired from *Zednet*. An already existing *Wadiso* water network model file, which is a custom file format that was developed by *GLS Engineering Software* and is readable exclusively by the *Wadiso* software package, was used as the appropriate hydraulic model of the real-world water distribution system that needed to be analysed.

3.5.3. Swift

As mentioned in the previous section, the loads that are applied to the nodes of hydraulic models are important for the purposes of performing accurate simulations that are sufficiently representative of the actual conditions in the field. It is for this reason that *GLS Consulting* makes use of yet another software package, known as *Swift*, to accurately populate their water and sewer models with the latest treasury data regarding water consumption and sewage production that are being imposed on existing systems by different types of customers.

According to (GLS Software (Pty.) Ltd., 2017b), *Swift* is a computer program that performs statistical analyses of data in municipal databases and provides important information to the municipal infrastructure manager. The GLS Software (Pty.) Ltd. (2017b) webpage furthermore states that *Swift* also provides a link between sewer and water distribution models, for accurate modelling of water demand or effluent production.

As part of the procedure that was evaluated/developed during this study, the author made use of *Swift* data, which represents measured consumption/production by customers, to simulate demand scenarios with which to compare the logged supply flow against. A more detailed discussion of the exact way by which this was done will be provided in a subsequent section.

3.6. Analysis and Results

All hydraulic analyses for this methodology are performed in the *Wadiso* software package. A typical analyses procedure involves several extended period simulation trials that performed in a sequential manner. For each simulation trial the value of a specific variable is adjusted. This is done to achieve an equilibrium between two variables representing total daily input volumes. All the aforesaid variables are introduced and explained in the next chapter. After an equilibrium has been established between the two volumes, as mentioned above, the value for the original variable is accepted as correct and a single simulation trial is complete.

When enough simulation trials have been performed for a selected number of trial values for yet another variable, the temporal profile patterns of some simulated curves are compared with that of the measured curve, which originates from real-world data. This comparison is done to determine the appropriate split between leakage and all other types of non-metered water. Once this split has been determined the goal of this methodology has been achieved. All details regarding the performing of analysis and the obtaining of results are explained in greater depth in Chapter 4 and then demonstrated in Chapter 5.

3.7. Framework

A schematic representation of the framework for the research methodology is presented in Figure 3.1. This figure clearly illustrates the way in which the various components of the methodology are linked together in a sequential process, which ultimately results in the conclusions reached after this research study.

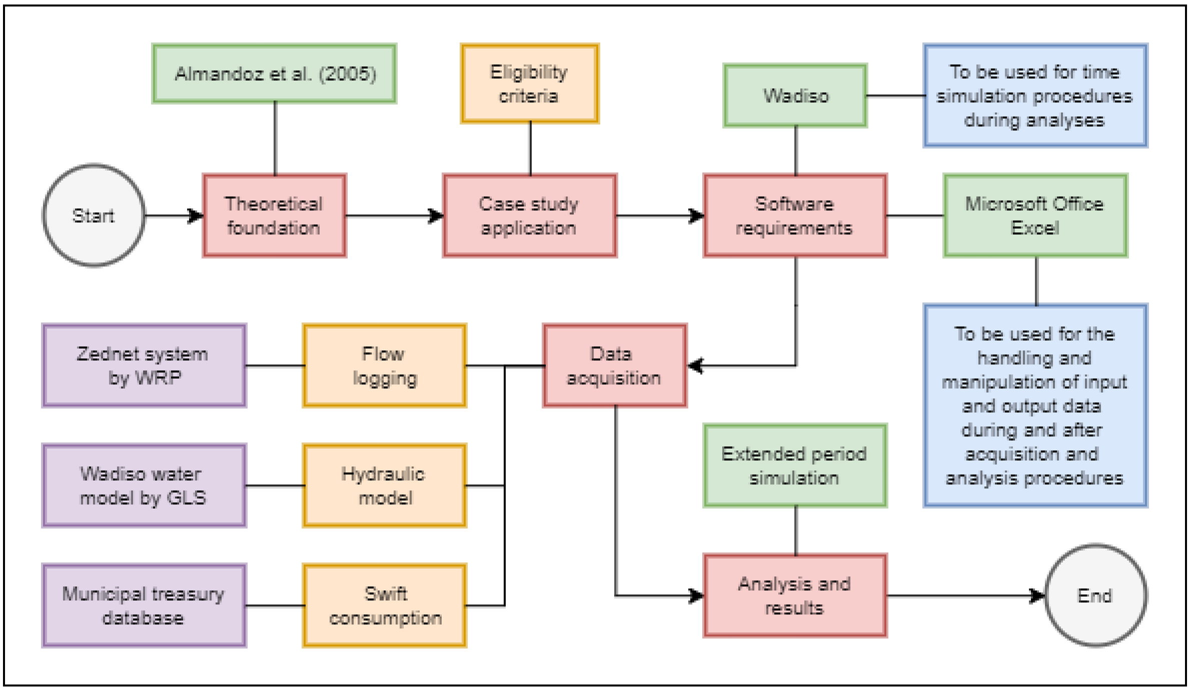

Figure 3.1: Methodology framework

4. SEGLEAK: A PROCEDURE FOR SEGREGATING LEAKAGE FROM OTHER TYPES OF NON-METERED WATER

4.1. Overview

As stated before, some tools are available to allocate metered water consumption to hydraulic model nodes (Jacobs & Fair, 2012). The process of correctly allocating non-metered water components in hydraulic models often proves to be much more challenging. This chapter introduces a procedure that attempts to aid in the aforesaid regard. The procedure is based on the segregation of real losses from all other types/components of non-metered water. Real losses are subsequently modelled as a function of pressure head, whereas the remainder of the total non-metered water is then distributed spatially among all model nodes having consumption values associated with it, according to a percentage value of the consumption at the related nodes.

The *SEGLEAK* procedure therefore serves as a computer-based technique for achieving increased accuracy regarding the modelling of non-metered in water distribution systems. It should be noted that, although the other components of non-metered water (i.e. apparent loss, non-metered authorised consumption) are also modelled to some extent through this procedure, the focus of the technique is, however, on leakage (real losses).

The already existing procedure by Almandoz *et al.* (2005) that was investigated, further developed, and then applied to a case study problem, as part of this research study, is fully described in this chapter. A simplified water balance that was created specifically in support of the procedure is presented and discussed briefly. The modelling approach to the *SEGLEAK* procedure is explained in the third part of this chapter, which includes: input parameters, fundamental logic formulations, output parameters, and the allocation of water loss to nodes in hydraulic models. For the fourth part of this chapter, some detailed theory and equations are presented regarding the calculations that are performed during analysis procedures, when making use of the *SEGLEAK* approach. Finally, the simulation aspect of *SEGLEAK* is explained and demonstrated to some extent, by making use of some examples.

It is important to understand that all the above-mentioned theory, equations, and procedures of analysis have either been developed as part of this research study itself, or is based on the earlier work by Almandoz *et al.* (2005). Since a large portion of the work by Almandoz *et al.* (2005) has been used, but also modified during the development of the *SEGLEAK* approach, it is difficult to clearly distinguish between the work of Almandoz *et al.* (2005) and the work that originated from this study specifically. An attempt has however been made to clarify the origin of most theories, concepts, equations, and analysis procedures as far as possible, to avoid potential confusion regarding this matter.

4.2. Simplified Water Balance

As mentioned before, a water balance structure that is slightly different to the one proposed by Almandoz *et al.* (2005) was developed and used for the classification of the separate water components, as part of the *SEGLEAK* procedure. It is important to note that this water balance has been developed as part of this research study, but is largely based on the water balance by Almandoz *et al.* (2005), of which an adapted version has been illustrated before in Figure 2.2, as part of Chapter 2.

The goal of the alternative water balance that has been developed as part of the *SEGLEAK* approach is to properly represent the basic concepts of the well-known IWA water balance, while also following a more technical approach, rather than a purely managerial one. In other words, the water balance does not focus much on whether there is revenue associated with the distinct water components, but rather on whether the ultimate destination of the water is known. The simplified water balance, which was created specifically for the purposes of the *SEGLEAK* procedure, is illustrated accordingly in Figure 4.1.

<table>
<tr><td rowspan="5">Total system input Q_i</td><td>Metered Q_m</td><td>Metered authorised consumption Q_{mac}</td><td>Domestic consumption, commercial consumption, industrial consumption, official consumption, etc.</td><td>Accounted-for water</td></tr>
<tr><td rowspan="4">Non-metered Q_n</td><td>Non-metered authorised consumption Q_{nac}</td><td>Billed at a fixed rate or non-billed (e.g. public taps, schools, hospitals, public parks, system flushing, etc.)</td><td rowspan="4">Water loss</td></tr>
<tr><td rowspan="2">Non-metered apparent losses Q_{nal}</td><td>Unauthorised consumption (e.g. theft)</td></tr>
<tr><td>Consumer meter inaccuracies</td></tr>
<tr><td>Non-metered real losses Q_{nrl}</td><td>All forms of physical leakage from mains, temporary storage facilities, and service connections</td></tr>
</table>

Figure 4.1: Simplified water balance

4.3. Modelling Approach

4.3.1. Known Input Parameters

4.3.1.1. Measured Total Input Flow Rate

The first known input parameter required is the temporal profile of the measured total input flow rate, $Q_i^m(t)$, to the sector under consideration during a single day period. A sector, in this sense, can comprise either an entire water distribution system, or simply a distinct district metering area. This implies that all entry mains to the relevant sector need to be fitted with properly calibrated flow meters, to ensure accurate measurement of total input flow rates to the sector itself.

4.3.1.2. Daily Average Metered Consumption

Secondly, the daily average metered consumption flow rate at each node, $(\bar{Q}_{mc})_j$, in the hydraulic model is required. Since most municipal treasury databases typically comprise comprehensive volumes of data regarding measured consumption values for individual consumers, this data could be obtained and used with a reasonable measure of certainty.

4.3.1.3. Total Daily Metered Consumption Volume

The third known input parameter required is the total daily volume of metered consumption, V_{mc} , within the relevant sector. The value for V_{mc} is calculated simply by taking the sum of all the $(\bar{Q}_{mc})_j$ values and converting it from an instantaneous flow rate to a total daily volume.

4.3.2. Uncertain Input Parameters

4.3.2.1. Spatial Distribution of Water Loss

An uncertain input parameter relates to the assumptions that are made in advance of the modelling process, of which the validity cannot be verified in any practical sense. The spatial distribution of water loss in a sector is considered uncertain in the *SEGLEAK* procedure. All volumes of non-metered water (water loss) are classified as belonging to one of three distinct components, which include: (1) non-metered real loss, Q_{nrl} ; (2) non-metered apparent loss, Q_{nal} ; and (3) non-metered authorised consumption, Q_{nac} . This agrees with the classification structure of the simplified water balance, as presented before in this chapter. Realistic assumptions need to be made regarding the spatial distribution of each of the above-mentioned components in a hydraulic model.

4.3.2.2. Diurnal Consumption Patterns

Reliable information or knowledge regarding diurnal consumption patterns for consumers within the sector is also considered to be a relatively uncertain input parameter, since such consumption patterns could easily change from one day to the next.

Typical consumption patterns for different types of land-use categories are generally available, but it should be noted that such patterns are usually rather generic in nature and therefore not necessarily accurate for all specific cases.

4.3.3. Fundamental Logic Formulations

4.3.3.1. Requirement for Procedure

Some logical formulations are necessary for successful implementation of the *SEGLEAK* procedure. The most fundamental formulations are provided in this section, whereas the more complex and detailed equations are introduced in some of the subsequent sections, as part of the more detailed specifics relating to the *SEGLEAK* procedure.

4.3.3.2. Real Losses Fraction

First, the ratio of the total non-metered real loss flow rate, Q_{nrl} , to the total non-metered flow rate, Q_n , is defined as f_{nrl} , which is expressed accordingly by Eq. (8). Almandoz *et al.* (2005) expressed that this parameter could possibly be considered as the most significant, since it represents the fraction of the total non-metered flow rate, Q_n , that is due to leakage. The real losses fraction, f_{nrl} , can be calculated in different ways, some of which are explained further on in this chapter.

$$f_{nrl} = \frac{Q_{nrl}}{Q_n} \tag{8}$$

where:

$0 \leq f_{nrl} \leq 1$

f_{nrl}	-	Real losses fraction of non-metered water
Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
Q_n	-	Total non-metered flow rate [ℓ/s].

4.3.3.3. Total Daily Input Volume

Secondly, the measured total daily input volume, V_i^m, is determined from the measured total input flow rate, $Q_i^m(t)$, curve, by using the simple summation formulation, as presented by Eq. (9). Likewise, the simulated total daily input volume, V_i^s, is calculated from the simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curve, as presented by Eq. (10). As part of the *SEGLEAK* process, the values for V_i^m and V_i^s are required to match, which is achieved through the adjustment of the average leakage constant, K_l . This constant and the adjustment thereof are introduced and discussed more fully in the subsequent sections that deal with the specifics relating to the *SEGLEAK* procedure.

$$V_i^m = \sum_{j=1}^{24} Q_i^m(t)_j \tag{9}$$

$$V_i^s = \sum_{j=1}^{24} Q_i^s(t, f_{nrl})_j \tag{10}$$

where:

V_i^m	-	Measured total daily input volume [m^3]
t	-	Time [h]
$Q_i^m(t)_j$	-	Measured total input flow rate at hour *j* [m^3/h]
V_i^s	-	Simulated total daily input volume [m^3]
$Q_i^s(t, f_{nrl})_j$	-	Simulated total input flow rate at hour *j* [m^3/h].

4.3.3.4. Daily Average System Efficiency

The third fundamental formulation involves the aspect of daily average system efficiency, $\bar{\eta}_s$, which is defined as the ratio of the total daily volume of metered consumption, V_{mc} , to the measured total daily input volume, V_i^m, as presented by Eq. (11). Since the volumes of metered consumption might possibly be recorded on a timescale that is different to that of daily measurement (e.g. monthly measurements) in some cases, these values would need first to be converted to average daily volumes, before making use of Eq. (11).

$$\bar{\eta}_s = \frac{V_{mc}}{V_i^m} \tag{11}$$

where:

$\bar{\eta}_s$	-	Daily average system efficiency
V_{mc}	-	Total daily volume of metered consumption [m^3/d]
V_i^m	-	Measured total daily input volume [m^3/d].

4.3.3.5. General Leakage Equation

The general equation for leakage from a water distribution system is presented by Eq. (12), which also serves as the fourth fundamental logic formulation. Eq. (12) is equivalent to that presented as Eq. (4) in Chapter 2, and provides the expression for leakage flow rate as a function of pressure head. It should be noted that there is an entire range of different values available for the leakage exponent, $N1$, and that the value assumed for this parameter has significant implications for the volumes of leakage that are to be expected. The specific value assumed during the modelling procedure must therefore be clearly specified and sufficiently motivated.

$$Q_{nrl} = Ch^{N1} \tag{12}$$

where:

Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
C	-	Leakage coefficient
$N1$	-	Leakage exponent (typically between 0.5 and 1.5)
h	-	Pressure head [m].

4.3.4. Output Parameters

The 24-hour temporal profile of the simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curve is the only output parameter required for the *SEGLEAK* procedure. A typical example of such a curve, which was obtained as a result from a single trial simulation, is presented in Figure 4.2.

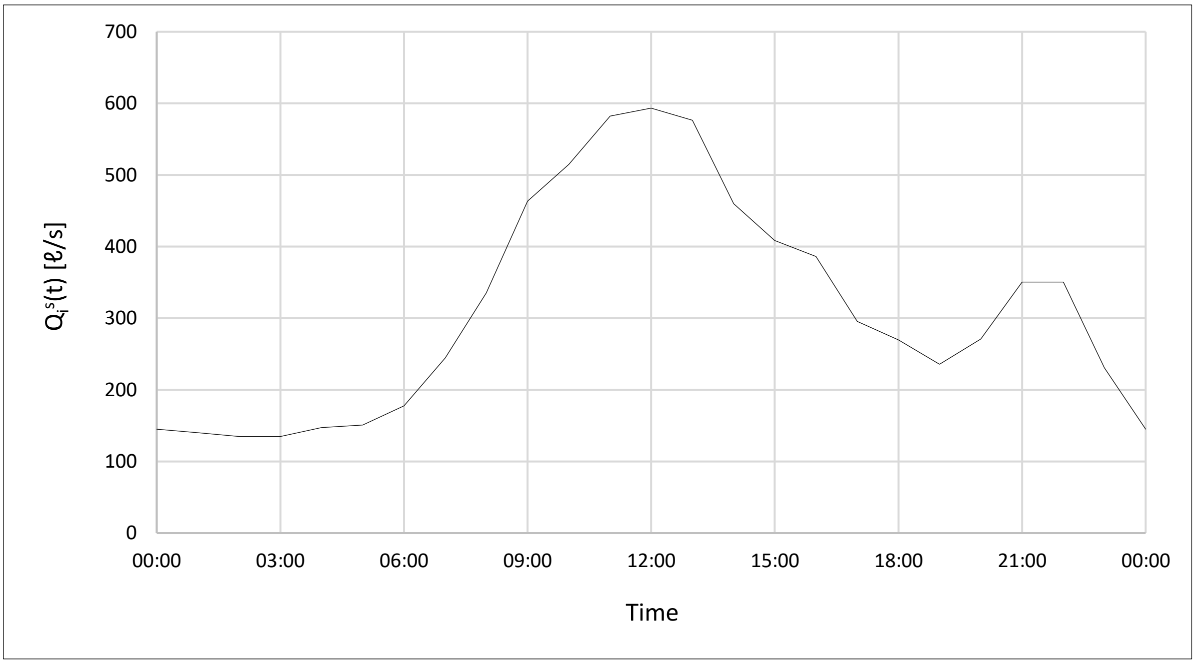

Figure 4.2: Example total input flow rate curve to sector for single trial simulation

The main reason for interest in this type of curve is that it is used in the *SEGLEAK* procedure to estimate the correct value for the real losses fraction, f_{nrl} , in any given sector, for which several successive simulation trials are usually necessary. This iterative process is implemented for different trial values of the real losses fraction, f_{nrl} , for which the corresponding values (typically hourly) of $Q_i^s(t, f_{nrl})$ are then subsequently compared to the measured values of $Q_i^m(t)$. The purpose of this process of comparison is clearly to obtain the most accurate representation of what occurs in the real-world.

Comparison of the relevant flow rate values can be done by implementing either one of two possible techniques, which are discussed in greater detail in a subsequent section. The result from this process of comparison is the estimated value of f_{nrl} .

4.3.5. Allocation of Water Loss

This section introduces a brief discussion of the *SEGLEAK* approach to the allocation of water losses to the nodes in a hydraulic model, and therefore relates to the discussion provided near the end of Chapter 2, which essentially involves the same topic. The *SEGLEAK* approach for spatially allocating water loss is defined as a type of combined distribution technique. This technique serves as a combination of some of the other techniques that have been introduced in Chapter 2, to better estimate reality regarding the actual spatial distribution of water loss throughout a water distribution system, or sector of such a system.

A pressure-dependent distribution (as discussed in Chapter 2) forms the basis of the combined distribution technique, whereas the weighted distribution (also discussed in Chapter 2) serves as an additional means of making provision for various other possible contributing factors. The length of the mains connected to a specific node is by default accepted as a contributing factor to leakage, for the purposes of the *SEGLEAK* procedure. One of the subsequent sections in this chapter thoroughly explains the combined distribution in sufficient detail and it is, for this reason, not given here.

4.4. Detailed Theory and Equations

4.4.1. Components of Nodal Demand

The nodal demand flow rate, $(Q_d)_j$, allocated to each distinct node in the hydraulic model of a water distribution system is assumed to consist of three basic components, as presented by Eq. (13). The three components are: (1) metered consumption flow rate, $(Q_{mc})_j$; (2) non-metered consumption flow rate, $(Q_{nc})_j$; and (3) non-metered real loss flow rate, $(Q_{nrl})_j$. The non-metered consumption flow rate, $(Q_{nc})_j$, represents the combination of non-metered authorised consumption flow rate, $(Q_{nac})_j$, and non-metered apparent loss flow rate, $(Q_{nal})_j$, as expressed by Eq. (14).

$$(Q_d)_j = (Q_{mc})_j + (Q_{nc})_j + (Q_{nrl})_j \tag{13}$$

$$(Q_{nc})_j = (Q_{nac})_j + (Q_{nal})_j \tag{14}$$

where:

$(Q_d)_j$	-	Demand flow rate at node j [ℓ/s]
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s]
$(Q_{nc})_j$	-	Non-metered consumption flow rate at node j [ℓ/s]
$(Q_{nrl})_j$	-	Non-metered real loss flow rate at node j [ℓ/s]
$(Q_{nac})_j$	-	Non-metered authorised consumption flow rate at node j [ℓ/s]
$(Q_{nal})_j$	-	Non-metered apparent loss flow rate at node j [ℓ/s].

Figure 4.3 presents a graphical illustration of the components of nodal demand, which specifically relates to the terms used in this chapter and is in accordance with the simplified water balance classification structure.

<table>
<tr><th colspan="2">Hydraulic model classification:</th><th colspan="3">Simplified water balance classification:</th></tr>
<tr><td rowspan="4">Nodal demand</td><td rowspan="3">Output</td><td>Metered water</td><td colspan="2">Metered consumption</td></tr>
<tr><td rowspan="3">Non-metered water</td><td rowspan="2">Non-metered consumption</td><td>Non-metered authorised consumption</td></tr>
<tr><td>Apparent loss</td></tr>
<tr><td>Emitter flow</td><td colspan="2">Leakage</td></tr>
</table>

Figure 4.3: Components of nodal demand according to simplified water balance

4.4.2. Metered Consumption Component

The total metered consumption flow rate, Q_{mc} , can be calculated as the sum of the metered consumption flow rates at each distinct node, $(Q_{mc})_j$, through the principle of continuity, as presented by Eq. (15). The metered consumption flow rate at a node, $(Q_{mc})_j$, is the only component of the nodal demand flow rate, $(Q_d)_j$, that can be estimated and allocated with a proper measure of certainty. This is because the values associated with this component can be verified in a practical sense from consumer records, which are generally available from municipal treasury databases.

$$Q_{mc} = \sum_{j=1}^{n} (Q_{mc})_j \tag{15}$$

where:

Q_{mc}	-	Total metered consumption flow rate [ℓ/s]
n	-	Number of nodes
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s].

It is assumed that this component is independent of residual pressure at the relevant node, although this is not always necessarily the case for real-world conditions. This assumption is, nevertheless, accepted as reasonable for the purposes of this study.

4.4.3. Non-Metered Consumption Component

As for the previous component, the continuity principle can again be implemented to obtain the total non-metered consumption flow rate, Q_{nc} , as the sum of the non-metered consumption flow rates at each distinct node, $(Q_{nc})_j$, as expressed accordingly by Eq. (16).

$$Q_{nc} = \sum_{j=1}^{n} (Q_{nc})_j \tag{16}$$

where:

Q_{nc}	-	Total non-metered consumption flow rate [ℓ/s]
n	-	Number of nodes
$(Q_{nc})_j$	-	Non-metered consumption flow rate at node j [ℓ/s].

Furthermore, the total non-metered consumption flow rate, Q_{nc} , needs to be distributed spatially among the distinct nodes, as part of the process of modelling a water distribution system. For the *SEGLEAK* procedure, it is assumed that there is proportionality between the metered consumption flow rate at any single node, $(Q_{mc})_j$, and the non-metered consumption flow rate at the same node, $(Q_{nc})_j$, as presented by Eq. (17). This assumption is particularly appropriate in the absence of additional criteria that specifically indicates otherwise. More detailed knowledge regarding the actual situation in the system is an example of such criteria.

$$(Q_{nc})_j = k \cdot (Q_{mc})_j \tag{17}$$

where:

$(Q_{nc})_j$	-	Non-metered consumption flow rate at node j [ℓ/s]
k	-	Ratio of non-metered consumption to metered consumption
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s].

By extending the previous assumption to be valid for all nodes in the water distribution system, an expression for k is obtained as a function of other familiar variables, as presented by Eq. (18).

$$k = \frac{Q_{nc}}{Q_{mc}} = \frac{Q_n - Q_{nrl}}{Q_{mc}} = \frac{Q_n}{Q_{mc}}\left[1 - \frac{Q_{nrl}}{Q_n}\right] = \frac{Q_n}{Q_{mc}}(1 - f_{nrl}) \tag{18}$$

where:

k	-	Ratio of non-metered consumption to metered consumption
Q_{nc}	-	Total non-metered consumption flow rate [ℓ/s]
Q_{mc}	-	Total metered consumption flow rate [ℓ/s]
Q_n	-	Total non-metered flow rate [ℓ/s]
Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
f_{nrl}	-	Real losses fraction of non-metered water.

The ratio of the total non-metered flow rate, Q_n , to the total metered consumption flow rate, Q_{mc} , is further expressed as a function of the daily average system efficiency, $\bar{\eta}_s$, as presented by Eq. (19).

$$\bar{\eta}_s = \frac{Q_{mc}}{Q_i} = \frac{Q_{mc}}{Q_{mc} + Q_n} = 1 + \frac{Q_{mc}}{Q_n}$$

$$\Rightarrow \frac{Q_n}{Q_{mc}} = \frac{1}{\bar{\eta}_s} - 1 \tag{19}$$

where:

$\bar{\eta}_s$	-	Daily average system efficiency
Q_{mc}	-	Total metered consumption flow rate [ℓ/s]
Q_i	-	Total input flow rate [ℓ/s]
Q_n	-	Total non-metered flow rate [ℓ/s].

By combining Eq. (17), (18), and (19), an expression is obtained for the non-metered consumption at a single node, $(Q_{nc})_j$, as presented in Eq. (20).

$$(Q_{nc})_j = (Q_{mc})_j(1 - f_{nrl})\left[\frac{1}{\bar{\eta}_s} - 1\right] \tag{20}$$

where:

$(Q_{nc})_j$	-	Non-metered consumption flow rate at node j [ℓ/s]
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s]
f_{nrl}	-	Real losses fraction of non-metered water
$\bar{\eta}_s$	-	Daily average system efficiency.

The total non-metered consumption flow rate, Q_{nc} , is subsequently expressed by Eq. (21), through substitution of Eq. (20) into Eq. (16).

$$Q_{nc} = \sum_{j=1}^{n}(Q_{mc})_j(1 - f_{nrl})\left[\frac{1}{\bar{\eta}_s} - 1\right] \tag{21}$$

where:

Q_{nc}	-	Total non-metered consumption flow rate [ℓ/s]
n	-	Number of nodes
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s]
f_{nrl}	-	Real losses fraction of non-metered water
$\bar{\eta}_s$	-	Daily average system efficiency.

4.4.4. Non-Metered Real Loss Component

4.4.4.1. Fundamental Concepts

The flow rate resulting from total non-metered real loss, Q_{nrl} , in a hydraulic model is calculated by making use of the principle of continuity, as expressed by Eq. (22). This equation simply states that the total non-metered real loss flow rate, Q_{nrl} , is equal to the sum of the non-metered real loss flow rates allocated to each individual node, $(Q_{nrl})_j$. Since all occurrences of leakage are modelled exclusively at system nodes, the formulation presented by Eq. (22) is accepted as completely valid, although leakage commonly occurs along the length of pipe mains in real-world situations.

$$Q_{nrl} = \sum_{j=1}^{n} (Q_{nrl})_j \tag{22}$$

where:

Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
n	-	Number of nodes
$(Q_{nrl})_j$	-	Non-metered real loss flow rate at node j [ℓ/s].

To determine the leakage flow rate that needs to be associated with each individual node, the effect of pressure within the proximity of the nodes must be accounted for. This is done by making use of the basic pressure-leakage relationship, as is presented in Chapter 2 by Eq. (4), which subsequently results in Eq. (23) for this context. It should be noted that the value for the leakage exponent, $N1$, is valid for the entire system if *Wadiso* is selected as the preferred software package for analysis purposes, which was indeed the case for this research study. The value for $N1$ is therefore not adjustable between distinct nodes when making use of *Wadiso*. Unfortunately, this is a limitation of the modelling procedure, which needed to be kept in mind when the results obtained from analysis trials were being interpreted.

$$(Q_{nrl})_j = C_j {h_j}^{N1} \tag{23}$$

where:

$(Q_{nrl})_j$	-	Non-metered real loss flow rate at node j [ℓ/s]
C_j	-	Leakage coefficient for node j
h_j	-	Pressure head at node j [m]
$N1$	-	Leakage exponent (typically between 0.5 and 1.5).

The appropriate leakage coefficient for each individual node, C_j , is found through an iterative process by making use of the principle of continuity, as presented by Eq. (22). The reason that the use of the continuity principle is in fact possible, is because the flow rate associated with total non-metered real loss, Q_{nrl} , can be determined from other known parameters. These known parameters include the total input flow rate, Q_i , total metered consumption flow rate, Q_{mc} , and total non-metered consumption flow rate, Q_{nc} . All leakage coefficients are essentially determined by the various factors assumed to contribute to leakage, and can therefore be approximated using different weighting factors and leakage constants. The concept of using weighting factors and leakage constants to spatially allocate leakage throughout the entire water distribution system is discussed more extensively in the next section.

4.4.4.2. Weighting Factors and Leakage Constants

Since there are many different types of contributing factor that could potentially affect the spatial distribution of leakage from a water distribution system, the non-metered real loss flow rate allocated to each node, $(Q_{nrl})_j$, should ideally consider the impact of as many as possible of these factors. The potential impact of any type of contributing factor can be imitated to a certain extent by using certain weighting factors and leakage constants. Possibly the simplest case of using weighting factors is when the water distribution system under consideration is regarded as completely homogeneous, with specific reference to system mains. For the above-mentioned case, the allocation of leakage flow rates to nodes is simplified by considering only the length of the mains as relevant weighting factors. Consequently, 50% of the total length of mains connected to a certain node would subsequently be assigned to the node itself, which ultimately results in the formulations expressed as Eq. (24), (25), and (26).

$$L_T = \sum_{j=1}^{n} \bar{L}_j \tag{24}$$

$$\bar{L}_j = \frac{L_j}{L_T} \tag{25}$$

$$\sum_{j=1}^{n} \bar{L}_j = 1 \tag{26}$$

where:

L_T	-	Total length of mains [m]
n	-	Number of nodes
L_j	-	50% of total length of mains connected to node j [m]
$\bar{L}_j$	-	Length weighting factor for node j.

Nodal leakage coefficients, C_j , are calculated by using Eq. (27), which implements a length weighting factor, $\bar{L}_j$, for this specific case. In principle, a weighting factor can also be created for any other system parameter that significantly influences the spatial distribution of leakage. Furthermore, a leakage constant, K_{lj} , is assigned to each node, to assist in the achievement of the continuity balance, as presented by Eq. (22). However, the leakage constant, K_{lj} , can also be used as a further means by which to assign a relative weight to a selected node regarding leakage from that node. This implies that the nodal leakage constant, K_{lj} , provides an additional measure of variability for numerically representing any possible differences regarding contributing factors between distinct nodes.

$$C_j = K_{lj} \cdot \bar{L}_j \tag{27}$$

where:

C_j	-	Leakage coefficient for node j
K_{lj}	-	Leakage constant for node j
$\bar{L}_j$	-	Length weighting factor for node j.

Since the nodal leakage constant, K_{lj} , is used partially to achieve a volumetric flow rate balance, an average leakage constant, K_l , can be introduced throughout the entire system for the case where all nodes prove to be affected similarly by the various contributing factors to leakage that are accounted for. Generally, the nodal leakage constant, K_{lj} , can be related to the average leakage constant, K_l , through Eq. (28), where α_j is defined as a nodal leakage constant weight for relating the leakage constant of a specific node, K_{lj} , to the average leakage constant, K_l .

$$K_{lj} = \alpha_j \cdot K_l \tag{28}$$

where:

K_{lj}	-	Leakage constant for node j
α_j	-	Leakage constant weight for node j
K_l	-	Average leakage constant.

4.4.4.3. Resulting Formulations

Several consecutive substitutions are appropriate regarding the preceding equations, to obtain complete integration between all the separate formulations that are presented in the preceding sections. As a start, Eq. (27) is combined with Eq. (28), which is then further combined with Eq. (23) and finally results in Eq. (29). Subsequently, Eq. (29) is further combined with Eq. (22), to obtain Eq. (30). The latter equation provides the relationship between the total non-metered real loss flow rate, Q_{nrl} , in a hydraulic model and the various parameters used to imitate the effects of the contributing factors to leakage at distinct nodes.

$$(Q_{nrl})_j = \alpha_j K_l \bar{L}_j {h_j}^{N1} \tag{29}$$

$$Q_{nrl} = \sum_{j=1}^{n} \alpha_j K_l \bar{L}_j {h_j}^{N1} \tag{30}$$

where:

$(Q_{nrl})_j$	-	Non-metered real loss flow rate at node j [ℓ/s]
α_j	-	Leakage constant weight for node j
K_l	-	Average leakage constant
$\bar{L}_j$	-	Length weighting factor for node j
h_j	-	Pressure head at node j [m]
$N1$	-	Leakage exponent (typically between 0.5 and 1.5)
Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
n	-	Number of nodes.

For the specific case where an average leakage constant, K_l , is applicable throughout the entire network, and therefore the value of α_j = 1.0 at all nodes, the expression provided by Eq. (30) can be simplified and presented simply as Eq. (31).

$$Q_{nrl} = K_l(f_{nrl}) \sum_{j=1}^{n} \bar{L}_j {h_j}^{N1} \tag{31}$$

where:

Q_{nrl}	-	Total non-metered real loss flow rate [ℓ/s]
$K_l(f_{nrl})$	-	Average leakage constant
n	-	Number of nodes
$\bar{L}_j$	-	Length weighting factor for node j
h_j	-	Pressure head at node j [m]
$N1$	-	Leakage exponent (typically between 0.5 and 1.5).

The value for the average leakage constant, K_l , is evidently a function of the real losses fraction, f_{nrl} , since the value for the total non-metered real losses flow rate, Q_{nrl} , is known to be a function of f_{nrl} , and all other parameters that appear in Eq. (31) are known to be independent of the value of f_{nrl} . This means that prior to each separate simulation for a specific trial value of f_{nrl} , the corresponding value of $K_l(f_{nrl})$ first needs to be determined through an iterative process, by making use of Eq. (31).

4.4.5. Final Volumetric Flow Rate Balance

A final volumetric flow rate balance is eventually obtained by theoretically integrating the various expressions that are provided in the preceding sections. Firstly, Eq. (20) and (29) are combined with Eq. (13), to obtain a detailed expression for the demand flow rate at a single node, $(Q_d)_j$, which is expressed accordingly by Eq. (32). As before, the expression provided by Eq. (32) can be extended to include all nodes through the principle of continuity, which is illustrated accordingly by Eq. (33). An average leakage constant, $K_l(f_{nrl})$, is once again assumed for the entire system, which means that α_j = 1.0 for all nodes.

$$(Q_d)_j = (Q_{mc})_j \left[1 + (1 - f_{nrl})\left[\frac{1}{\bar{\eta}_s} - 1\right]\right] + \alpha_j K_l(f_{nrl})\bar{L}_j {h_j}^{N1} \qquad (32)$$

$$Q_d = \sum_{j=1}^{n} (Q_{mc})_j \left[1 + (1 - f_{nrl})\left[\frac{1}{\bar{\eta}_s} - 1\right]\right] + K_l(f_{nrl}) \sum_{j=1}^{n} \bar{L}_j {h_j}^{N1} \qquad (33)$$

where:

$(Q_d)_j$	-	Demand flow rate at node j [ℓ/s]
$(Q_{mc})_j$	-	Metered consumption flow rate at node j [ℓ/s]
f_{nrl}	-	Real losses fraction of non-metered water
$\bar{\eta}_s$	-	Daily average system efficiency
α_j	-	Leakage constant weight for node j
$K_l(f_{nrl})$	-	Average leakage constant
$\bar{L}_j$	-	Length weighting factor for node j
h_j	-	Pressure head at node j [m]
$N1$	-	Leakage exponent (typically between 0.5 and 1.5)
Q_d	-	Total demand flow rate [ℓ/s]
n	-	Number of nodes.

4.5. Simulation Aspect

4.5.1. General Purpose

The general purpose of the simulation aspect of *SEGLEAK* is to provide a practical approach to distinguishing between different types of non-metered water in a distribution system, through the comparison of the diurnal patterns of simulated and measured total input flow rate curves. In a similar fashion to the method proposed by Almandoz *et al.* (2005), an estimate for the fractional split between non-metered real loss and non-metered consumption is obtained as a result, through implementation of the simulation aspect of this procedure.

Several simulation trials are usually necessary during implementation of the simulation aspect, which correspond to a selected set of trial values for the real losses fraction of non-metered water, f_{nrl} . Each trial value of f_{nrl} produces a distinct simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curve. Two different techniques available for estimating the correct value of f_{nrl} are presented and thoroughly explained in a subsequent section.

4.5.2. Evaluation of Average Leakage Constant

Prior to the execution of a simulation that corresponds to a certain value of the real losses fraction of non-metered water, f_{nrl} , the appropriate average leakage constant, $K_l(f_{nrl})$, needs first to be evaluated. Every trial value of f_{nrl} has a unique corresponding value for $K_l(f_{nrl})$ that equates the simulated total daily input volume, V_i^s, to the measured total daily input volume, V_i^m. For this research study, the hourly values for any particular $Q_i^s(t, f_{nrl})$ curve were obtained as an output from *Wadiso*, during implementation of the simulation process, from which the corresponding value of V_i^s was then calculated by summing the above-mentioned hourly values over a 24-hour period. If a different software package is, however, selected (e.g. *EPANET*), the resulting flow rates of the $Q_i^s(t, f_{nrl})$ curve would be necessary for evaluating the value of $K_l(f_{nrl})$.

The expression presented as Eq. (33), is particularly applicable for the evaluation of the average leakage constant, $K_l(f_{nrl})$. Calculation of the appropriate value for $K_l(f_{nrl})$ is an iterative procedure, which means that any selected value for $K_l(f_{nrl})$ can be used as an initial estimate. The value of $K_l(f_{nrl})$ is adjusted accordingly (either increased or decreased), to achieve the balance of volumes, as stated in the preceding paragraph.

4.5.3. Estimation of Real Losses Fraction

To accurately estimate the correct value of the real losses fraction of non-metered water, f_{nrl} , for any selected sector, an appropriate technique is required for comparing the results corresponding to selected trial values of f_{nrl} with the actual measured data. Comparison between the diurnal patterns of the simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curves and the measured total input flow rate, $Q_i^m(t)$, curve, is regarded as a reliable method for estimating the correct value of f_{nrl} . Accordingly, two separate techniques of comparison that are available for use are introduced in this section.

As mentioned before, the *SEGLEAK* technique assumes that non-metered consumption varies, primarily, in proportion to demand patterns and that leakage varies mostly according to the pressure-leakage relationship. The method selected by Almandoz *et al.* (2005) for comparing the $Q_i^s(t, f_{nrl})$ values with the $Q_i^m(t)$ values is by means of equating their daily standard deviations, as presented by Eq. (34). This method is, accordingly, referred to as the *method of standard deviations*. It should be noted that each distinct simulation corresponding to a certain value of f_{nrl} produces a unique simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curve.

$$\sigma^s(f_{nrl}) = \sigma^m \tag{34}$$

where:

$\sigma^s(f_{nrl})$	-	Standard deviation of $Q_i^s(t, f_{nrl})$ values over a single day period [ℓ/s]
σ^m	-	Standard deviation of $Q_i^m(t)$ values over a single day period [ℓ/s].

However, although the $Q_i^s(t, f_{nrl})$ curves are distinctive for different values of f_{nrl} , each such curve effectively supplies the same simulated total daily input volume, V_i^s. The aspect that makes each $Q_i^s(t, f_{nrl})$ curve unique, however, is its measure of deviance from the mean flow rate value. The deviance of a curve can easily be determined by making use of the *method of standard deviations*, as mentioned before. A useful technique for comparing the standard deviations of the various $Q_i^s(t, f_{nrl})$ curves with that of the single $Q_i^m(t)$ curve is to plot the calculated $\sigma^s(f_{nrl})$ values against the range of trial values for f_{nrl} , which are tested as part of the simulation trials. An example of such a plotted series is illustrated in Figure 4.4.

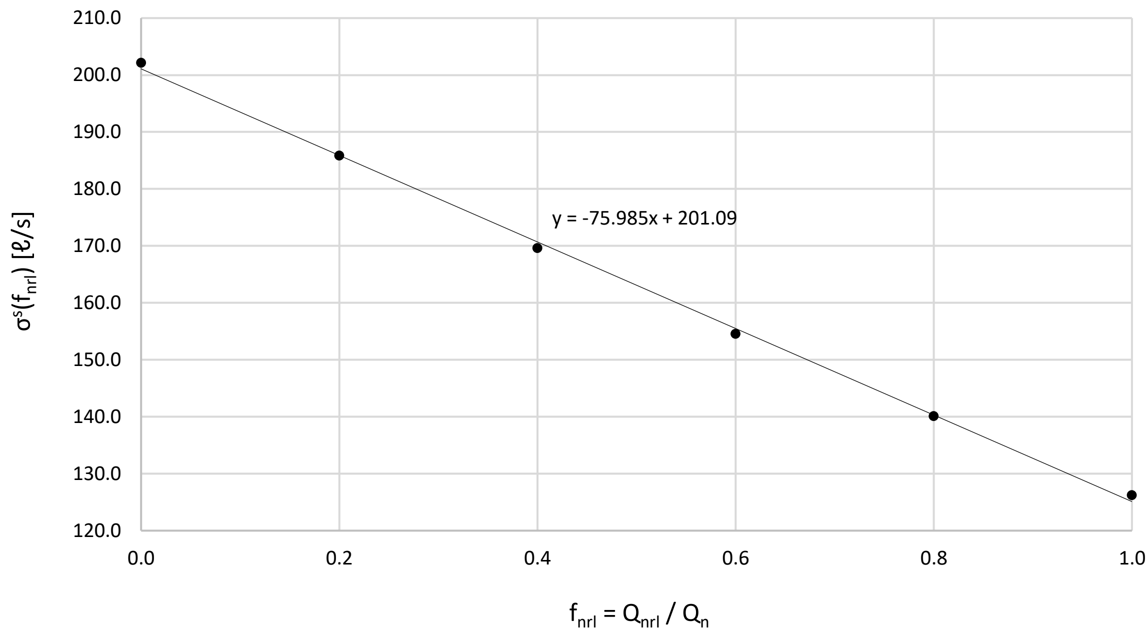

Figure 4.4: Method of standard deviations example

It should be clear from this series that the relationship between f_{nrl} and $\sigma^s(f_{nrl})$ is almost completely linear. In fact, a linear function can be fitted to this series, as shown by Figure 4.4, to estimate the approximate value of f_{nrl} for any specified $\sigma^s(f_{nrl})$ value. This technique is thus particularly useful for obtaining the correct f_{nrl} value that corresponds to the actual known σ^m value. Suppose it is known that σ^m = 150 ℓ/s, then from a linear curve fitted to the series presented in Figure 4.4, for which the corresponding expression is also presented, the value of f_{nrl} ≈ 0.67.

The method presented above is effective for estimating the correct value of f_{nrl} , but could be improved in certain cases. A good example would be the case where a relatively smooth measured total input, $Q_i^m(t)$, curve is established for a sector. For such a case, the standard deviations of the values for the simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curves might be very close to that of the measured total input flow rate, $Q_i^m(t)$, curve, although the temporal profiles of the curves do not necessarily match very well. It would therefore be more useful to compare each of the distinct $Q_i^s(t, f_{nrl})$ values over the relevant temporal period, to each of the corresponding $Q_i^m(t)$ values. This can be done by means of identifying the value of f_{nrl} that minimises the sum of the calculated squared residuals from the separate simulations performed, as better illustrated by Eq. (35). All references to this technique are subsequently done as the *method of squared residuals*.

$$\min_{0 \le f_{nrl} \le 1} \left[\sum_{j=1}^{24} \left[Q_i^m(t)_j - Q_i^s(t, f_{nrl})_j \right]^2 \right] \tag{35}$$

where:

f_{nrl}	-	Real losses fraction of non-metered water
t	-	Time [h]
$Q_i^m(t)_j$	-	Measured total input flow rate at hour j [ℓ/s]
$Q_i^s(t, f_{nrl})_j$	-	Simulated total input flow rate at hour j [ℓ/s].

The values for the *method of squared residuals* can be plotted against the range of trial values for f_{nrl} , which are tested as part of the separate simulation trials. Figure 4.5 illustrates a typical example of such a series, which can be used in a similar fashion to estimate the correct value of f_{nrl} for a sector. It should be noted that for this example each sum of squared residual values is first normalised before being plotted. This normalising step involves dividing the sum-total value for each distinct simulation by the single largest sum-total value obtained for all simulations, after which the resulting set of values is plotted against the range of corresponding f_{nrl} values. The rationale behind the normalisation step is simply to avoid having to work with exceptionally large values, since the sum of squared residual values can prove to be relatively high, depending on the units of measurement used.

From Figure 4.5, it should be clear that a second-order polynomial function can be used to accurately approximate the trend indicated by the series of calculated points. By using the expression obtained for the polynomial function, as indicated by Figure 4.5, the minimum value for the normalised sum of squared residuals is obtained, corresponding to a value of $f_{nrl} \approx 0.79$, for this specific example. It is interesting to compare the difference in the values obtained for f_{nrl} between the two separate estimation techniques, since both techniques were implemented for the same sample problem. The *method of squared residuals* is possibly slightly more accurate in estimating the correct value of f_{nrl} , as opposed to the *method of standard deviations*, whereas the latter method seems to be somewhat less complex in application. Selection of either one of these two methods, therefore, depends on the personal preference of the user.

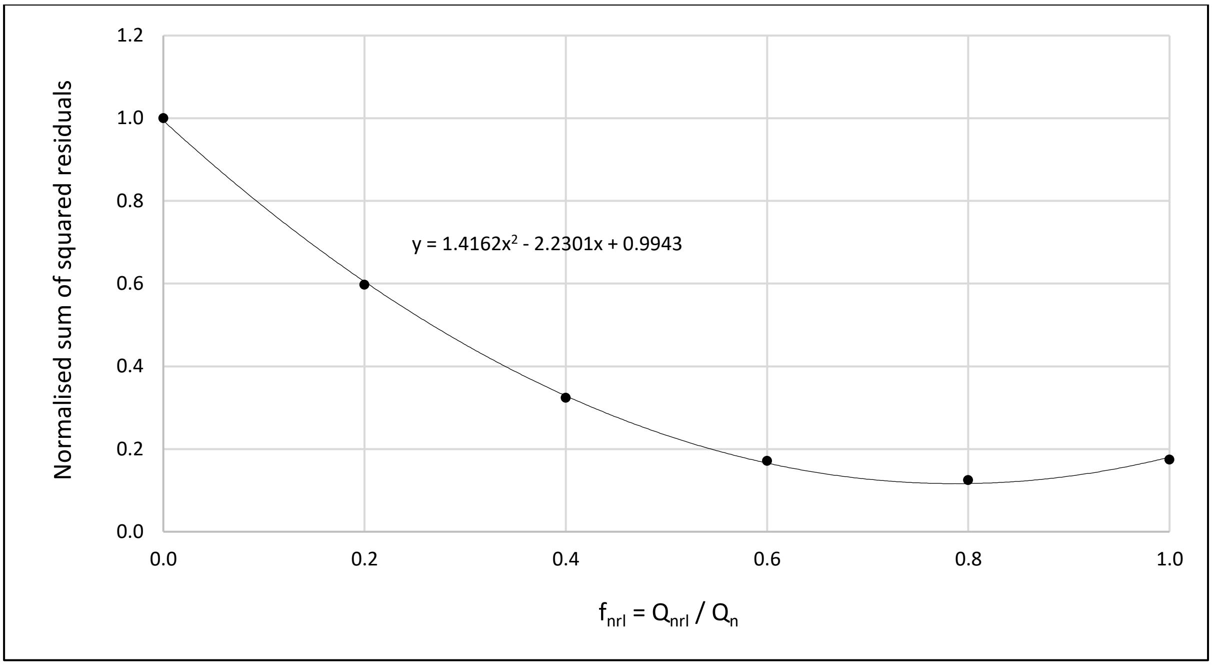

Figure 4.5: Method of squared residuals example

5. CASE STUDY PROBLEM: A REAL RESERVOIR SUPPLY ZONE IN SOUTH AFRICA

5.1. Overview

The practical application of the *SEGLEAK* procedure is systematically demonstrated in this chapter by implementing it on a real-world case study problem. All methodological steps forming part of the overall *SEGLEAK* procedure are presented and explained in this chapter. Furthermore, the results obtained from the related analyses are subsequently presented and discussed accordingly.

5.2. Data Acquisition

5.2.1. Flow Logging

5.2.1.1. Selection Criteria

As stated in Chapter 3, a large set of real-world flow logging data was used to evaluate the *SEGLEAK* procedure. The author started off by accessing the *Zednet* web-based system, from which an appropriate data set was identified and selected. It was also stated in Chapter 3 that selection of this data set was done based on the reliability of the logged data, according to the following categories of selection criteria: (1) continuity (i.e. prevalence of gaps and/or spikes); (2) record length; (3) consistency (in terms of the temporal profile pattern); (4) size of the distribution zone / district metering area; and (5) number of different land-use categories represented within the distribution zone / district metering area. The study area that was accordingly selected and used for the purposes of this study is introduced in the following section.

5.2.1.2. Study Area

An existing water distribution zone, which is referred to as *CaseStudyReservoir* for the purposes of this research study, was identified as sufficiently fulfilling the requirements that were regarded as acceptable, according to the selection criteria that have been outlined in Table 3.1, as part of Chapter 3. This supply zone forms part of the greater *City of Tshwane Metropolitan Municipality*, which is based in Gauteng, South Africa. The actual name of the supply zone was specifically substituted with *CaseStudyReservoir*, to avoid ethical issues regarding breaching of privacy or confidentiality. After visually inspecting the most recent flow logging data available for the *CaseStudyReservoir* supply zone, an appropriate flow logging record interval was successfully identified, selected, and acquired from the *Zednet* system. Some of the relevant details regarding this flow logging record are provided in Table 5.1.

Table 5.1: *CaseStudyReservoir* flow logging record details

Data set details:	
Site name:	*CaseStudyReservoir* OUT - Meter ##
Record length:	6 months
Record start:	2016-11-01 00:00
Record end:	2017-05-01 00:00
Recording freqency:	15 min intervals
Units:	m^3/h

A screenshot (as it appears on *Zednet*) of the temporal profile of the selected record interval is illustrated in Figure 5.1. The complete record of this data set is also provided graphically in Figure A1.1, as part of Appendix A1.

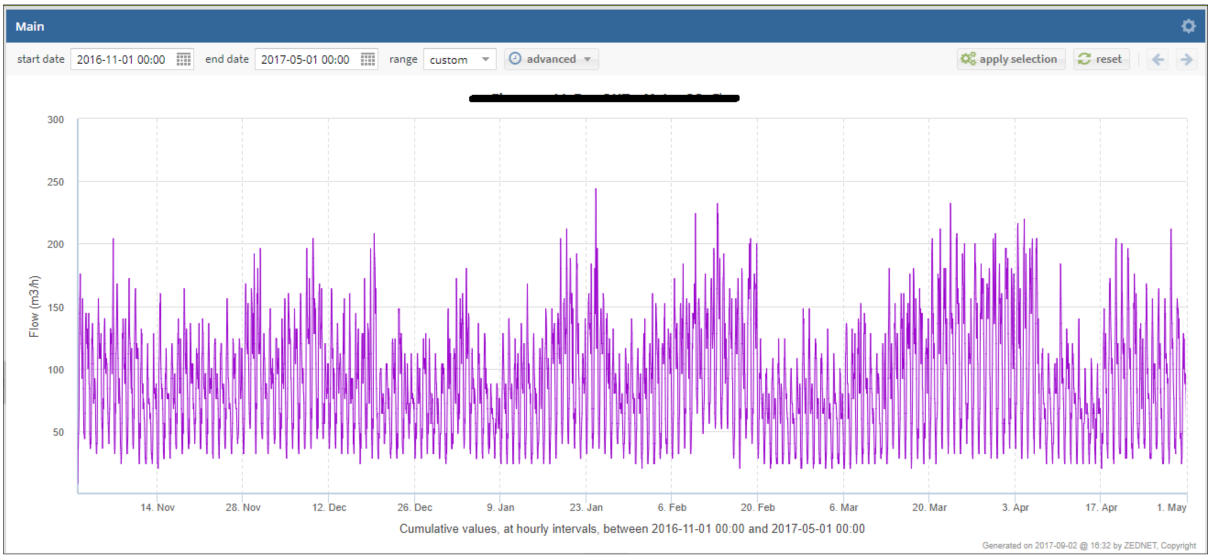

Figure 5.1: Selected flow logging data as presented by *Zednet* (WRP (Pty.) Ltd., 2017c)

As mentioned above, this flow record conformed with all the measures/limits of acceptability that were outlined in Table 3.1: Firstly, the continuity of the record is considered excellent, since there are zero gaps and very few significant spikes in the data set. The length of this record is also considered to be satisfactory, since it well exceeded a 1-month period. A consistent temporal profile pattern is evident from the record, with generally two peaks throughout the day for most of days in the record. Furthermore, the size of the supply zone is relatively small, with an annual average daily demand (AADD) of less than 1 000 kℓ/d. *CaseStudyReservoir* zone also had a single land-use category during the period corresponding to the selected record, which was particularly favoured for the purposes of testing and demonstrating application of the *SEGLEAK* procedure.

5.2.2. Hydraulic Model

As mentioned before in Chapter 3, a *Wadiso* hydraulic model of the real-world water distribution system of *CaseStudyReservoir* was acquired from *GLS Consulting* for the purposes of performing hydraulic analyses, to apply and evaluate the *SEGLEAK* procedure. A screenshot of the *CaseStudyReservoir* hydraulic model, as it appeared in *Wadiso* during the analysis procedures, is illustrated in Figure 5.2. Some additional information regarding this hydraulic model, as well as an appropriate schematic drawing of the related water distribution system is provided as part of Appendix A2. The schematic drawing is titled Drawing_01.

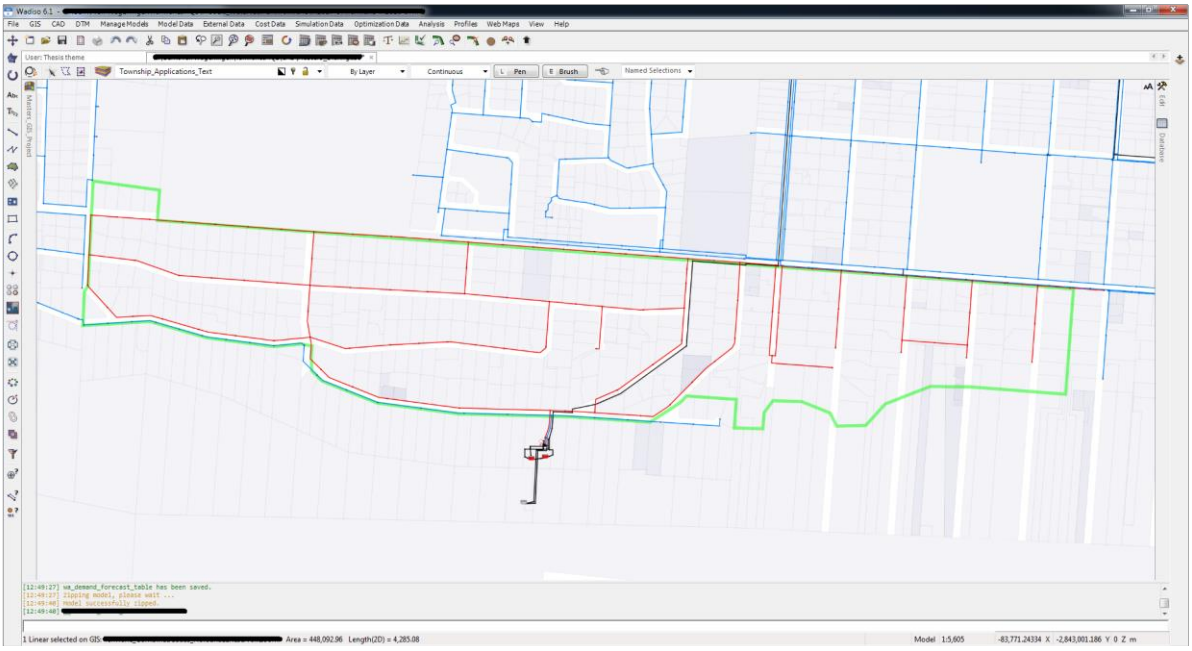

Figure 5.2: Screenshot of *CaseStudyReservoir* hydraulic model in *Wadiso*

5.2.3. Swift

The *CaseStudyReservoir* hydraulic model was already populated with the relevant Swift data at the time of acquisition of the model. A complete record of this Swift data set for the relevant nodes of the model is provided in Table A3.1, as part of Appendix A3. As a result, the author of this research study fortunately did not need to perform the rather time-consuming exercise of first cleaning up, filtering through, and manipulating the Swift data, before linking it to the *CaseStudyReservoir* hydraulic model, since this procedure had already been performed.

An example of a *Wadiso* hydraulic model, for which the node table had previously been populated with Swift data, is illustrated by a screenshot in Figure 5.3. The node table of the aforesaid example model is shown in Figure 5.3, with its various fields containing either necessary or simply informative data regarding the nodes of the model.

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Layout All Fields

Index	Node Type	Node Cod	Exists	Elevation (m)	Output (L/s)	Emitter Coefficient	AADD (kL/d)	AADD + UAW (kL/d)	Th AADD (kL/d)	Th AADD + Vac (kL/d)	Th AADD Zone (kL/d)
62,818	NODE	100001		1,414.6	0.28	0.00	3.94	4.93	6.00	6.00	6.00
62,819	NODE	100021		1,459.5	0.64	0.00	10.34	12.92	14.80	14.80	15.40
62,821	NODE	100028		1,420.5	0.25	0.00	7.99	9.98	5.44	5.44	5.44
62,826	NODE	100053		1,459.4	1.12	0.00	16.88	21.10	26.15	26.15	26.95
62,827	NODE	100054		1,460.9	0.64	0.00	6.23	7.79	15.00	15.00	15.40
62,828	NODE	100055		1,471.9	0.16	0.00	4.08	5.10	3.85	3.85	3.85
62,831	NODE	100058		1,509.8	0.24	0.00	3.41	4.01	5.75	5.75	5.75
62,832	NODE	100059		1,509.2	0.27	0.00	0.89	1.05	1.63	6.37	6.37
62,833	NODE	100060		1,509.7	0.61	0.00	9.79	11.51	14.56	14.56	14.56
62,834	NODE	100061		1,511.9	0.45	0.00	6.71	7.90	10.81	10.81	10.81
62,836	NODE	100063		1,479.4	0.96	0.00	23.27	29.09	15.40	23.10	23.10
62,837	NODE	100064		1,517.7	0.55	0.00	8.48	9.98	13.12	13.12	13.12
62,838	NODE	100065		1,517.8	0.76	0.00	13.67	16.09	18.19	18.19	18.19
62,839	NODE	100066		1,469.6	0.32	0.00	4.94	6.18	7.50	7.50	7.70
62,840	NODE	100068		1,484.7	0.32	0.00	1.52	1.90	3.65	7.50	7.70
62,842	NODE	100070		1,470.8	0.80	0.00	9.59	11.98	15.20	19.05	19.25
62,844	NODE	100073		1,518.3	0.96	0.00	11.22	13.20	23.00	23.00	23.00
62,845	NODE	100074		1,520.2	0.83	0.00	12.47	14.66	20.00	20.00	20.00
62,846	NODE	100075		1,519.0	0.97	0.00	16.52	19.44	21.75	23.19	23.19
62,847	NODE	100076		1,520.2	0.87	0.00	13.17	15.49	20.76	20.76	20.76
62,849	NODE	100078		1,482.7	0.16	0.00	2.87	3.59	3.85	3.85	3.85
62,850	NODE	100079		1,519.3	0.35	0.00	5.14	6.05	6.87	8.31	8.31
62,851	NODE	100080		1,509.6	0.54	0.00	9.33	10.97	12.94	12.94	12.94
62,852	NODE	100081		1,519.9	0.85	0.00	18.49	21.75	20.31	20.31	20.31
62,853	NODE	100082		1,519.8	1.02	0.00	18.86	22.19	24.40	24.40	24.40
62,854	NODE	100083		1,445.7	1.08	0.00	27.45	34.31	18.85	25.53	25.93
62,855	NODE	100084		1,519.4	0.40	0.00	5.39	6.34	9.56	9.56	9.56
62,856	NODE	100085		1,469.3	0.80	0.00	10.53	13.16	11.35	18.85	19.25
62,857	NODE	100086		1,520.0	0.79	0.00	15.20	17.88	18.88	18.88	18.88
62,858	NODE	100087		1,519.6	0.18	0.00	2.25	2.64	4.31	4.31	4.31
62,859	NODE	100088		1,519.8	0.42	0.00	5.55	6.53	10.13	10.13	10.13

Figure 5.3: Screenshot of a *Wadiso* hydraulic model populated with Swift data

From Figure 5.3, it is evident that the five rightmost columns (colour coded purple) contain information regarding consumption data for five different demand scenarios. These columns represent the fields containing the Swift data that has been linked to the hydraulic model.

5.3. Modelling Process

5.3.1. Known Input Parameters

5.3.1.1. Measured Total Input Flow Rate

According to Chapter 4, the first known input parameter required is the temporal profile of the measured total input flow rate, $Q_i^m(t)$, to the sector under consideration, during a single day period. This information was obtained as part of the flow logging data acquisition process. The measured total input flow rate, $Q_i^m(t)$, to *CaseStudyReservoir* zone for a typical weekday is shown in Figure 5.4.

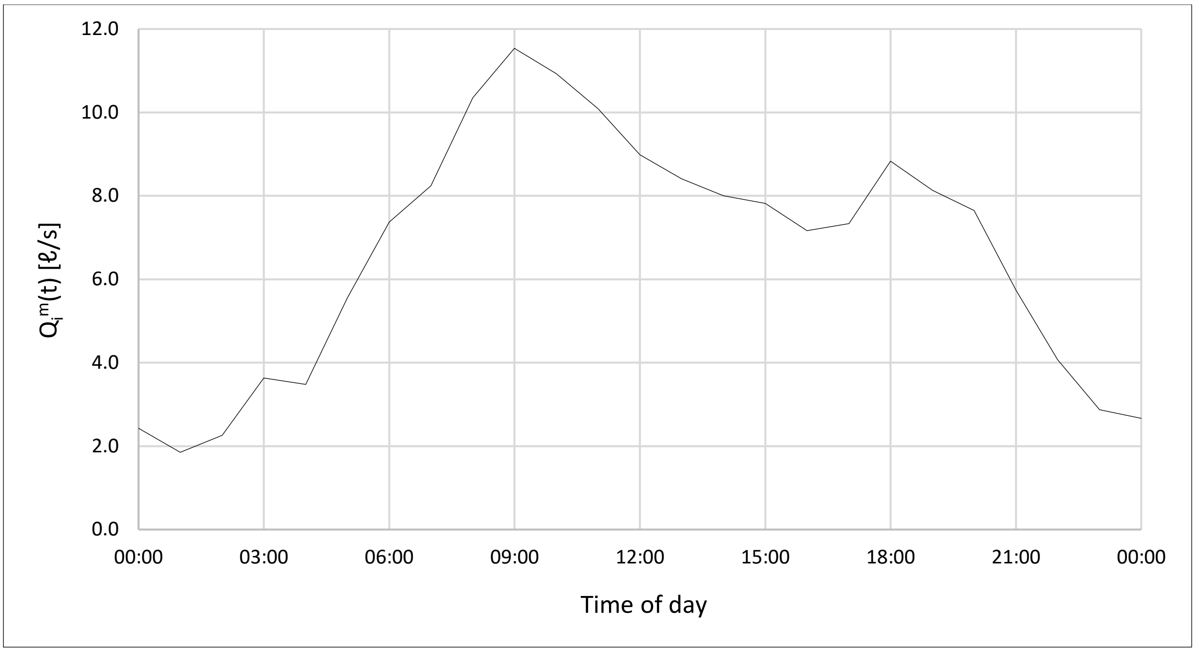

Figure 5.4: Measured total input flow rate to *CaseStudyReservoir* zone for a typical weekday

5.3.1.2. Daily Average Metered Consumption

The second known input parameter, according to Chapter 4, is the daily average metered consumption flow rate at each node, $(\bar{Q}_{mc})_j$, in the hydraulic model. This information was obtained in the form of Swift data, which already formed part of the hydraulic model of *CaseStudyReservoir*. Acquisition of the Swift data and the corresponding hydraulic model has been discussed in the previous sections of this chapter. As mentioned before, the complete record of the relevant Swift data set is provided in Table A3.1, as part of Appendix A3.

5.3.1.3. Total Daily Metered Consumption Volume

The third known input parameter is the total daily volume of metered consumption, V_{mc} , within the relevant sector. Chapter 4 explains that the value for V_{mc} is calculated simply by taking the sum of all the $(\bar{Q}_{mc})_j$ values at the various nodes. For the *CaseStudyReservoir* case study, the value for V_{mc} = 490.3 kℓ/d.

5.3.2. Uncertain Input Parameters

5.3.2.1. Spatial Distribution of Water Loss

According to Chapter 4, the first uncertain input parameter for the *SEGLEAK* procedure pertains to the spatial distribution of water loss in a water distribution system. The concept of using weighting factors and leakage constants for correlating instances of leakage to the primary contributing factors of leakage has already been introduced in Chapter 4, as part of the *SEGLEAK* procedure. This technique was used, accordingly, for the hydraulic model of the *CaseStudyReservoir* case study.

For this case study, it was assumed that the water distribution system is completely homogeneous regarding the various possible contributing factors that could potentially influence the occurrence of leakage (excluding the effects of pressure). This implies that the total length of mains connected to a specific node was assumed exclusively to govern the relative weight assigned to that node for leakage. The appropriate length weighting factors for the relevant nodes of the *CaseStudyReservoir* hydraulic model are listed in Table A2.1, as part of Appendix A2.

The spatial distribution of apparent loss and non-metered authorised consumption in the *CaseStudyReservoir* system also needed to be considered. Together, these two components make up the total non-metered consumption in a water distribution system. No pertinent distinction is made between the two components by the *SEGLEAK* approach, and both components were therefore modelled together as a single component at each relevant node. This was done by summing the two non-metered consumption flow rates at each relevant node, and adding it to the metered consumption flow rate at each corresponding node. The proposed assumption regarding proportionality between the non-metered and metered consumption (as presented in Chapter 4) was accepted as valid for the *CaseStudyReservoir* case study, since no definite information indicating otherwise was available.

5.3.2.2. Diurnal Consumption Patterns

The second uncertain input parameter, according to Chapter 4, involves information regarding the diurnal consumption patterns of consumers in the *CaseStudyReservoir* distribution zone. Since the accuracy of the diurnal consumption patterns used is crucially important for the success of this technique, the decision was made to not make use of some already available generic consumption patterns that are generally utilised by the engineers at *GLS Consulting*, and to which the author therefore also had access to. The above-mentioned generic patterns are based on the different land use types that are generally used for classification of various types of consumers regarding water demand behaviour. Instead, the extensive flow logging record from *Zednet* was used to develop custom/case-specific consumption patterns for the consumers within *CaseStudyReservoir* zone. Development of these custom consumption patterns from the *Zednet* data set is discussed in the following section.

5.3.2.3. Development of Consumption Patterns for *CaseStudyReservoir*

As stated in the previous section, some case-specific diurnal consumption patterns were developed for the *CaseStudyReservoir* case study. The entire 6-month data set, of which the details have already been provided in Table 5.1, was used to develop diurnal consumption patterns for each day of a standard 7-day week. All public holidays were excluded from the data set, since those days were regarded to not be representative of the typical consumption in the zone during normal circumstances. For each one of the 7 days of a standard week (i.e. Monday, Tuesday, etc.), the mean and standard deviation of the flow rate into *CaseStudyReservoir* zone was calculated for each hour of the day. It was assumed that each set of values at a specific hour on a specific day of the week (e.g. a set of values for 03:00 on a Monday morning) was normally distributed.

Furthermore, to ensure exclusion of outlier values in each set, a confidence interval of 90% was used to include all values that were distributed within 1.645 times the standard deviation from the mean value. After exclusion of the outlier values, an adjusted mean value was calculated for each hour of each day of the week. The resulting adjusted mean values were then used to generate the case-specific consumption patterns for every day of the week, by dividing each hourly value by the daily mean value. The case-specific consumption patterns that were developed is illustrated graphically in Figure 5.5 below and in tabular format in Table A1.1, as part of Appendix A1.

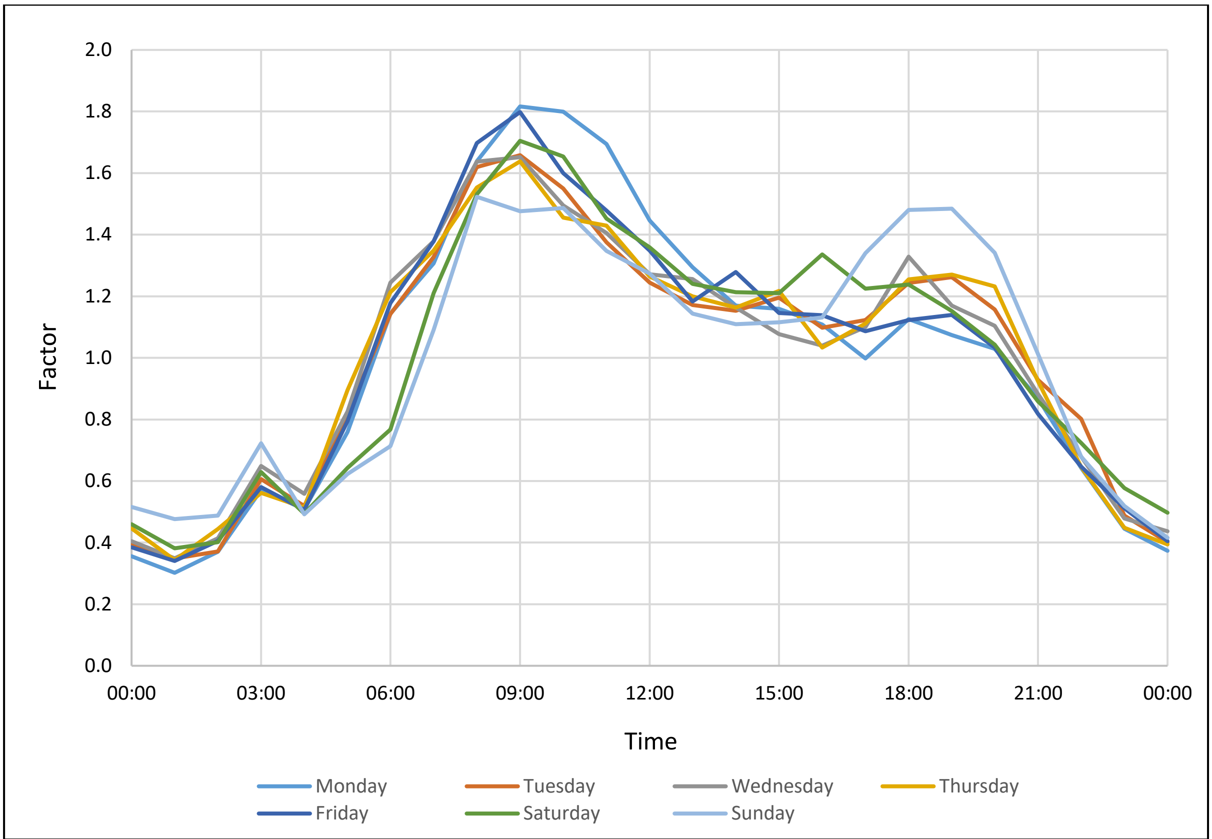

Figure 5.5: Diurnal consumption patterns developed for *CaseStudyReservoir*

5.3.3. Fundamental Logic Formulations

5.3.3.1. Real Losses Fraction

The first fundamental formulation involves the ratio of the total non-metered real loss flow rate, Q_{nrl} , to the total non-metered flow rate, Q_n , which was defined as the real losses fraction, f_{nrl} , in Chapter 4. The real losses fraction, f_{nrl} , is obtained by implementing the *SEGLEAK* procedure, and its value remains unknown at this point.

5.3.3.2. Total Daily Input Volume

The second fundamental formulation in Chapter 4 provided an expression, in the form of Eq. (9), for calculating the measured total daily input volume, V_i^m, from the measured total input flow rate, $Q_i^m(t)$, curve. By making use of Eq. (9), the measured total daily input volume was calculated as V_i^m ≈ 587 m^3/d.

5.3.3.3. Daily Average System Efficiency

Estimation of the daily average system efficiency, $\bar{\eta}_s$, forms part of the third fundamental formulation in Chapter 4. The daily average system efficiency, $\bar{\eta}_s$, was defined as the ratio of the total daily volume of metered consumption, V_{mc} , to the measured total daily input volume, V_i^m. Since the values for both V_{mc} and V_i^m are known at this point, the value of the daily average system efficiency, $\bar{\eta}_s$, is accordingly calculated as $\bar{\eta}_s$ ≈ 0.84.

5.3.3.4. General Leakage Equation

The fourth fundamental formulation does not involve a calculation and simply serves as theoretical background to the pressure-leakage relationship. A value of 1.0 was selected as appropriate for the leakage exponent, $N1$. This decision was based on a recommendation by McKenzie (1999), and was used as illustration of the *SEGLEAK* procedure presented in this thesis. Other values of the $N1$ exponent could, of course, be used as deemed appropriate for future application to actual water distribution systems.

5.3.4. Simulation Trials

All necessary input parameters and fundamental logic formulations for the case study problem have clearly been specified at this point, which meant that the modelling process was ready to be implemented. To obtain the correct value of the real losses fraction, f_{nrl} , for the case study water distribution system, several simulations trials needed to be performed for a selected set of f_{nrl} values. Since the value of f_{nrl} always varies between 0 and 1, a selected number of trial values (multiples of 0.2) were tested within the possible range. The output flow rates (sum of metered and non-metered consumption, as illustrated by Figure 4.3 in Chapter 4) that were allocated to the relevant nodes of the hydraulic model, are presented in Table A2.2 and Table A2.3, as part of Appendix A2. Each set of output flow rates corresponds to the selected trial values for f_{nrl} .

As explained before, an appropriate value for the average leakage constant, $K_l(f_{nrl})$, is required, to equate the simulated total daily input volume, V_i^s, to the measured total daily input volume, V_i^m. The tedious iterative process of equating the V_i^s value to the V_i^m value, through adjustment of the $K_l(f_{nrl})$ value, involves a sequence of repetitive steps. Computational time is therefore an inevitable and unfortunate drawback of the current version of the procedure. The above-mentioned sequence of steps that was followed, is:

1. Selection of an appropriate value for $K_l(f_{nrl})$.
2. Updating of the nodal leakage coefficients in the hydraulic model.
3. Execution of a single extended period simulation trial.
4. Extraction of the $Q_i^s(t, f_{nrl})$ curve flow rate values from the simulation results.

5. Summation of the $Q_i^s(t, f_{nrl})$ curve flow rate values over a single day period, to obtain the simulated total daily input volume, V_i^s.
6. Comparison of the measured total daily input volume, V_i^m, with the simulated total daily input volume, V_i^s.
7. If the volumes in step 6 were equal, the selected value of $K_l(f_{nrl})$ had been correct and the simulation for the trial value of f_{nrl} was complete. If, however, the volumes in step 6 were not equal, the entire process needed to be repeated, from step 1 to 6, until equity between the volumes were achieved.

The final resulting values for $K_l(f_{nrl})$ are provided in Table 5.3, as part of the subsequent section, for each of the selected trial values of f_{nrl} . A tolerance of ≤ 5.0% between the values for V_i^m and V_i^s was accepted as satisfactory. Setting a smaller tolerance resulted in an extremely time-consuming process and did not lead to a notable difference in the result.

5.3.5. Resulting Flow Rates

Table 5.2 provides the resulting values for the measured total input flow rate, $Q_i^m(t)$, curve, as well as the set of simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curves. The $Q_i^m(t)$ column that corresponds to a daily average system efficiency, $\overline{\eta_s}$, of 1.0, represents the hypothetical case where all water that is input to the system is ultimately measured by consumer meters (i.e. accounted for). This implies that, in this case, the total non-metered consumption flow rate, Q_{nc} , and total non-metered real loss flow rate, Q_{nrl} , are both assumed to be equal to zero. The resulting set of simulated total input flow rate, $Q_i^s(t, f_{nrl})$, curves, corresponding to the resulting flow rate values provided in Table 5.2, are illustrated graphically in Figure A4.1, as part of Appendix A4.

It should be noted that the totals provided at the bottom of Table 5.2 (as indicated by Σ), are not the values corresponding to the measured total daily input volume, V_i^m, and the simulated total daily input volume, V_i^s, since the totals in Table 5.2 represent merely the sum of each column's average flow rate (in ℓ/s) during every hour. However, since the totals in Table 5.2 could easily be converted to daily volumes by application of an appropriate conversion constant, their values could just as easily be compared with one another, with the same ultimate outcome. The difference (tolerance) between the totals in Table 5.2 for the various simulation columns, and the total for the measured column ($\overline{\eta_s}$ ≈ 0.84), were either less than, or equal to 5%, as should be clear from Table 5.2.

Table 5.3 provides a summary of the results obtained from the extended period simulation trials that were performed on the hydraulic model of the water distribution system that was used as part of the case study problem. An appropriate value for the average leakage constant, $K_l(f_{nrl})$, that corresponds to each trial value of f_{nrl} , is presented in Table 5.3. Furthermore, this table also provides the average values for the various components of $Q_i^s(t, f_{nrl})$, corresponding to each of the simulation trails that were performed.

Table 5.2: Total input flow rate curve values for selected trial values of real losses fraction

Time	$Q_i^m(t)$ [ℓ/s] for η_s =		$Q_i^s(t, f_{nrl})$ [ℓ/s] for f_{nrl} =					
	0.8	**1.0**	**0.0**	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**
00:00	2.43	2.03	2.24	2.36	2.54	2.61	2.83	2.89
01:00	1.85	1.55	2.24	2.37	2.55	2.63	2.85	2.92
02:00	2.26	1.89	2.24	2.37	2.55	2.63	2.85	2.92
03:00	3.63	3.03	3.19	3.29	3.44	3.49	3.68	3.71
04:00	3.48	2.91	2.85	2.96	3.12	3.18	3.38	3.42
05:00	5.56	4.64	5.50	5.52	5.60	5.57	5.68	5.64
06:00	7.37	6.16	7.26	7.23	7.25	7.16	7.21	7.11
07:00	8.24	6.89	8.96	8.87	8.83	8.69	8.68	8.53
08:00	10.35	8.65	10.52	10.38	10.29	10.09	10.03	9.83
09:00	11.54	9.64	12.01	11.82	11.69	11.44	11.33	11.07
10:00	10.93	9.13	11.13	10.97	10.86	10.64	10.56	10.33
11:00	10.09	8.43	10.72	10.57	10.48	10.27	10.20	9.99
12:00	8.98	7.51	9.98	9.85	9.78	9.60	9.56	9.37
13:00	8.41	7.03	8.96	8.87	8.83	8.68	8.67	8.51
14:00	8.00	6.69	8.62	8.54	8.51	8.37	8.37	8.23
15:00	7.81	6.53	8.89	8.80	8.76	8.62	8.61	8.45
16:00	7.17	5.99	7.19	7.16	7.17	7.09	7.13	7.04
17:00	7.33	6.13	7.40	7.36	7.36	7.27	7.31	7.20
18:00	8.83	7.38	7.40	7.35	7.36	7.27	7.31	7.20
19:00	8.13	6.79	6.92	6.89	6.92	6.84	6.90	6.80
20:00	7.65	6.39	6.58	6.57	6.60	6.53	6.60	6.52
21:00	5.74	4.80	5.57	5.58	5.65	5.61	5.71	5.67
22:00	4.07	3.40	3.87	3.94	4.06	4.08	4.24	4.25
23:00	2.87	2.40	2.65	2.76	2.92	2.98	3.18	3.23
00:00	2.67	2.23	2.24	2.36	2.54	2.61	2.83	2.89
Σ	162.96	136.19	162.89	162.36	163.11	161.33	162.86	160.81
% Δ (Σ)	0.00	16.43	0.04	0.37	0.09	1.00	0.06	1.32
μ	6.79	5.67	6.79	6.76	6.80	6.72	6.79	6.70
σ	2.88	2.41	3.12	3.02	2.91	2.81	2.71	2.60

Table 5.3: Summary of results from simulation trials performed on *CaseStudyReservoir*

$f_{nrl} = Q_{nrl} / Q_n$	$K_l(f_{nrl})$ [ℓ/s/m]	Average values for components of $Q_i^s(t, f_{nrl})$:			
		Q_{nrl} [ℓ/s]	Q_{mc} [ℓ/s]	Q_{nc} [ℓ/s]	Q_i [ℓ/s]
0.0	0.000	0.00	5.67	1.12	6.79
0.2	0.031	0.22	5.67	0.89	6.79
0.4	0.035	0.45	5.67	0.67	6.79
0.6	0.037	0.67	5.67	0.45	6.79
0.8	0.038	0.89	5.67	0.22	6.79
1.0	0.040	1.12	5.67	0.00	6.79

5.3.6. Estimation of Real Losses Fraction

Two distinct techniques for estimating the appropriate value of the real losses fraction, f_{nrl} , as part of the simulation aspect have been proposed in Chapter 4. The first technique has been referred to as the *method of standard deviations*, whereas the second was labelled as the *method of squared residuals*. Since each of these two techniques has been explained clearly in Chapter 4, they have simply been implemented directly for the case study problem in this section, without further discussion of their application procedures. It has also previously been stated that the user is responsible for selecting either one of the techniques as the more appropriate, based on personal preference or matter of opinion. However, for the sake of comprehensiveness, both techniques have been implemented for the case study problem.

For the *method of standard deviations*, Table 5.2 provides the appropriate standard deviation values (as indicated by σ), which were subsequently plotted against the selected set of trial values of f_{nrl} , as illustrated in Figure 5.6. The expression for the linear curve that was fitted to the series of data points is presented by Eq. (36).

$$y = -0.514\,x + 3.118 \tag{36}$$

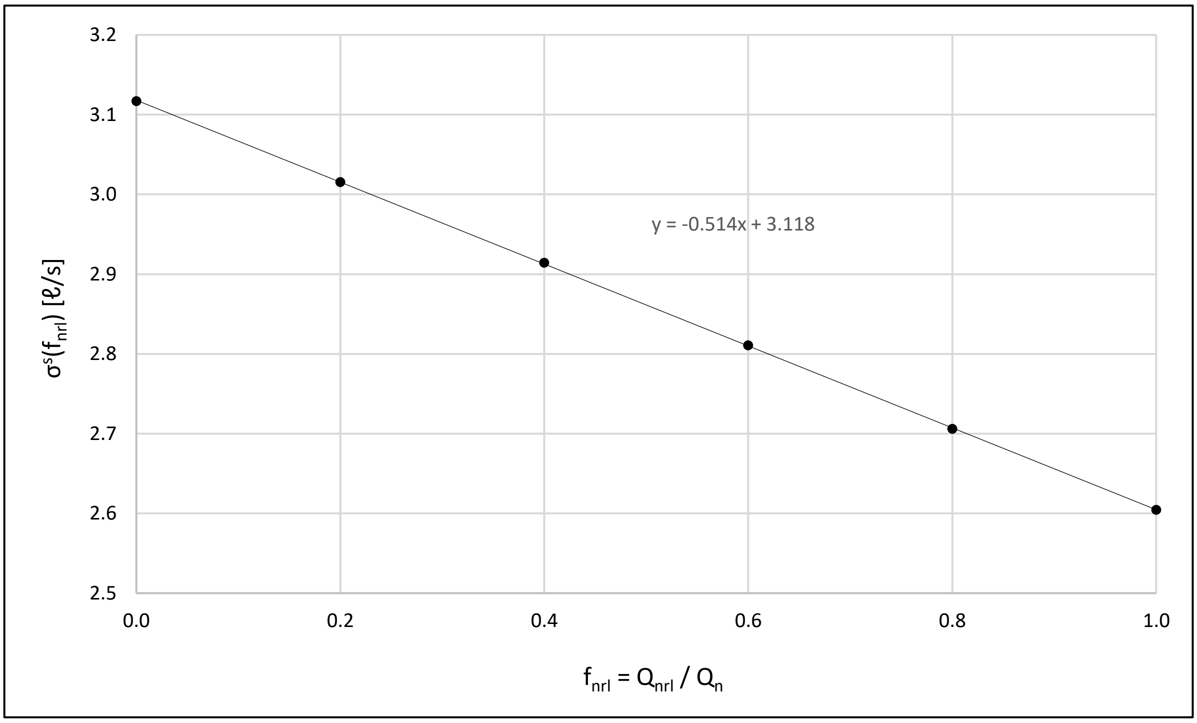

Figure 5.6: Estimation of real losses fraction using method of standard deviations

By making use of Eq. (36), the value of f_{nrl} that corresponded to a value of σ^m = 2.88 ℓ/s (from Table 5.2), was calculated as f_{nrl} ≈ 0.46. This means that, according to the *method of standard deviations*, approximately 46% of the water loss in the water distribution system of the case study problem is estimated to be due to leakage (real loss).

As an alternative, the *method of squared residuals* was also implemented, to estimate the best value of f_{nrl} . Like the *method of standard deviations*, an appropriate curve was fitted to the series of data points for this technique, as illustrated in Figure 5.7. The corresponding expression for this curve is presented by Eq. (37), which is a second-order polynomial function.

$$y = 0.636\, x^2 - 0.705\, x + 1.001 \tag{37}$$

By setting the derivative of the expression in Eq. (37) equal to zero, the minimum value of the curve illustrated in Figure 5.7 was found, corresponding to f_{nrl} ≈ 0.55. In other words, according to the *method of squared residuals*, approximately 55% of the water loss in the water distribution system of the case study problem is estimated to be due to leakage (real loss).

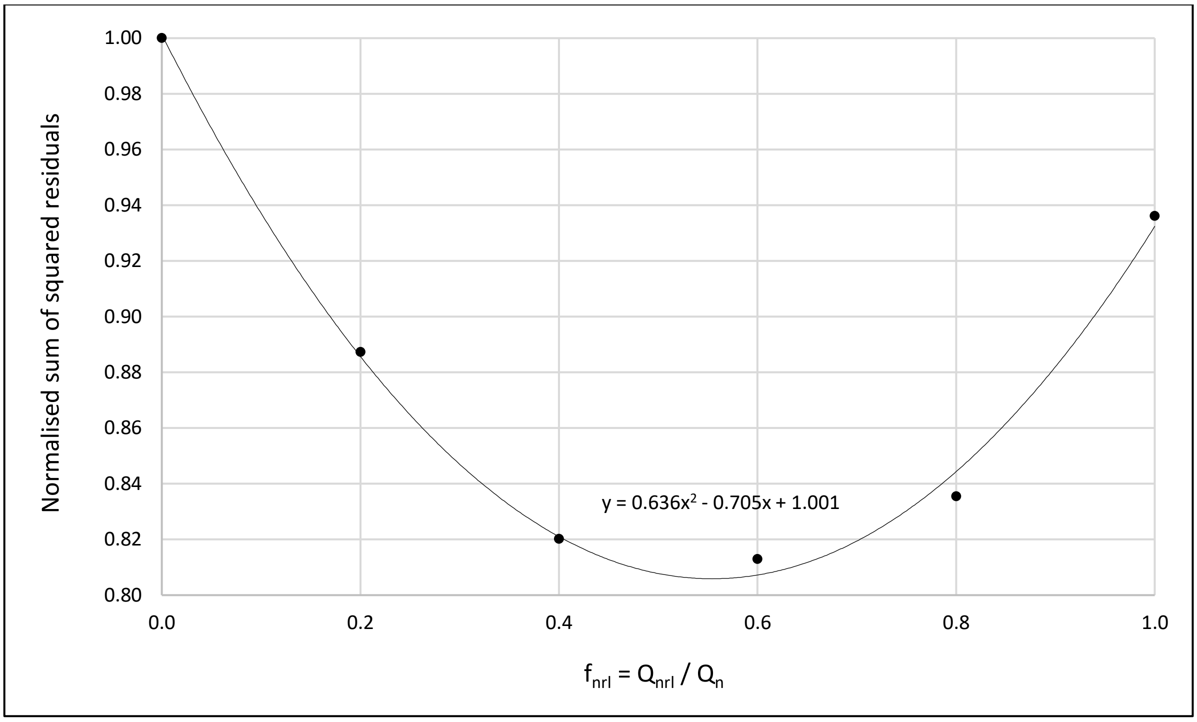

Figure 5.7: Estimation of real losses fraction using method of squared residuals

5.4. Discussion of Results

Two distinct techniques, namely the *method of standard deviations* and the *method of squared residuals*, were used to estimate the best value of the real losses fraction, f_{nrl} . These two techniques were implemented on a set of flow rates, which had resulted from several simulation trials that had been performed on a hydraulic model of the real-world water distribution system that was used as part of the case study.

The *method of standard deviations* estimated the value of f_{nrl} to be approximately 0.46, whereas the *method of squared residuals* estimated the value of f_{nrl} to be approximately 0.55. Since it has been stated before, that the user is responsible for selecting either one or the other of these techniques as the more correct, the second technique was selected in this research, based on the author's own preference. The *method of squared residuals* was selected, because it compares each of the corresponding flow rate values between the two distinct curves on an hourly level of precision. In contrast, the *method of standard deviations* considers only the overall standard deviations of two separate flow rate curves, which might, by chance, have comparable standard deviations, but very different profiles in a time-related sense.

According to the results obtained, leakage (real loss) from the water distribution system contributes to about 55% of the total non-metered water in the system. This value is considered high, since the components of non-metered consumption are also known to contribute significantly to total non-metered water in a distribution system. Two such examples, for the situation in South Africa, specifically, would be: (1) non-metered unauthorised consumption of water (i.e. theft); and (2) consumer meter inaccuracies, resulting from either vandalism or age-related deterioration of such water meters.

6. CONCLUSION

6.1. Summary of Findings

During the initial design process of a water distribution system, it is essential that the components of the total water demand being imposed on the system be estimated as accurately as possible. Leakage and non-metered consumption are relatively complicated to estimate accurately regarding both the extent thereof and the spatial distribution within a water distribution system. According to the literature reviewed, various methodologies are available for estimating leakage from water distribution systems and for modelling leakage as a function pressure (Van Zyl *et al.*, 2017). Some of these methodologies involve theoretical equations for pressure-leakage relationships, BABE principles, and the principles of FAVAD.

A number of computer software packages pertaining to leakage have been developed in the past, including: *SANFLOW* (McKenzie, 1999); *PRESMAC* (McKenzie & Langenhoven, 2001); *BENCHLEAK* (Seago *et al.*, 2007); and *ECONOLEAK* (McKenzie & Lambert, 2002). Different techniques are used in practice for the allocation of water loss to hydraulic models of water distribution systems. The consumption-dependent distribution is an example of such a technique, which involves distributing the total volume of water loss to the nodes of the hydraulic model that specifically have consumption values associated with them, as a percentage of the metered consumption at the relevant nodes. Almandoz *et al.* (2005) presented some findings pertaining to the allocation of non-metered water to nodes in a hydraulic model.

A computer-based modelling procedure for estimating aspects of leakage in hydraulic models of water distribution systems was developed as part of this research. The modelling procedure was titled *SEGLEAK* and was implemented on a hydraulic model of a real-world water distribution system that formed part of a case study problem, to illustrate the application of the *SEGLEAK* procedure. Two practical approaches to estimating the real losses fraction, f_{nrl} , were derived as part of the procedure.

6.2. Conclusions from Research

The first conclusion that can be made from this research is that comparison of the temporal profiles of the total input flow rate curves to a water distribution system could be used to distinguishing between components of leakage and those of other types of non-metered water (e.g. non-metered authorised consumption, apparent loss). Secondly, the estimate of the value of the real losses fraction, f_{nrl} , produced by the *method of standard deviations* could possibly differ from that produced by the *method of squared residuals*, as was evident from the testing of the *SEGLEAK* procedure on the real-world case study problem. From this study, it was clear that the *method of squared residuals* considers the hourly deviation of the temporal profile of the total input flow rate curve, whereas the *method of standard deviations* merely considers the daily average deviation of the temporal profile of the total input flow rate curve. The former of the two was, therefore, considered to be the more accurate method.

6.3. Suggestions for Further Research

This research study focused on the development of a procedure to practically model all non-metered water, including leakage and non-metered consumption, in hydraulic models of water distribution systems. Although some of the other components of non-metered water (excluding leakage) were incorporated as part of this research, the assumptions regarding the hydraulics theory were simpler for non-metered consumption components than for the leakage components. It could be beneficial to place a greater focus on the other components of non-metered water (i.e. apparent loss, non-metered authorised consumption) in a future study.

Many different contributing factors could influence the occurrence of leakage in water distribution systems. Many of these factors could include system dependent parameters, such as pipe diameter, pipe material, surrounding soil conditions, and the spatial density of service connections. This study addressed the effects of pressure on leakage, since pressure was expected to have the most significant effect on the extent to which leakage occurs. The length of mains was incorporated as being, in some measure, a contributing factor to the spatial distribution of leakage within a water distribution system. It would be interesting to know what measure of impact some other potential contributing factors have, especially regarding the extent and the spatial distribution of leakage. Future work could thus investigate the potential effects of some other contributing factors, such as pipe diameter, pipe material, surrounding soil conditions, and the spatial density of service connections.

Microsoft Office Excel was used for many of the manual calculations performed during this study and proved very necessary for that matter. Numerous adjustments of input values in *Excel* needed to be performed repeatedly in-between successive hydraulic simulation trials. The greater part of actual simulation trials had, however, been performed automatically in the *Wadiso* software package. Consequently, the process of alternating between the two software packages during simulation trials required a labour intensive, repetitive process from the point of view of the analyst. Future work involving the use of the *SEGLEAK* procedure could benefit from integration of *SEGLEAK* and hydraulic modelling software, such as *EPANET* or *Wadiso*.

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APPENDIX A

A1. Flow Logging Data for *CaseStudyReservoir*

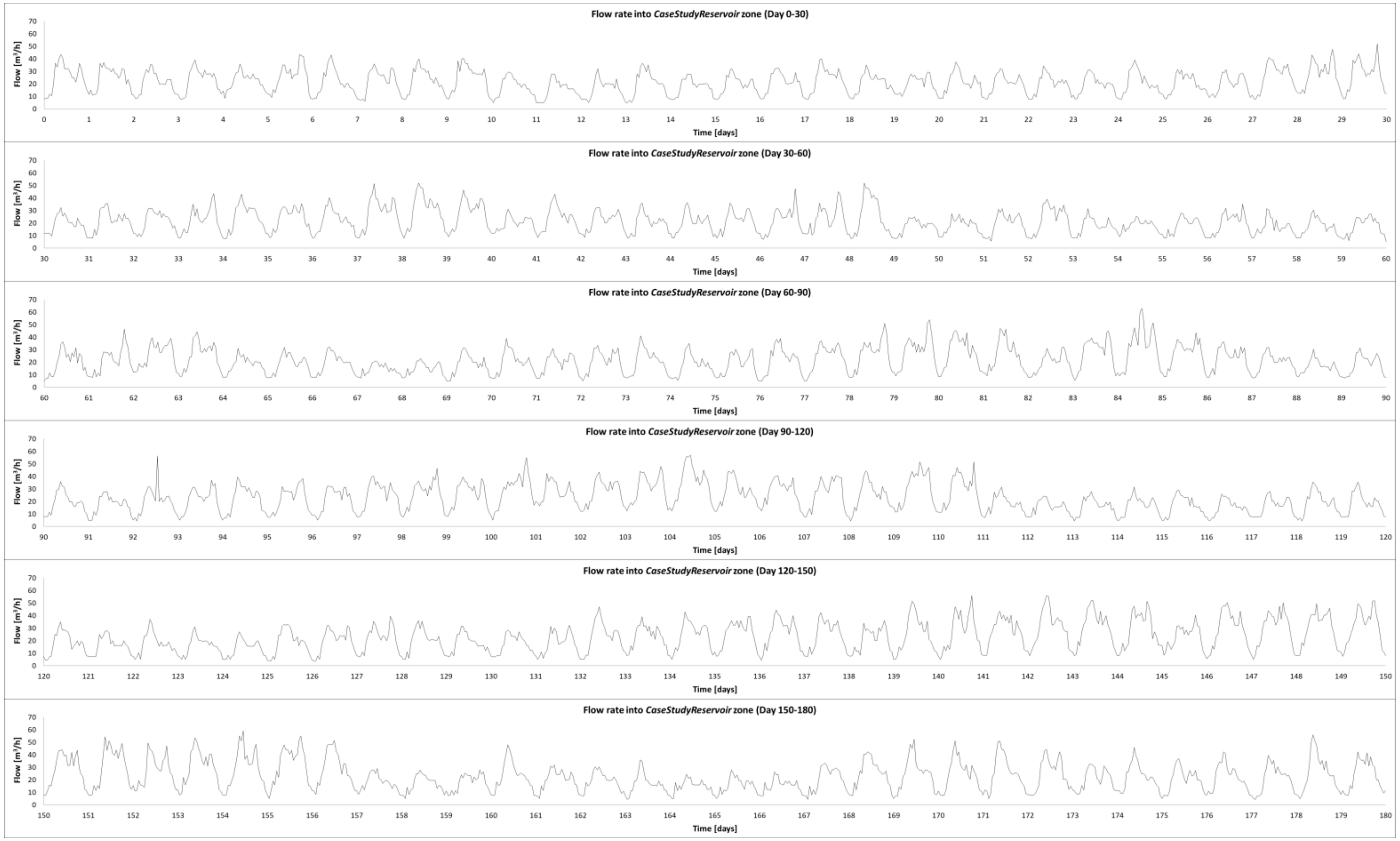

Figure A1.1: *CaseStudyReservoir* flow logging data

Table A1.1: Diurnal consumption patterns developed for *CaseStudyReservoir*

Time	Day of the week						
	Mon	Tue	Wed	Thu	Fri	Sat	Sun
00:00	0.36	0.40	0.40	0.45	0.38	0.46	0.52
01:00	0.30	0.35	0.35	0.34	0.34	0.38	0.48
02:00	0.37	0.37	0.41	0.44	0.41	0.40	0.49
03:00	0.57	0.61	0.65	0.56	0.58	0.63	0.72
04:00	0.51	0.52	0.56	0.51	0.51	0.49	0.49
05:00	0.76	0.82	0.83	0.90	0.79	0.64	0.62
06:00	1.14	1.14	1.24	1.21	1.18	0.77	0.71
07:00	1.31	1.33	1.38	1.35	1.38	1.21	1.09
08:00	1.64	1.62	1.64	1.55	1.70	1.53	1.52
09:00	1.82	1.66	1.65	1.64	1.80	1.70	1.48
10:00	1.80	1.55	1.50	1.46	1.60	1.65	1.49
11:00	1.69	1.37	1.41	1.43	1.48	1.45	1.35
12:00	1.45	1.24	1.27	1.26	1.35	1.36	1.27
13:00	1.29	1.17	1.26	1.20	1.18	1.24	1.14
14:00	1.17	1.15	1.16	1.16	1.28	1.21	1.11
15:00	1.16	1.20	1.08	1.22	1.15	1.21	1.12
16:00	1.11	1.10	1.04	1.03	1.14	1.34	1.13
17:00	1.00	1.12	1.10	1.11	1.09	1.22	1.34
18:00	1.13	1.24	1.33	1.25	1.12	1.24	1.48
19:00	1.07	1.26	1.17	1.27	1.14	1.15	1.48
20:00	1.03	1.16	1.10	1.23	1.03	1.04	1.34
21:00	0.87	0.93	0.88	0.92	0.82	0.86	1.01
22:00	0.64	0.80	0.68	0.65	0.65	0.72	0.68
23:00	0.45	0.49	0.48	0.45	0.51	0.58	0.52
00:00	0.37	0.40	0.44	0.39	0.40	0.50	0.42

A2. Hydraulic Model for *CaseStudyReservoir*

Table A2.1: *CaseStudyReservoir* length weighting factors

Node no.	Length weighting factor	Node no.	Length weighting factor	Node no.	Length weighting factor	Node no.	Length weighting factor
351003	0.0079	351058	0.0135	351105	0.0152	351148	0.0118
351004	0.0155	351059	0.0107	351106	0.0155	351149	0.0124
351007	0.0079	351062	0.0096	351109	0.0085	351151	0.0113
351012	0.0082	351063	0.0107	351110	0.0085	351154	0.0082
351014	0.0079	351064	0.0107	351111	0.0085	351155	0.0054
351015	0.0124	351065	0.0102	351113	0.0110	351156	0.0104
351016	0.0107	351067	0.0149	351114	0.0062	351157	0.0133
351018	0.0090	351068	0.0223	351116	0.0113	351161	0.0085
351019	0.0090	351071	0.0079	351122	0.0107	351162	0.0017
351021	0.0090	351072	0.0104	351123	0.0113	351163	0.0107
351022	0.0090	351074	0.0079	351125	0.0085	351164	0.0090
351025	0.0102	351076	0.0102	351127	0.0087	351165	0.0048
351026	0.0090	351077	0.0141	351131	0.0096	351166	0.0090
351034	0.0090	351079	0.0107	351132	0.0133	351172	0.0090
351035	0.0090	351084	0.0068	351133	0.0054	351173	0.0017
351038	0.0104	351085	0.0135	351135	0.0085	351174	0.0107
351041	0.0102	351086	0.0090	351136	0.0085	351175	0.0090
351042	0.0090	351087	0.0079	351137	0.0073	351176	0.0135
351043	0.0090	351091	0.0085	351139	0.0034	351177	0.0090
351045	0.0073	351092	0.0102	351141	0.0093	351181	0.0051
351046	0.0172	351093	0.0113	351143	0.0102	351182	0.0102
351052	0.0113	351095	0.0124	351144	0.0135	351183	0.0141
351053	0.0192	351098	0.0059	351145	0.0093	351184	0.0090
351054	0.0102	351103	0.0155	351146	0.0217	351185	0.0090
351055	0.0073	351104	0.0073	351147	0.0124		

Table A2.2: Output flow rates assigned to nodes in *CaseStudyReservoir* hydraulic model (1)

Node no.	Output flow rate [ℓ/s]					
	$f_{nrl} = Q_{nrl} / Q_n$					
	0.0	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**
351003	0.024	0.023	0.023	0.022	0.021	0.020
351004	0.015	0.015	0.014	0.014	0.013	0.013
351007	0.036	0.035	0.033	0.032	0.031	0.030
351012	0.021	0.020	0.020	0.019	0.018	0.018
351014	0.025	0.025	0.024	0.023	0.022	0.021
351015	0.048	0.046	0.045	0.043	0.041	0.040
351016	0.097	0.094	0.090	0.087	0.084	0.081
351018	0.073	0.071	0.069	0.066	0.064	0.061
351019	0.130	0.126	0.121	0.117	0.113	0.108
351021	0.017	0.017	0.016	0.016	0.015	0.015
351022	0.100	0.097	0.094	0.090	0.087	0.084
351025	0.052	0.050	0.049	0.047	0.045	0.044
351026	0.095	0.092	0.089	0.086	0.083	0.080
351034	0.101	0.097	0.094	0.091	0.088	0.084
351035	0.107	0.104	0.100	0.097	0.093	0.090
351038	0.058	0.056	0.054	0.052	0.050	0.048
351041	0.017	0.017	0.016	0.016	0.015	0.014
351042	0.081	0.078	0.075	0.073	0.070	0.067
351043	0.050	0.048	0.046	0.045	0.043	0.042
351045	0.087	0.084	0.081	0.078	0.075	0.073
351046	0.039	0.038	0.036	0.035	0.034	0.033
351052	0.063	0.061	0.059	0.057	0.055	0.053
351053	0.304	0.294	0.284	0.274	0.264	0.254
351054	0.089	0.086	0.083	0.080	0.077	0.074
351055	0.084	0.082	0.079	0.076	0.073	0.070
351058	0.039	0.038	0.036	0.035	0.034	0.033
351059	0.067	0.065	0.063	0.060	0.058	0.056
351062	0.102	0.099	0.096	0.092	0.089	0.085
351063	0.194	0.187	0.181	0.175	0.168	0.162
351064	0.020	0.019	0.019	0.018	0.017	0.017
351065	0.023	0.022	0.021	0.020	0.020	0.019
351067	0.039	0.038	0.036	0.035	0.034	0.033
351068	0.050	0.049	0.047	0.045	0.044	0.042

Table A2.3: Output flow rates assigned to nodes in *CaseStudyReservoir* hydraulic model (2)

Node no.	Output flow rate [ℓ/s]					
	$f_{nrl} = Q_{nrl} / Q_n$					
	0.0	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**
351071	0.108	0.105	0.101	0.098	0.094	0.090
351072	0.140	0.135	0.130	0.126	0.121	0.117
351074	0.089	0.086	0.083	0.080	0.077	0.074
351076	0.060	0.058	0.057	0.055	0.053	0.051
351077	0.056	0.055	0.053	0.051	0.049	0.047
351079	0.034	0.033	0.032	0.031	0.030	0.029
351084	0.047	0.046	0.044	0.043	0.041	0.040
351085	0.008	0.008	0.008	0.008	0.007	0.007
351086	0.021	0.020	0.020	0.019	0.018	0.017
351087	0.076	0.074	0.071	0.069	0.066	0.064
351091	0.023	0.022	0.022	0.021	0.020	0.019
351092	0.130	0.126	0.121	0.117	0.113	0.109
351093	0.084	0.081	0.079	0.076	0.073	0.070
351095	0.192	0.186	0.180	0.173	0.167	0.161
351098	0.088	0.085	0.083	0.080	0.077	0.074
351103	0.045	0.043	0.042	0.040	0.039	0.037
351104	0.076	0.073	0.071	0.068	0.066	0.063
351105	0.015	0.014	0.014	0.014	0.013	0.013
351106	0.089	0.086	0.083	0.080	0.077	0.074
351109	0.039	0.038	0.037	0.035	0.034	0.033
351110	0.063	0.061	0.059	0.057	0.055	0.053
351111	0.041	0.040	0.038	0.037	0.036	0.034
351113	0.028	0.027	0.026	0.025	0.024	0.023
351114	0.122	0.118	0.114	0.110	0.106	0.102
351116	0.090	0.087	0.084	0.081	0.078	0.075
351122	0.075	0.073	0.070	0.068	0.065	0.063
351123	0.152	0.147	0.142	0.137	0.132	0.127
351125	0.039	0.038	0.037	0.036	0.034	0.033
351127	0.048	0.047	0.045	0.044	0.042	0.040
351131	0.076	0.073	0.071	0.068	0.066	0.063
351132	0.068	0.066	0.064	0.061	0.059	0.057
351133	0.083	0.080	0.078	0.075	0.072	0.069
351135	0.071	0.069	0.067	0.064	0.062	0.060

Table A2.4: Output flow rates assigned to nodes in *CaseStudyReservoir* hydraulic model (3)

Node no.	Output flow rate [ℓ/s]					
	$f_{nrl} = Q_{nrl} / Q_n$					
	0.0	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**
351136	0.069	0.066	0.064	0.062	0.060	0.057
351137	0.019	0.019	0.018	0.017	0.017	0.016
351139	0.021	0.020	0.020	0.019	0.018	0.017
351141	0.046	0.045	0.043	0.041	0.040	0.038
351143	0.034	0.033	0.032	0.031	0.030	0.029
351144	0.013	0.013	0.012	0.012	0.012	0.011
351145	0.178	0.172	0.166	0.161	0.155	0.149
351146	0.173	0.168	0.162	0.156	0.150	0.145
351147	0.075	0.072	0.070	0.067	0.065	0.063
351148	0.010	0.010	0.009	0.009	0.009	0.008
351149	0.048	0.046	0.045	0.043	0.042	0.040
351151	0.031	0.030	0.029	0.028	0.027	0.026
351154	0.031	0.030	0.029	0.028	0.027	0.026
351155	0.052	0.050	0.049	0.047	0.045	0.044
351156	0.089	0.086	0.083	0.080	0.077	0.074
351157	0.018	0.017	0.017	0.016	0.015	0.015
351161	0.020	0.020	0.019	0.018	0.018	0.017
351162	0.102	0.098	0.095	0.092	0.088	0.085
351163	0.041	0.039	0.038	0.037	0.035	0.034
351164	0.110	0.106	0.103	0.099	0.095	0.092
351165	0.018	0.017	0.017	0.016	0.015	0.015
351166	0.092	0.089	0.086	0.083	0.080	0.077
351172	0.042	0.040	0.039	0.038	0.036	0.035
351173	0.137	0.132	0.128	0.123	0.119	0.114
351174	0.062	0.060	0.058	0.056	0.054	0.052
351175	0.093	0.090	0.087	0.084	0.081	0.078
351176	0.060	0.058	0.056	0.054	0.052	0.050
351177	0.038	0.037	0.036	0.034	0.033	0.032
351181	0.147	0.142	0.137	0.133	0.128	0.123
351182	0.079	0.077	0.074	0.071	0.069	0.066
351183	0.029	0.028	0.027	0.027	0.026	0.025
351184	0.005	0.005	0.005	0.005	0.004	0.004
351185	0.082	0.079	0.077	0.074	0.071	0.068

A3. Swift Data for *CaseStudyReservoir*

Table A3.1: ***CaseStudyReservoir*** **Swift data**

Node no.	AADD [kℓ/d]	AADD + UAW [kℓ/d]	Node no.	AADD [kℓ/d]	AADD + UAW [kℓ/d]	Node no.	AADD [kℓ/d]	AADD + UAW [kℓ/d]
21948	0.000	0.000	351062	7.387	9.234	351134	0.000	0.000
21949	0.000	0.000	351063	13.991	17.489	351135	5.148	6.435
21950	0.000	0.000	351064	1.438	1.798	351136	4.955	6.194
21953	0.000	0.000	351065	1.634	2.043	351137	1.388	1.735
21954	0.000	0.000	351067	2.819	3.524	351139	1.511	1.889
21972	0.000	0.000	351068	3.637	4.546	351141	3.323	4.154
21973	0.000	0.000	351071	7.815	9.769	351143	2.484	3.105
21974	0.000	0.000	351072	10.074	12.593	351144	0.959	1.199
21975	0.000	0.000	351074	6.391	7.989	351145	12.865	16.081
21987	0.000	0.000	351076	4.367	5.459	351146	12.505	15.631
21992	0.000	0.000	351077	4.069	5.086	351147	5.401	6.751
21994	0.000	0.000	351079	2.478	3.098	351148	0.724	0.905
351003	1.741	2.176	351084	3.423	4.279	351149	3.467	4.334
351004	1.112	1.390	351085	0.604	0.755	351151	2.219	2.774
351007	2.585	3.231	351086	1.508	1.885	351154	2.243	2.804
351012	1.530	1.913	351087	5.489	6.861	351155	3.768	4.710
351014	1.839	2.299	351091	1.664	2.080	351156	6.398	7.998
351015	3.443	4.304	351092	9.385	11.731	351157	1.287	1.609
351016	6.984	8.730	351093	6.071	7.589	351161	1.475	1.844
351018	5.306	6.633	351095	13.893	17.366	351162	7.339	9.174
351019	9.374	11.718	351097	0.000	0.000	351163	2.927	3.659
351021	1.254	1.568	351098	6.382	7.977	351164	7.936	9.920
351022	7.233	9.041	351103	3.216	4.020	351165	1.284	1.605
351025	3.767	4.709	351104	5.472	6.840	351166	6.669	8.336
351026	6.872	8.590	351105	1.082	1.353	351167	0.000	0.000
351034	7.276	9.095	351106	6.430	8.038	351172	3.011	3.764
351035	7.754	9.693	351109	2.822	3.528	351173	9.890	12.363
351038	4.164	5.205	351110	4.576	5.720	351174	4.509	5.636
351041	1.252	1.565	351111	2.953	3.691	351175	6.707	8.384
351042	5.823	7.279	351113	2.019	2.524	351176	4.303	5.379
351043	3.590	4.487	351114	8.780	10.975	351177	2.746	3.432
351045	6.270	7.838	351116	6.469	8.086	351181	10.626	13.283
351046	2.820	3.525	351122	5.432	6.790	351182	5.724	7.155
351052	4.553	5.691	351123	10.959	13.699	351183	2.123	2.654
351053	21.954	27.443	351125	2.849	3.561	351184	0.369	0.461
351054	6.391	7.989	351127	3.492	4.365	351185	5.918	7.397
351055	6.087	7.609	351131	5.462	6.828	351186	0.000	0.000
351058	2.820	3.525	351132	4.910	6.137			
351059	4.839	6.049	351133	5.992	7.490			

A4. Results from Analyses Performed

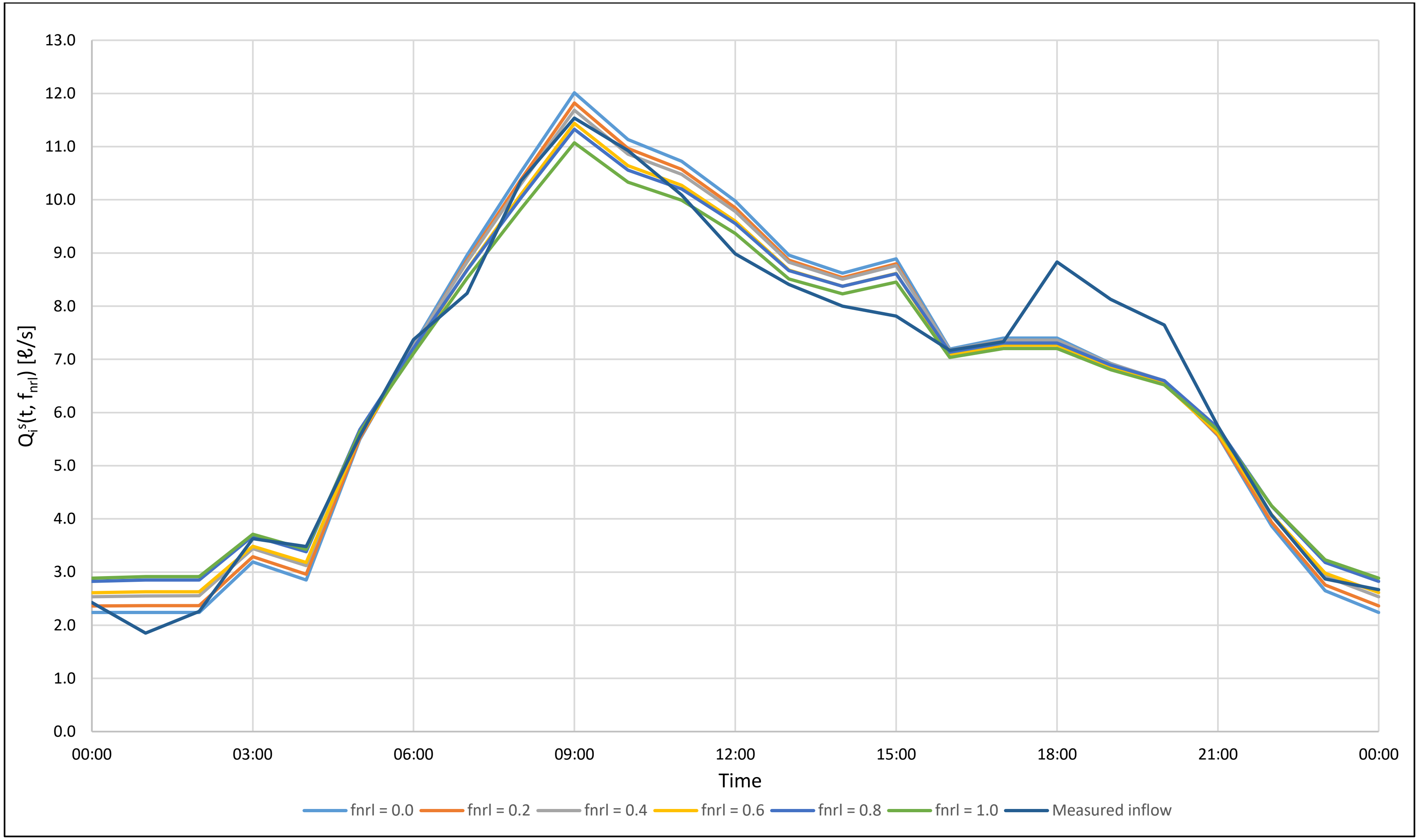

Figure A4.1: Total input flow rate curves for selected trial values of real losses fraction

Scale 1:6500

0 60 120 180 240 300 m

Legend:

- Bulk pipe
- Reticulation pipe
- Reticulation pipe (CaseStudyReservoir zone)
- Water distribution zone

Drawing_01

CaseStudyReservoir zone