EVALUATION OF MINIMUM PRESSURE HEAD DURING PEAK FLOW AS DESIGN CRITERION FOR WATER DISTRIBUTION SYSTEMS

by

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DECLARATION

By submitting this thesis electronically, I, J.L. Strijdom, declare that the entirety of the work contained therein is my own, original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

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ABSTRACT

One of the factors that drives infrastructure cost for water distribution systems (WDS), whether it being the cost of newly designed systems or the cost for maintaining and upgrading existing systems, is the hydraulic design criterion. The South African civil engineering fraternity has generally grown to accept the design criterion for water distribution systems as providing a minimum residual pressure head (MPH) of 24m at the most critical node in the system under theoretical peak demand conditions.

Previous studies have indicated that the above criterion is relatively stringent, especially due to the fact that theoretical peak factors used in design to simulate the peak demand condition are often too conservative. The aim of this study was to evaluate the criterion with the focus on the MPH value of 24m employed as a guideline in South Africa. As part of the study, current hydraulic models of existing South African WDSs were evaluated. In total, 71 towns located within 17 municipalities were included in this study. A total of 52 hydraulic models comprising a total of 539 388 modelled nodes were analysed. The number of nodes experiencing pressures below 24m was determined and the time spent at pressures under 24m was assessed. Furthermore, the consequences of nodal pressures decreasing to below the minimum local standards were investigated.

The results of the study confirmed previous findings that the current design criterion of 24m is too stringent and recommendations were made for water-providing authorities to relax the current design criterion. Three alternatives for a relaxed criterion were proposed.

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LIST OF ABBREVIATIONS AND ACRONYMS

AADD	average annual daily demand
AFU	automatic flushing urinal
AM	asset management
AWWA	American Water Works Association
AWWARF	American Water Works Association Research Foundation
CSIR	Council for Scientific and Industrial Research
DDA	demand-driven analysis
EMM	Ekurhuleni Metropolitan Municipality
GSS	Guaranteed Standards Scheme
H	residual pressure head (measured in m water)
HDA	head dependent analysis
IPF	instantaneous peak factor
IWA	International Water Association
KPI	key performance indicator
LM	local municipality
MM	metropolitan municipality
MPH	minimum pressure head (measured in m water)
MSH	maximum static head (measured in m water)
NWCA	National Water Consumption Archive
OFWAT	Office of Water Services (England and Wales)
PDA	pressure-driven analysis
PDF	peak day factor
PHF	peak hour factor
PWF	peak week factor
UWD	unit water demand
WDS	water distribution system
WRC	Water Research Commission
WSAA	Water Services Association of Australia

1 INTRODUCTION

1.1 Background

One of the factors that drives infrastructure cost for water distribution systems (WDS), whether it be the cost of newly designed systems or the cost for maintaining and upgrading existing systems, is the hydraulic design criterion. The residual pressure head (H) in a WDS, measured in meters water is used as a measure to evaluate WDSs. The South African civil engineering fraternity has generally grown to accept the design criterion for water distribution systems as providing a minimum residual pressure head (MPH) of 24m at the most critical node in the system under theoretical peak demand conditions. Using MPH during peak demand as design criterion for water distribution systems was previously researched by the author (Jacobs & Strijdom, 2009). This study is a further extension of the original published paper.

In South Africa a backlog has developed over the years in terms of providing basic water services to poor and disadvantaged communities. With the change in the political climate in the early and mid-1990s, promises were made to supply these communities with basic infrastructure. Although providing basic water services was a positive move in terms of improving living conditions, the additional water demand had a significant impact on the rest of the affected water networks. The bulk of municipal budgets was allocated to the expansion of the water networks in providing water to disadvantaged communities. Limited budget was allocated to the subsequent upgrading, maintenance and renewal required for the existing water networks. For all new water services that were installed, the backlog on upgrading and maintenance of the existing water services increased.

As the water networks kept on expanding with the incorporation of additional water demand and subsequently increased peak flow rates, the residual pressures in the affected networks kept on decreasing – often to pressures below the minimum requirements as stipulated within the current design criterion.

This study is applicable to both the design of infrastructure required to improve pressures that are currently below the design criterion for existing water distribution systems as well as to the design of completely new water distribution systems (e.g. the internal network for a new “greenfields” development). The focus of this study was shifted towards the design of upgrading requirements of existing WDSs.

1.2 Problem statement

The current South African WDS design criterion consists of two aspects: (i) the MPH value of 24m at the most critical node in the system as well as (ii) the philosophy of designing for the theoretical peak flow condition. When pressures decrease to below the minimum criterion, upgrading requirements to the existing WDSs are designed in order for the networks to adhere to the design criterion. Often a constraint is placed on the implementation of these upgrading requirements by the limited funds available on the municipal budgets. Although different income groups and different land use types have different water demand patterns and different needs in terms of residual pressure, the criterion of an MPH value of 24m at the most critical node in the system under theoretical peak demand is applied throughout for all scenarios in the design of water distribution systems and the required infrastructure is sized accordingly. This often leads to significant overspending on infrastructure.

A design criterion and philosophy, which were established more than 30 years ago by CSIR (1983) for circumstances and requirements which have since changed dramatically, are to date still being applied in South Africa with little cognisance of the effects it has on driving infrastructure cost. Given the limitations on the available budget for upgrading and maintenance, overspending is something that should be avoided at all costs.

1.3 Motivation

The hydraulic design criterion of a WDS is one of the factors that drives infrastructure cost. Evaluation of the criterion in terms of its suitability for South African consumer needs is therefore considered a priority.

Hydraulic water models are available for a wide variety of cities and towns in South Africa. Analysing the hydraulic results of available water models in terms of low pressure areas, according to the current design criterion and comparing the results with customer complaints received within these low pressure areas, gives an indication of whether the design criterion is suitable for the unique South African conditions and customer needs. Significant low pressure related customer complaints within areas that experience MPH above the minimum criterion value of 24m may suggest that the criterion value is too low. On the other hand, limited or no complaints within areas where the MPH is below 24m may suggest that the criterion is too stringent. Performing a statistical analysis on modelled nodal results is therefore a key requirement in evaluating MPH as a design criterion.

Furthermore, a comparison between the MPH criterion value of 24m and the minimum pressure required for normal day-to-day water consuming activities is required to either justify or challenge the 24m criterion value.

The current hydraulic design criterion sets the pressure requirement for the most critical node within a WDS for the most critical demand period. In evaluating the criterion it is therefore important to analyse how frequently the critical demand period occurs, for how long it lasts, and what ranges of pressures are experienced in the rest of the network. Performing time simulation analyses on the available hydraulic models and statistically analysing the results provided an understanding of pressures experienced at all nodes over a period of time.

1.4 Purpose of the study

The main purpose of this study was to encourage water-providing authorities in South Africa to consider relaxing the current design criterion to a lower MPH requirement and to revise the philosophy of designing for the theoretical peak flow condition.

1.4.1 Minimum MPH value of 24m

The first aspect of the current South African design criteria relates to the minimum MPH value of 24m head. During this study, numerous areas were identified where the results of the hydraulic analyses indicated minimum nodal pressures of less than 24m. For a selected number of towns (all towns within the Ekurhuleni Metropolitan Municipality), each of their respective water depots were visited and extensive workshops were held with the operational staff during which these “low pressure” areas were discussed. According to the operational staff, they received no customer complaints from most of these areas. Some complaints do, however, originate from areas where the pressure decreases to below 10m. If there are cases where customers are not complaining about low pressures, does this maybe suggest that they are accepting these pressures because they simply do not need higher pressures? In the “Literature review” chapter it is noted that a minimum pressure of only 10m is required for all household appliances to operate efficiently (Jacobs & Strijdom, 2009).

1.4.2 Duration of peak flow condition

The second aspect of the current design criterion relates to the philosophy of designing for the peak demand condition. If a water distribution zone is designed in theory to supply a MPH of 24m at the most critical node in the network under peak demand, then, theoretically, it would suggest that the most critical node (and maybe a few of the surrounding nodes) would experience a MPH of 24m and slightly above for only the peak hour condition – all

other nodes will experience an MPH in excess of 24m. For the remainder of the time, other than the peak demand hour, the most critical node within the zone will also experience an MPH in excess of 24m. This justifies an investigation into how many times the theoretical peak hour actually occurs and for how long it lasts. Booyens (2000) suggested that the theoretical peak hour condition is an unlikely flow scenario that occurs once a year and lasts for less than an hour.

1.4.3 Overestimation of peak flow

Given the fact that theoretical peak factors used for design (CSIR, 2003) have been reported to be conservative (Booyens & Haarhoff, 2002), together with the fact that the peak flow condition seldom occurs (Booyens, 2000), the current design criterion probably represents a scenario that may never occur in practice.

1.4.4 Consequence of MPH below 24m

If, in practice, it does occur that the critical nodes within a water distribution system experience an MPH of below 24m, what are the consequences of such low pressures and can these consequences be considered as catastrophical? It has already been noted that the MPH has to decrease to below 10m before certain household appliances will fail to operate efficiently (Jacobs & Strijdom, 2009). The consequences of MPH between 10m and the minimum requirement of 24m can therefore be limited to longer waiting times for filling of containers (baths, basins, water bottles, etc.) and less efficient irrigation systems. The latter can easily be addressed by adding additional irrigation points, having longer irrigation times or installing small booster pumps (which are very common in irrigation systems). The consequences of MPH values decreasing to below 24m (but not below 10m) are therefore not considered to be insurmountable.

1.4.5 Advantages of lower MPH criterion

1.4.5.1 Saving on infrastructure cost

The most obvious advantage of designing for a lower MPH criterion is the decrease in size of required new infrastructure and the associated decrease in costs. For new pipes this decrease in costs is twofold: firstly, there is a saving on the supply of the pipes (smaller pipes are cheaper) and secondly, a saving on the construction. Not only is the construction of a smaller infrastructure cheaper, but the construction of upgrading might become unnecessary if a specific existing pipe can supply a decreased MPH requirement.

1.4.5.2 Saving on energy cost

For networks that are supplied either from a water tower into which water is pumped or directly from booster pumps, a lower MPH requirement will require lower water towers and smaller booster pumps. Again, the construction of lower water towers will be cheaper, but more importantly, the lower required pumping head for the affected pumps in the system will result in a continuous saving in energy over the lifetime of the network. With the energy crisis currently being experienced in South Africa it is not only the intermittent power supply (load shedding) that needs to be taken into consideration but also the expected exponential increase in energy costs.

1.4.5.3 Saving on water use

South Africa is estimated to be the 30th driest nation on earth and the nation's water resources are under pressure due to lower than normal rainfall seasons experienced in the last decade (Hes, 21013). Crowley (2015) stated that South Africa is currently facing the worst drought in two decades. KwaZulu-Natal is the worst affected province with dam levels at 17.5%. Durban, South Africa's third-biggest city, started rationing water supplies in 2015 to save water. Water supply to the northern areas of the city will be restricted for six hours a day with immediate effect, the eThekwini Municipality said in a statement on 3 July 2015 (Crowley, 2015). According to estimates by National Treasury (2015), South Africa's water demand will exceed supply by 2030.

Reduced pressure in a water distribution system leads to consumers using less water. As the water yield of a tap, for instance, is directly dependent on the pressure in the pipe supplying the tap, less water is used by consumers during their day-to-day water consumption activities if the pressure is lower. For instance, consumers tend to use less water for filling baths and basins because the filling time is longer. Similarly the total of volume of water used for irrigation purposes is less for the same period of time when the pressure is lower.

1.4.5.4 Saving on water losses

Leaks in South African water systems are costing the economy approximately R7.2-billion per year (Hes, 2013). Because a leak in a water distribution system pipe can be considered as an orifice (Van Zyl, 2014), the flow rate Q through a leak can be calculated by the well understood and widely used orifice equation as a function of the pressure head h as follows:

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

From the orifice equation it can be concluded that the higher the pressure in a water distribution system, the higher the leakage component of water losses in the system will be. The above is true for orifices with a fixed area. Furthermore, the equation also shows that increasing the area A of the leak increases the flow rate through the leak. Higher pressure can also increase the area A of the leak (Cassa & Van Zyl, 2014) for deforming pipes. Therefore, in practice, the increasing effect of high pressure on leakage could be exacerbated even further due to the increasing effect on the area of the leak.

As decreasing the MPH requirement will decrease pressure in a water system, the volume of water that is being lost through leakage can be reduced significantly. Although water-providing authorities spend a significant amount of effort and cost on finding and repairing leaks, many leaks are never found and the loss of water and revenue is therefore a continuous expense over the lifetime of such affected water systems. Many different methods are currently being applied by water-providing authorities to reduce water loss. Van Zyl and Sheppard (2015) stated that pressure reduction has proven to be the method with the most significant immediate impact on reducing water loss. Moult (2015) supported the statements made by Van Zyl and Sheppard by confirming that the same trends were observed in the Johannesburg water loss reduction projects.

The purpose of this study was therefore to investigate both of the aspects of the current design criterion. Firstly, the minimum MPH value of 24m was investigated. The following questions needed to be answered: Is such a high MPH value really required? What are the consequences when systems do experience pressures below this minimum value? What is the current status of existing South African water systems in terms of the number of modelled nodes that do experience MPH values of below 24m? Secondly, the design philosophy was investigated in terms of whether it is logical to design for the peak hour condition – a condition that might, in practice, never occur within a specific water distribution zone and when it does occur it only theoretically reduces the pressure at one single node, the most critical node in the system, to 24m.

The indirect purpose of this study is to encourage South African water authorities to investigate a possible alternative to the current design criterion – one which is less robust, more comprehensive, more relaxed, more customer-specific and most importantly, one which will result in more cost-effective designs, less water being used, less water being lost and less energy being used while maintaining satisfactory levels of service.

1.5 Brief summary of research objectives

The following research objectives were set for this study:

- To conduct a comprehensive literature review with regard to the following:
    - Historical design criterion for water distribution systems in South Africa as well as current criterion used internationally
    - Design philosophy for water distribution systems
    - Negative consequences of high system pressure.
- To obtain or compile hydraulic water models for a variety of existing South African water distribution systems and to populate these models with the following:
    - The latest actual measured water demand figures obtained from the respective treasury systems (to be used for existing system analysis)
    - Theoretical water demand figures based on actual demand figures, land use type and property size (to be used for future system analysis)
    - Theoretical peak factors commonly used in design.
- To perform steady state hydraulic analyses on the existing water systems under current peak demand conditions. In addition, to evaluate the hydraulic results in terms of the number of modelled nodes experiencing peak demand pressures below the minimum criterion of 24m head.
- To perform a time-simulation hydraulic analysis on one of the selected existing water models and to evaluate the hydraulic results in terms of the number of modelled nodes experiencing peak demand pressures below the minimum criterion of 24m head and the amount of time these nodes are subjected to pressures below 24m.
- To perform time-simulation hydraulic analyses on typical theoretical optimised models and to evaluate the hydraulic results in terms of the amount of time that nodes are subjected to pressures below 24m when the MPH at the critical node is reduced to 15m during peak demand.
- To perform a complete master plan for one of the selected hydraulic models based on the existing design criterion of 24m head and to compile a comprehensive cost estimate for all upgrading work required.
- To re-perform the complete master plan for the above selected hydraulic model based on a relaxed design criterion of 15m head, including a comprehensive cost estimate and to compare the difference in the upgrading cost requirement.

- To propose a possible alternative to the MPH value of 24m as design criterion as well as to propose a possible alternative to the current design philosophy of using MPH during theoretical peak demand (under steady state demand driven analysis) as design philosophy.

1.6 Limitations of the study

1.6.1 Fire flow

This study has not taken into account the possible limitations of a modelled network to supply the regulatory fire flow when a “relaxed” design criterion is applied to design the system.

The current South African water distribution design criterion stipulates that potable water supply systems should have the capacity to provide water for firefighting purposes. The decision to enforce these requirements is certainly a topic that should be thoroughly investigated and debated. Because the required flows for firefighting purposes considerably exceed the peak demand flow under normal circumstances, these fire flows generally govern the design of the water distribution system.

Snyder et al. (2002) investigated the impacts of designing a potable water distribution system to supply fire flow and found that, apart from the obvious increases in cost for the larger required infrastructure, larger infrastructure also have a degrading effect on water quality due to the increase in water age in the system. They proposed a number of firefighting alternatives including automatic sprinkler systems, on-site fire storage and dual systems, each of which have their own set of advantages and disadvantages.

Certain South African water-providing authorities have acknowledged the fact that their potable water systems cannot always provide the regulatory fire flow requirements in the more critical areas of the network (Anonymous, 2015) and that designing and implementing an upgrade to meet these requirements would not be possible due to the significant cost implications. However, for all new developments within their areas of jurisdiction they investigate whether fire flow can be supplied from the potable water system. If it is found that fire flow cannot be supplied, they require the developers to provide on-site storage for firefighting purposes. Furthermore, Myburgh and Jacobs (2014) found that only 8.6% of all fires within their study area were extinguished using water from the potable water system. They found that the majority of fires were extinguished by means of water ejected from pre-filled tanker vehicles.

1.6.2 Use of theoretical peak factors

Actual measured monthly water meter readings were used for this study to populate the water demands in the analysed hydraulic models in order to model a scenario as close as possible to the actual scenario. The actual water meter readings are only sufficient to calculate an average annual daily demand (AADD). When performing the hydraulic analysis for the peak demand hour, these actual demands have to be multiplied by theoretical peak hour factors (PHF) to derive a peak flow. The theoretical peak factors used in this study are discussed in Chapter 3.

1.6.3 Demand-driven analysis

For all hydraulic analyses performed in this study, the commercial software package WADISO 5.11.0629 (GLS, 2015) was used. WADISO uses the standard EPANET engine to perform a demand-driven hydraulic analysis (DDA). A demand-driven analysis first imposes the demands on the network and then analyses the resulting pressures, meaning that demands are known functions at time and are independent of the pressure in the system. The relationship between pressure and demand is thus ignored (Cheung et al., 2005), making it unsuitable for systems with low pressures where the fixed demand causes theoretical nodal pressure to decrease to below zero. Such negative pressures are typically a result of over-estimation of the theoretical peak factors.

Although software packages are available to perform analyses that incorporate the relationship between demand and pressure (referred to as pressure driven analysis – PDA), none of these were available during this study. Furthermore, an extension of the standard EPANET solver engine exists that directly includes pressure demand modelling. In this extension the data structures and algorithms within EPANET source code are modified in such a way that it assumes fixed demand above a given critical pressure, zero demand below a given minimum pressure and some proportional relationship between pressure and demand for intermediate pressures (Cheung et al., 2005). As this extension of EPANET was not available for the purposes of this study, a manual exercise similar to the above EPANET extension was performed on the hydraulic models where the DDA resulted in negative nodal pressures. The modelled peak demand of all the affected nodes within the specific distribution zone was reduced incrementally until a “realistic” minimum of H>0 was reached (Jacobs & Strijdom, 2009).

As described in a similar paper (Wagner et al., 1988), a simulation method was proposed where “Nodes are targeted to receive a given supply at a given head. If this head is not attainable, supply at the node is reduced”.

2 LITERATURE REVIEW

2.1 Review of pressure head criterion

Minimum pressure head and maximum pressure head are parameters that can easily be obtained from modelled simulation results and are quantifiable. The result is that both parameters are an obvious choice as meaningful performance indicators for water-providing authorities. MPH is used worldwide as criterion for the design of water distribution systems. An investigation into MPH as design criterion in South Africa and other countries was conducted as part of this research.

2.1.1 South African local criterion

An investigation into the history of design criterion for the MPH in water distribution systems in South Africa shows that an MPH of ± 24 m has long since been the norm, despite some changes to the criterion over the years.

During this literature review the earliest reported MPH criterion in South Africa that could be traced was by Leslie (1957) and suggested an "absolute minimum" of 12m for low-income and 15m for high-income areas. These values became outdated with improved standards of living during the 1970s. By the mid-1970s the MPH criterion published in guidelines had increased to 25m.

The criterion of H > 24m was reinforced in 1983 when the Council for Scientific and Industrial Research (CSIR) published the “Guidelines for the Provision of Engineering Services in Residential Townships”, or more commonly known as the “Blue Book”, which included the MPH criterion (CSIR, 1983) as indicated in Table 2.1.

Table 2.1: Blue Book MPH standards (CSIR, 1983)

Types of development	MPH
High income	24m
Middle income	24m
Low income	12m

The “Blue Book” was revised and is commonly known today as the “Red Book” (CSIR, 2003). The MPH criterion table had been slightly adjusted, as indicated in Table 2.2 (CSIR, 2003).

Table 2.2: Red Book MPH standards (CSIR, 2003)

Types of development	MPH
Dwelling houses (house connections)	24m
Dwelling houses (yard taps + yard tanks)	10m

The wide publicity and use of the latter document series between 1983 and 2003, combined with the fact that the four last published criteria for MPH were either 24m or 25m, has resulted in the South African civil engineering fraternity generally accepting 24m as the design criterion for MPH in reticulation networks.

Many of the large metropolitan municipalities in South Africa use in-house criteria. The MPH values stipulated by the city of Tshwane, for example, are summarised in Table 2.3 (City of Tshwane, 2010).

Table 2.3: City of Tshwane standards

Flow condition	Min/Max pressure	Absolute Min/Max pressure
	(m)	(m)
Peak hour demand - minimum	20 to 24	16 to 20
Static (no demand) - maximum	90	120

The Ekurhuleni Metropolitan Municipality (2007) states the following in their local design guidelines: “The minimum allowable pressure under peak demand is 25m while the maximum pressures under no-flow conditions (static) are not to exceed 90m in the system. A minimum pressure of 15m may be acceptable for small areas provided the consequences are not unreasonable”.

The guidelines used by the City of Cape Town (2011) and the West Coast District Municipality (2013) are incidentally the same as used by the City of Tshwane (2010).

2.1.2 Minimum pressure requirement for appliances

Apart from the guidelines stipulating exactly what the MPH requirement in the water reticulation network is, the South African National Standards' (SANS – previously known as SABS) building regulations indicates the MPH required for certain domestic appliances to operate adequately (SANS, 2012). Among the sanitary fixtures and fittings the most critical item seems to be the water closet with an automatic shut-off flush valve (pressure flush toilet) with an MPH requirement of 20m.

Jacobs and Strijdom (2009) researched the minimum pressure requirements for appliances and stated that some end-users require a minimum pressure to operate, thus setting a physical lower limit for H in water networks. The question immediately arises, 'What is this lower limit?' If such a value were to exist it would dictate the MPH required in a system, thus justifying a brief review of appliance specifications. Various domestic appliances require a minimum pressure to operate satisfactorily.

A few examples of end-users with a minimum pressure requirement are summarised in Table 2.4.

Table 2.4: End-user appliance minimum specifications (Jacobs and Strijdom, 2009)

Appliance	Minimum required pressure head	Comments
Pop-up irrigation systems	H ≥ 20m	The installation of a small booster pump in the irrigation system is recommended by suppliers if this pressure is not available
Washing machines and dish washers	H ≥ 10m	This pressure is used as a typical customer guideline by local furniture suppliers
Pressure flush toilets		Commercially known in South Africa as "Flush Master" toilets; relatively uncommon in South Africa
Back entry type	H ≥ 15m	
Top entry type	H ≥ 20m	

The requirement for pop-up irrigation systems is the most significant with H > 20m, but this is not considered critical in view of minimum reticulation network pressure requirement, because such personal irrigation systems are easily boosted by small pumps at an insignificant cost to the owner. Irrigation systems are often boosted in this manner despite the availability of sufficient system pressure. This is particularly true when an alternative personal on-site water resource (e.g. borehole water, greywater or rainwater) is used for garden irrigation in addition to municipal supply.

Pressure flush toilets require about 15m pressure to operate effectively. However, considering the fact that pressure flush toilets are not very common in South Africa and could be replaced in critical areas with cistern-type flush toilets if the need arises, the MPH-requirement for toilets could be put aside for the moment.

The 10m requirement for washing machines and dishwashers remains. Some sources report lower H values for specific washing machines and dishwashers (H > 8 m). Also, some appliance manufacturers supply custom-designed equipment able to operate at even lower pressures, but such devices are an exception to the rule and are unlikely to be used widely by consumers in South Africa.

From the information available it is apparent that a system pressure of less than 10m could be regarded as insufficient in view of appliance requirements in residential areas of South Africa.

Schools and other public buildings often make use of automatic flushing urinals (AFU) or pressure-flush toilets as is the case for domestic use. AFUs are considered to be old and are banned in many areas (e.g. Overstrand Municipality and the City of Cape Town) due to their inefficient use of water. In limited cases these devices are still operational, but they were not considered a driver of the MPH-criterion for the purpose of this study.

Agricultural crop irrigation in serviced areas would require an MPH for efficient irrigation of crops. In some cases water is used for crop irrigation on either a private scale or commercial scale within urban areas and such areas would have to be identified separately in guidelines for MPH in networks. In such cases the irrigation system is designed to ensure a certain application rate (flow rate) and is dependent on the supply pressure in the water system. A head lower than required would result in two problems: low application rates and insufficient water reaching the crops. The irrigation radius of sprinkler systems would be reduced by the low pressure in comparison to design values resulting in crops far from the irrigation point

receiving no water at all. However, this type of water use is limited in South African urban areas and it was considered to fall beyond the scope of this study.

2.1.3 International criterion

The American Water Works Association (AWWA) is the water-governing body of the United States of America. Several publications are available wherein their design criteria are stipulated. The city of Richmond (Virginia, USA) published a design guidelines document (Department of Public Utilities, 2011) wherein it is stipulates that all designs shall conform to the latest revisions of AWWA after which it specifically stipulates that "…the minimum allowable service pressure during maximum hourly demands shall be 35psi…". A pressure of 35psi, when converted, calculates to approximately 24m head, which correlates well with the current South African criterion.

The Ohio State University (Undated) referenced an MPH requirement of 24m for adequate flow for residential areas and 29m for excellent flow to a 3 storey building.

Two water distribution systems types are considered in Australia, namely a potable water distribution system and a recycled water distribution system. The minimum and maximum pressure criterion for these two systems is exactly the same. The Australian water-governing body, the Water Services Association of Australia (WSAA, 2007), published a document in which the following tables regarding minimum pressure requirements are included.

Table 2.5: Australian water supply network design criterion (WSAA, 2007)

Measure	Values to be achieved
Minimum pressure	22m
Maximum pressure	80m
Target maximum pressure	50m
Target pressure differential	10m

City of Gold Coast, the local government area spanning the Gold Coast, Queensland and surrounding areas in Australia have several statutory water authorities that are governed by an independent board. These authorities include Allconnex, Queensland Urban Utilities and Unity Water. City of Gold Coast (2012) published the following table summarising their planning guidelines:

Table 2.6: City of Gold Coast water planning guidelines (City of Gold Coast, 2012)

Parameter	Allconnex Water			Queensland urban utilities	Unity water
Pressure	**Gold Coast**	**Logan**	**Redland**		
Desired minimum SERVICE pressure urban and rural normal operating conditions	22m at property boundary (under normal operating conditions)	21m at property boundary (under normal operating conditions)			
In areas defined by the SP, properties with domestic private booster pumps	12m at suction side of private booster				
Desired maximum SERVICE pressure	80m = maximum service pressure; 55m = TARGET service pressure			60m	

The 22m and 21m MPH indicated in Table 2.6 stipulated for desired minimum service pressure under normal operating conditions correlate well with the 22m MPH required by WSAA (2007).

In Colombia, South America, Colombian legislation ranks the cities and towns according to population and economic capacity of the citizens in classified groups called “system complexity levels.” For each of these levels, the minimum pressures are as indicated in Table 2.7 (Saldarriaga et al, 2009).

Table 2.7: Colombian MPH standards (Saldarriaga et al, 2009).

System complexity level	MPH
Low level (less than 2500 inhabitants)	10m
Medium level (between 2501 and 12500 inhabitants)	15m
Medium high level (between 12501 and 60000 inhabitants)	20m
High level (More than 60001 inhabitants):	20m in residential areas
	25-30m in commercial or industrial

In Canada, the City of Vernon stipulates the MPH requirement as 29psi in their guidelines (Greater Vernon Water, 2013) which converts to 20m head.

In the United Kingdom there is no specific 'design criterion'. The water industry is regulated by a regulating body entitled Office of Water Services (OFWAT) which stipulates the requirements that water-providing companies in England and Wales have to meet. OFWAT

uses a set of performance indicators which is used to measure the performance of water-providing companies. The companies' performances are evaluated annually and the reports including the results are made available online (OFWAT, 2014). One of the performance indicators referred to in these reports is the percentage of "Properties at risk of low pressure", but no specific indication is given as to exactly what pressure is considered to be "low pressure". OFWAT (2014) does not officially provide minimum pressure criterion for water-providing companies — it is up to the specific company to use the reference value(s) that will ensure satisfactory performance in terms of the performance indicator.

According to Kapelan (2009) the most water-providing companies in England and Wales use a reference value of 15m (on the water pipe in the street) but this can vary from about 10m to about 20m.

Apart from the above reference values, OFWAT does make use of a guaranteed standards scheme (OFWAT, 2008) whereby customers are paid out financially by a scheme when certain absolute minimum criteria are not adhered to. According to the regulations of this scheme, water-providing companies must maintain a minimum pressure of 7m in the communication pipe. If pressure falls below this on two occasions, each occasion lasting more than one hour, within a 28-day period, the company must automatically make a Guaranteed Standards Scheme (GSS) payment.

Following direct correspondence with OFWAT regarding minimum pressure values for their performance indicator entitled "Properties at risk of low pressure", the following response was emailed by their official enquiries team (Bannister, 2011):

> "Water supply companies are required to supply water at a constant pressure which will reach the upper floors of houses. This does not apply to buildings that use pumped systems, such as blocks of flats. Performance is measured against a standard set by OFWAT. This is called a Level of Service Indicator (known as DG2) and comprises ten metres head of pressure [1 bar] at the external stop tap at a flow of nine litres per minute. This should be sufficient to fill a one gallon container in thirty seconds".

For China, the World Bank has indicated that about 13% of urban water users receive water at inadequate pressure (Browder, 2007). However, no reference is made to what exact minimum pressure value was deemed "inadequate". For Vietnam, the national design standards indicate an MPH of 10m (Government of Vietnam, 2006).

In conclusion the references found to MPH requirements during this study are summarised in table 2.8.

Table 2.8: Summary of MPH requirements

Country, Region, City	Citation	MPH (m)
South Africa (Countrywide)	Leslie (1957)	12 - 15
	CSIR (1983)	12 - 24
	CSIR (2003)	10 - 24
South Africa, City of Tshwane	City of Tshwane (2010)	16 - 24
South Africa, Ekurhuleni	EMM (2007)	15 - 25
USA, Richmond (VA)	Department of Public Utilities (2011)	24
USA, Ohio	The Ohio State University (Undated)	24 - 29
Australia (countrywide)	WSAA (2007)	22
Australia, Gold Coast	City of Gold Coast (2012)	22
Australia, Logan		21
Australia, Redland		21
United Kingdom (Countrywide)	OFWAT (2008)	7
	Bannister (2011)	10
	Kapelan (2009)	10 - 20
Canada, Vernon	Greater Vernon Water (2013)	20
Columbia (Countrywide)	Saldarriaga et al (2009)	10 - 30
Vietnam (Countrywide)	Government of Vietnam (2006)	10

2.2 Design philosophy for water distribution systems

Under the current design criterion for water distribution systems, the philosophy of designing for the theoretical peak demand condition is adopted. Booyens (2000) found that the peak flow condition lasts for a very short time, maybe only an hour per year. Booyens (2000) furthermore investigated measured peak factors versus peak factors most commonly used in designing water distribution systems (CSIR, 2000) and found that measured peak factors are significantly lower than the peak factors provided by CSIR (2000).

Similar conclusions that the CSIR (1983) peak factors used for design are over conservative were made during other related studies such as Van Vuuren and Van Beek (1997) who measured peak factors in Pretoria. The conclusion published in the resulting report was that the measured peak factor of 2.75 was approximately 31% lower than the CSIR (1983) recommended peak factor of 4.0 for the same amount of equivalent erven. In the same year

Turner et al. (1997) analysed water demand logging data of 14 different areas within the Gauteng province over a period of 20 months. The 14 different areas logged represented smaller and larger zones as well as higher income and lower income consumers. The peak factors measured by Turner et al. (1997) were lower than the CSIR (1983) recommended peak factors for all measured zones.

In a third similar study (Hare, 1989) two different residential areas in Port Elizabeth were investigated. Hare (1989) found that for these areas with 62.5 and 820 equivalent erven respectively the measured peak factors were 35% and 20% lower than the CSIR (1983) peak factors for the same equivalent erven.

Booyens and Haarhoff (2002) went further and studied three individually metered bulk zones in Boksburg for which continuous flow logging data via the telemetry system was available. Within these three bulk zones two smaller sub-zones could be isolated and individually metered. The logging results were analysed and using FlowCalc software peak factors for different time intervals ranging from 1 minute to 240 minutes were derived. These, in turn, were used to determine the 99%, 98%, 95% and 90% percentiles for the various peak factors as graphically plotted in Figure 2.1.

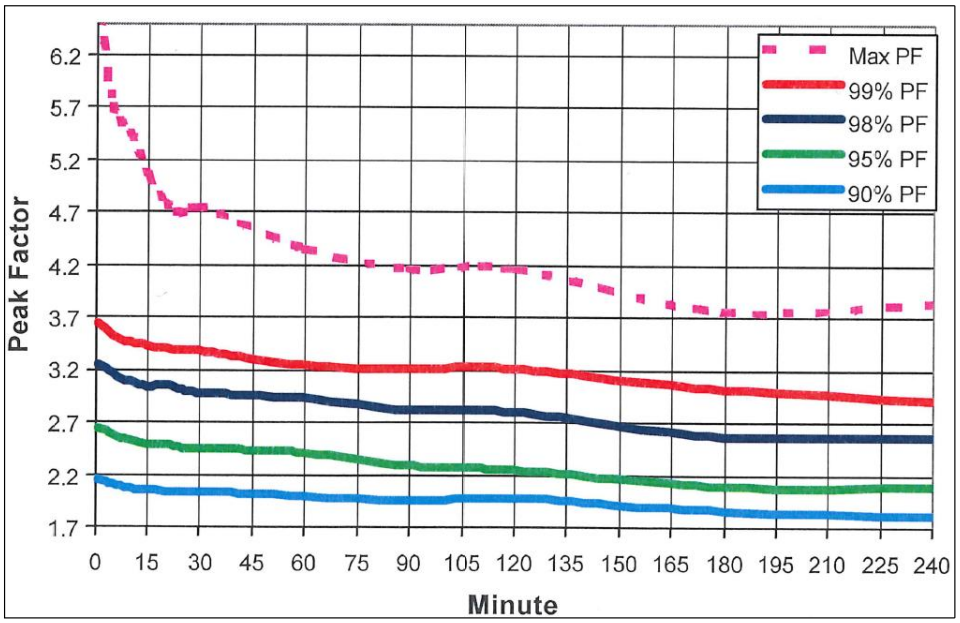


Figure 2.1: Peak factors for different time intervals (Booyens and Haarhoff, 2002)

Using the Weibull method for calculating return periods and the Gumbel distribution as a linearization technique, probabilistic peak factors for different time periods could be

determined by Booyens and Haarhoff (2002) and are shown in Figure 2.2 against the CSIR (2000) peak factors.

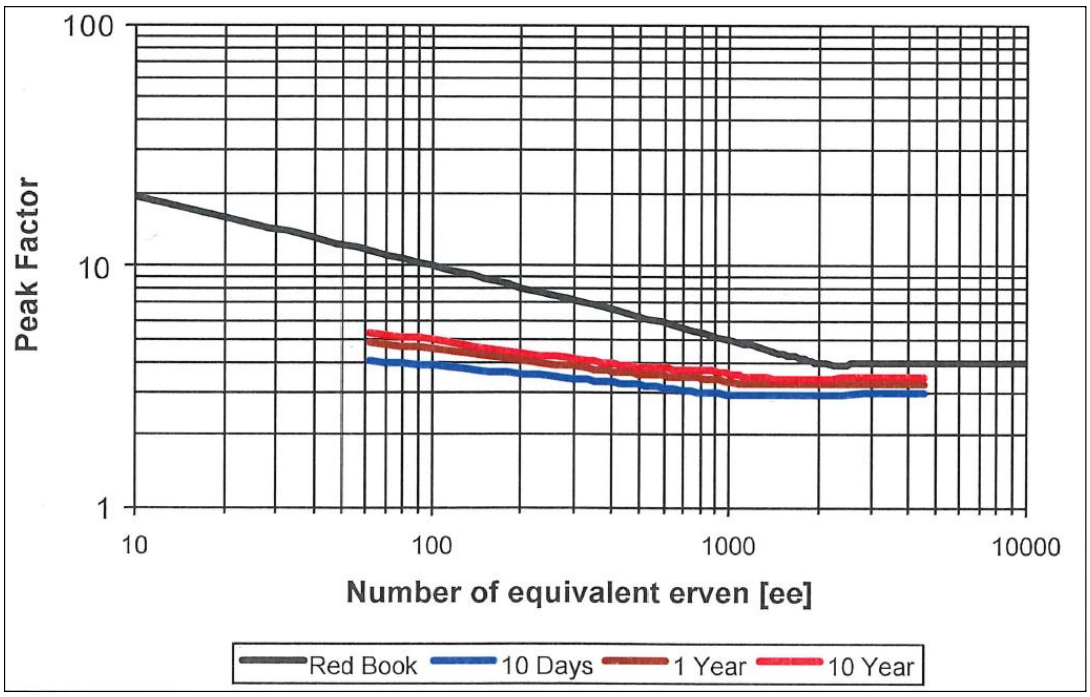


Figure 2.2: Probabilistic peak factors (Booyens and Haarhoff, 2002) in comparison with the CSIR (2000)

Booyens and Haarhoff's (2002) recommendation was that "the probabilistic nature of peak factors should be recognised and design guidelines should be adapted to allow designers to differentiate amongst different return periods, as has become standard practice in the field of hydrology".

2.3 Consequences of stringent design criterion

2.3.1 Over-spending on infrastructure

Booyens (2000) and Booyens and Haarhoff (2002) as well as Jacobs and Strijdom (2009) have found that the current design criterion for water systems is too stringent. Consequences of too stringent design criterion include over-spending on infrastructure. Strijdom (2008) found that this over-spending can add up to 32.5% based on a hypothetical relaxed criterion.

Municipal capital budgets are supported by the Municipal Infrastructure Grant (MIG) funding. The Department of Provincial and Local Government reported that in the first six months of

the 2007/2008 financial year approximately R2.9 billion of this grant had been spent of which the bulk (R1 112 million) had been spent on water infrastructure (The Presidency, 2008). Although the bulk of the municipal infrastructure budget has gradually been moved from water infrastructure to roads infrastructure since 2008, there has still been an enormous increase in the budget allocated for water infrastructure. The 2015 national budget review (National Treasury, 2015) indicates the following budget allocation for water infrastructure development:

Table 2.9: Water infrastructure development budget allocation

Water infrastructure expenditure	
Financial year	**Estimated expenditure (R billion)**
2015/16	12.4
2016/17	13.1
2017/18	14.7
Total	**40.2**

If approximately 32% can be saved on over-expenditure on water infrastructure due to too stringent design criterion (Strijdom, 2008), this will add up to a saving of R12.8 billion over the next three years.

GLS Consulting, the current master planning consultants for various South African municipalities, have indicated in their master plans (GLS, 2015) that the budget required to eradicate existing backlogs and to upgrade existing systems to be able to supply the future demand adds up to approximately R24 billion for the four largest metropoles alone (Tshwane R12.5bn, Cape Town R4.1bn, Johannesburg R3.7bn and Ekurhuleni R3.4bn).

3 STEADY STATE ANALYSES

This chapter presents the MPH results for several hydraulic models that were analysed in terms of the number of nodes experiencing MPH values below the design criterion of 24m under peak demand conditions for a steady state demand driven analyses.

3.1 Hydraulic models analysed

Following from the research by Jacobs and Strijdom (2009) where the hydraulic models for water distribution systems for 14 towns within five municipal areas were analysed (a total of 54 611 modelled nodes), the scope of this study was extended to provide an improved representative countrywide coverage.

Hydraulic models for all the metropolitan municipalities (MM) within South Africa excluding Nelson Mandela Bay MM (Port Elizabeth) and eThekwini MM (Durban) were analysed. To obtain a representative sample for all different types of consumers the hydraulic models for several district municipalities and smaller local municipalities spread over the rest of South Africa were also included. The smaller municipalities were selected specifically to include inland and coastal type consumers as well as to cover all the different climatic conditions in South Africa. In total, 71 towns located within 17 municipalities were included in this study. A total of 52 hydraulic models comprising a total of 539 388 modelled nodes were analysed.

The hydraulic models used in this study were all obtained from GLS Consulting (www.gls.co.za). However, for the purposes of this study the water demands for all the acquired hydraulic models were re-populated with the latest available AADDs (calculated from the latest available actual water meter readings) to obtain up-to-date hydraulic results. The method used for populating the hydraulic models is described in more detail under Section 3.2, Methodology.

3.2 Methodology

3.2.1 Calculating AADD

3.2.1.1 SWIFT-method

For the majority of the analysed municipal areas, the SWIFT software was used to calculate AADDs for each individual stand within the municipality. A stand is defined as a single plot or property. Jacobs and Fair (2012) provided a review of other research conducted with SWIFT and comprehensively describe the procedure. The method involves extracting the 12 latest monthly water meter readings from the treasury system to calculate the total annual

consumption, after which it was converted to an average annual daily demand. The treasury systems used in this study include SAP, Venus, Abacus, Samras and Edams.

3.2.1.2 Manual method

For a few of the smaller local municipalities the treasury data was either not available or, after scrutiny, deemed to be unreliable. For these areas a manual process was performed whereby theoretical AADDs based on land-use and stand size were allocated to each stand within the municipality, using the GIS-based functionality available within the WADISO software. Theoretical unit water demands (UWD) were derived over the last 20 years by GLS Consulting through analysis of actual metered data and collaboration with water providing authorities and were allocated to each stand as per Table 3.1.

Table 3.1: Theoretical unit water demands

Land use	Typical density (units/ha)		UWD		Unit
	Range	Assumed	kl/ha	kl/unit	
Rural	< 3	1.0	3.0	3.00	erf
Extra-large erven	3 to 5	4.0	10.0	2.40	erf
Large sized erven	5 to 8	6.5	12.0	2.00	erf
Medium sized erven	8 to 12	10.0	13.0	1.60	erf
Small sized erven	12 to 20	14.0	15.0	1.20	erf
Cluster 20 to 30	20 to 30	25.0	20.0	1.00	unit
Cluster 30 to 40	30 to 40	35.0	25.0	0.80	unit
Cluster 40 to 60	40 to 60	50.0	30.0	0.70	unit
Flats	60 to 100	80.0	50.0	0.60	unit
RDP	20 to 30	25.0	5.0	0.25	unit
Informal relocated	18 to 25	20.0	5.0	0.25	unit
Informal upgraded	18 to 25	20.0	15.0	0.75	unit
Informal upgraded RDP	18 to 25	20.0	5.0	0.25	unit
Low cost housing	15 to 20	20.0	13.0	0.60	erf
Business/Commercial	varies	40.0	25.0	0.80	100m² floor
Industrial	varies	40.0	20.0	0.40	100m² floor
Warehousing	varies	40.0	20.0	0.60	100m² floor
Mixed land use	varies	40.0	25.0	0.80	100m² floor
Parks & sports fields	n.a.	1.0	15.0	15.00	ha
Densification (Res)	varies	25.0	20.0	1.00	unit
Densification (BCI)	varies	60.0	40.0	0.80	100m² floor
Education	varies	40.0	15.0	20.00	unit
Institute	varies	40.0	15.0	20	100m² floor
Mine	n.a.	1.0	0.0	0	ha

For both the SWIFT-method and the manual method vacant stands were allocated a water demand of zero. For the SWIFT-method, “stand occupancy” information was also extracted from the treasury database to identify vacant stands. For the manual method the cadastral layout was superimposed onto the latest available aerial photography to identify which stands were vacant.

3.2.2 Populating existing system hydraulic models

The AADDs as calculated above for each stand were used to populate the analysed hydraulic models as follows:

Firstly, the water models were superimposed onto the cadastral stand layouts. The AADD of each stand was then allocated to the node nearest to the stand and all AADDs allocated to a certain node were added together, resulting in a total water demand for each node in the hydraulic model. This procedure was performed automatically using the SWIFT software where possible, but for the manual method a GIS-based spatial correlation was performed by manually using the GIS-functions available in the WADISO software.

Land use information for each stand was also extracted from the treasury system and allocated to each stand using SWIFT. For the manual method, land uses were allocated to stands using either aerial photography or correspondence with municipal operational staff. Land use information for each stand was then allocated to the node nearest to the stand resulting in a predominant land use per node.

The total AADD was used together with the predominant land use for each node and peak hour factors were allocated to each node according to the peak factors listed in Table 3.2. The AADD for each node was then multiplied with the peak hour factor allocated to the node, resulting in the modelled demand output for each node to be used in the hydraulic analysis.

Table 3.2: Theoretical peak factors (Vorster et al, 1995)

Predominant land use	AADD (kl/d)	Peak week factor (PWF)	Peak day factor (PDF)	Peak hour factor (PHF)
Residential (Low cost housing)	<1000	1.50	1.90	3.60
	1000 - 5000	1.40	1.80	3.40
	5000 - 10000	1.35	1.70	3.30
	10000 - 15000	1.30	1.50	3.20
	15000 - 20000	1.25	1.40	3.10
	>20000	1.25	1.40	3.00
Residential (Conventional)	<1000	1.80	2.20	4.60
	1000 - 5000	1.65	2.00	4.00
	5000 - 10000	1.50	1.80	3.60
	10000 - 15000	1.40	1.60	3.50
	15000 - 20000	1.35	1.50	3.30
	>20000	1.30	1.50	3.00
Business Commercial Industrial	<5000	1.45	1.70	3.30
	5000 -10000	1.30	1.60	3.15
	>10000	1.25	1.50	3.00

3.2.3 Performing hydraulic analyses

As mentioned earlier, the commercial software package WADISO 5.11 (GLS, 2015) was used for all hydraulic analyses performed in this study. A steady state demand-driven analysis under peak demand conditions for the existing operational scenario was performed for each hydraulic model. A steady state analysis involves modelling a fixed demand (in this case the theoretical peak hour demand) at each output node in a hydraulic model. Therefore, fluctuations in water demand patterns are ignored.

3.2.4 Statistical analyses

After performing the hydraulic analyses, the nodal result tables were exported from WADISO to Microsoft Excel. The statistical analyses were performed using Microsoft Excel. For each model run, the set of nodal results were statistically analysed to include the sample size (number of nodes), average MPH (average of the minimum head of each node), standard deviation of MPH and the percentage of nodes with MPH values within certain predefined H-categories. The H-categories were set up to indicate relative frequency for H (i.e. percentage

of nodes falling within the upper and the lower H limit of a certain H-category) as well as to indicate cumulative frequency for H (i.e. percentage of nodes with pressures below a certain value of H).

3.2.5 Results

The nodal result tables and the results of the statistical analyses for each model run are included in Appendix A. A summary of the statistics for the steady state analyses is represented in Table 3.3. Hydraulic models where more than 40% of the nodes reported residual pressures below 24m have been highlighted in red in the last column of the table.

Table 3.3: Summary statistics of all models analysed

City/Town/Depot	Municipality	Province	Number of nodes	Average MPH (m)	Standard deviation of H	% Nodes with H < 24m
Plettenberg Bay	Bitou	Western Cape	3012	41.17	27.27	**33%**
East London Coastal	Buffalo City	Eastern Cape	14726	37.69	18.25	**25%**
East London Midlands	Buffalo City	Eastern Cape	8602	42.26	17.86	**16%**
East London Inlands	Buffalo City	Eastern Cape	6651	36.90	22.52	**29%**
Cape Town	City of Cape Town	Western Cape	126072	50.96	18.73	**11%**
Ennerdale	City of Johannesburg	Gauteng	10131	40.59	26.04	**27%**
Hamberg	City of Johannesburg	Gauteng	13503	50.38	21.68	**8%**
Klipspruit Avolon	City of Johannesburg	Gauteng	31420	37.37	19.11	**22%**
Langlaagte	City of Johannesburg	Gauteng	13340	58.54	24.07	**4%**
Randburg	City of Johannesburg	Gauteng	12412	63.18	25.15	**3%**
Midrand	City of Johannesburg	Gauteng	9479	47.57	21.76	**11%**
Southdale	City of Johannesburg	Gauteng	12636	62.56	25.91	**6%**
Zandfontein North	City of Johannesburg	Gauteng	12538	54.66	26.77	**13%**
Zandfontein South	City of Johannesburg	Gauteng	17420	64.75	22.33	**4%**
Atteridgeville	City of Tshwane	Gauteng	2879	47.96	18.99	**8%**
Bronkhorstspruit & Ekangala	City of Tshwane	Gauteng	4765	49.81	39.35	**21%**
Pretoria	City of Tshwane	Gauteng	44605	57.47	26.49	**7%**
Cullinan, Rayton & Refilwe	City of Tshwane	Gauteng	2138	31.27	15.07	**30%**
Odi	City of Tshwane	Gauteng	8471	44.81	22.10	**17%**

City/Town/Depot	Municipality	Province	Number of nodes	Average MPH (m)	Standard deviation of H	% Nodes with H < 24m
Madibeng	City of Tshwane	Gauteng	779	48.48	27.63	**18%**
Mamelodi	City of Tshwane	Gauteng	5668	48.40	19.06	**7%**
Akasia	City of Tshwane	Gauteng	4138	58.04	19.73	**2%**
Soshanguve	City of Tshwane	Gauteng	12723	41.60	19.94	**15%**
Centurion	City of Tshwane	Gauteng	15443	47.30	28.24	**19%**
Temba	City of Tshwane	Gauteng	10042	30.29	13.94	**35%**
Wallmannsthal	City of Tshwane	Gauteng	2338	53.12	30.08	**17%**
Alberton	Ekurhuleni	Gauteng	7813	55.67	20.50	**4%**
Benoni	Ekurhuleni	Gauteng	11411	31.78	15.27	**28%**
Boksburg	Ekurhuleni	Gauteng	14255	38.47	13.43	**13%**
Brakpan	Ekurhuleni	Gauteng	6744	23.94	14.72	**46%**
Edenvale	Ekurhuleni	Gauteng	3041	39.12	15.80	**18%**
Germiston	Ekurhuleni	Gauteng	15388	30.38	20.57	**42%**
Kempton Park	Ekurhuleni	Gauteng	12409	41.09	19.69	**17%**
Nigel	Ekurhuleni	Gauteng	3287	23.12	17.26	**54%**
Springs	Ekurhuleni	Gauteng	6182	25.44	56.67	**44%**
Ladysmith	Emnambithi/Ladysmith	KwaZulu-Natal	7617	38.31	21.75	**31%**
De Aar	Emthanjeni	Northern Cape	936	27.52	7.18	**31%**
George	George	Western Cape	15981	41.55	20.25	**21%**
Stilbaai	Hessequa	Western Cape	1472	24.67	20.73	**55%**
Koffiefontein	Letsemeng	Free State	669	11.11	12.71	**81%**
Nelspruit	Mbombela	Mpumalanga	5892	44.39	23.33	**24%**
Pietermaritzburg (Vulindlela)	Msunduzi	KwaZulu-Natal	4867	60.53	49.84	**16%**
Hermanus	Overstrand	Western Cape	5715	40.82	14.13	**8%**
Hopefield	Saldanha Bay	Western Cape	339	37.37	18.62	**19%**
Jacobsbaai	Saldanha Bay	Western Cape	95	60.62	5.60	**0%**
Langebaan	Saldanha Bay	Western Cape	1411	40.04	18.50	**21%**
Paternoster	Saldanha Bay	Western Cape	247	29.61	6.36	**27%**
Saldanha	Saldanha Bay	Western Cape	1017	56.68	22.40	**8%**

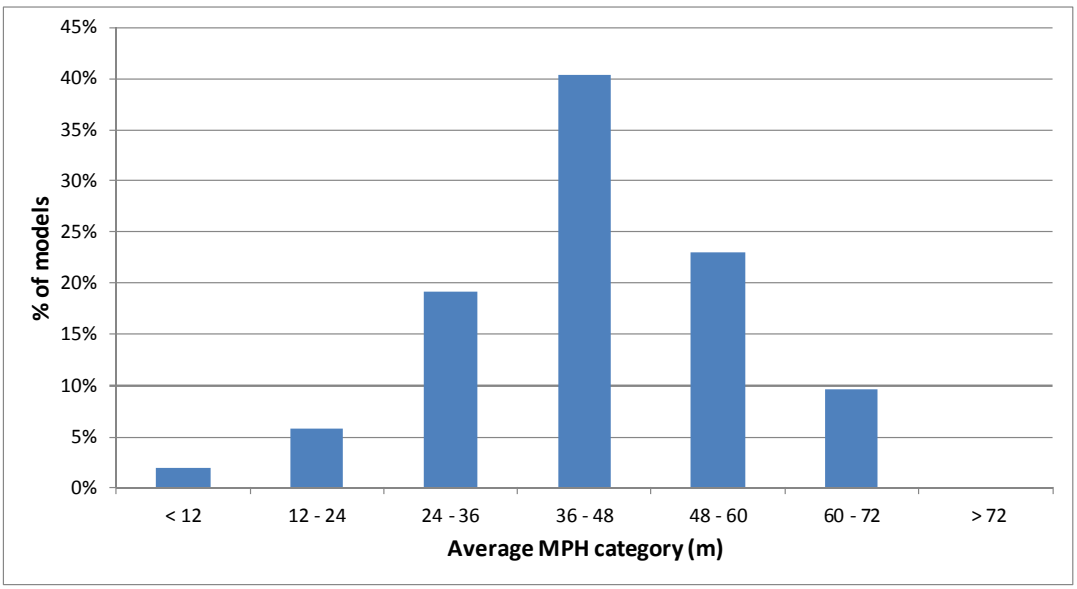
City/Town/Depot	Municipality	Province	Number of nodes	Average MPH (m)	Standard deviation of H	% Nodes with H < 24m
St Helena Bay	Saldanha Bay	Western Cape	1418	30.05	26.34	**43%**
Vredenburg	Saldanha Bay	Western Cape	1271	47.12	23.28	**13%**
Malmesbury	Swartland	Western Cape	2796	29.75	21.86	**40%**
Postmasburg	Tsantsabane	Northern Cape	1184	19.25	11.67	**60%**
		Total	**539388**		**Average**	**16%**

3.2.6 General

From Table 3.3, it is clear that the selection of models analysed allowed for a well-balanced countrywide spread of towns covering different town types (coastal, inland, urban metropoles and rural towns) as well as different climate types of South Africa. A map of South Arica indicating the locations of the analysed models are included in Appendix Furthermore, the selection allows for sufficient variation in the number of nodes per model analysed. The largest model analysed was the City of Cape Town where the whole city (including all surrounding suburbs) was merged into a single model consisting of approximately 126 000 nodes. The smallest model analysed was the coastal holiday town of Jacobsbaai consisting of 95 nodes with only one pressure zone.

3.2.7 Average MPH

It is furthermore surprising to note the great variation in average MPH for the models analysed. Average MPH for the models ranges from as low as 11.11m (Koffiefontein) to as high as 64.75m (Zandfontein South depot in Johannesburg). Four models have an average MPH of below 24m which, according to the design criterion, is the minimum value for the most critical node in the system. Average MPH, however, is not considered to be a good indication of whether design criterion is being adhered to for a specific model. Although Plettenberg Bay, for example, has a relatively high average MPH of 41m, 33% of the nodes in the system experienced MPH values below the minimum criterion of 24m. A frequency histogram of the average MPH for the models analysed is shown in Figure 3.1.



Ave MPH category	< 12	12 - 24	24 - 36	36 - 48	48 - 60	60 - 72	> 72
Number of models	1	3	10	21	12	5	0
% of models	2%	6%	19%	40%	23%	10%	0%

Figure 3.1: Frequency histogram for average MPH

From Figure 3.1, it can be noted that most of the models analysed have an average MPH of between 36m – 48m. This is considered a significant finding because it illustrates that while the majority of water-providing authorities are attempting to supply an MPH of 24m at the most critical node in their system they are actually supplying the rest of the nodes in the system with MPH values way in excess of 24m.

3.2.8 Nodes with MPH < 24m

At this stage of the study, the results with the most significance are the percentage of nodes with MPH < 24m as indicated in the far right column of Table 3.3. Once again a great variation is noted from models with zero nodes with MPH < 24 (therefore completely complying with the current design criterion) to models with up to 81% of nodes not complying. The “best” system is Jacobsbaai with all nodes complying, which is not really significant as it is also the smallest model with only 95 nodes and a single pressure zone. In total there are 18 models with less than 15% of nodes not complying. The most significant is the City of Cape Town with only 11% of its approximately 126 000 nodes not complying to the MPH criteria.

On the other end of the scale, the “worst” system is Koffiefontein with 81% of nodes not complying, although the Koffiefontein model only comprises of 669 nodes. In total nine models have more than 40% of nodes not complying. The most significant is the Germiston model with 42% of its 15 388 nodes not complying. In total 16% of all modelled nodes are not complying with the current design criterion.

3.2.9 Relative and cumulative frequency for MPH

A better interpretation of the results is possible when the relative and cumulative frequency of the percentage of nodes with MPH within the predefined H-categories mentioned earlier is calculated. A summary of these results is included in Table 3.4.

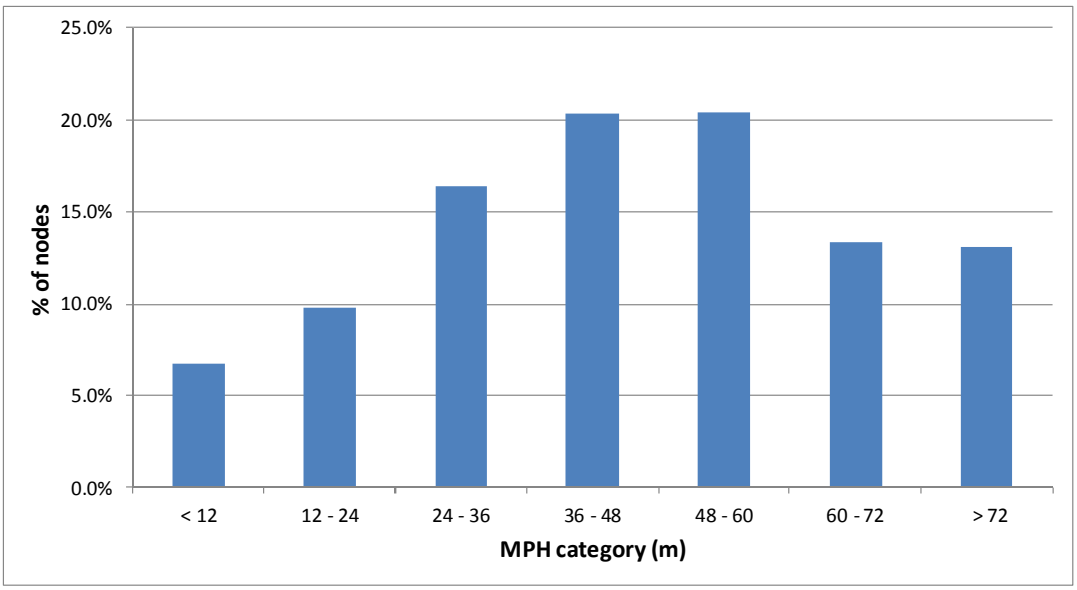
Table 3.4: Relative and cumulative frequency for MPH in all models

	Relative frequency (% of nodes)					Cumulative frequency (% of nodes)				
City/Town/Depot	**H < 12**	**12 < H < 24**	**24 < H < 36**	**36 < H < 48**	**48 < H < 60**	**H < 12**	**H < 24**	**H < 36**	**H < 48**	**H < 60**
Plettenberg Bay	15%	18%	13%	14%	14%	15%	**33%**	46%	61%	75%
East London Coastal	8%	17%	24%	23%	16%	8%	**25%**	49%	72%	88%
East London Midlands	3%	12%	23%	25%	19%	3%	**16%**	39%	64%	83%
East London Inlands	18%	11%	16%	22%	18%	18%	**29%**	45%	67%	85%
Cape Town	3%	9%	9%	20%	30%	3%	**11%**	20%	40%	69%
Ennerdale	18%	9%	18%	16%	10%	18%	**27%**	46%	62%	71%
Hamberg	4%	4%	16%	24%	21%	4%	**8%**	24%	48%	69%
Klipspruit Avolon	11%	11%	23%	26%	17%	11%	**22%**	45%	71%	89%
Langlaagte	2%	2%	10%	23%	21%	2%	**4%**	14%	37%	58%
Randburg	2%	2%	7%	16%	22%	2%	**3%**	11%	27%	49%
Midrand	4%	7%	18%	25%	21%	4%	**11%**	29%	55%	76%
Southdale	2%	3%	10%	15%	16%	2%	**6%**	16%	30%	46%
Zandfontein North	6%	6%	11%	15%	20%	6%	**13%**	23%	39%	59%
Zandfontein South	2%	2%	5%	9%	24%	2%	**4%**	9%	18%	42%
Atteridgeville	3%	6%	19%	26%	19%	3%	**8%**	28%	54%	73%
Bronkhorstspruit & Ekangala	7%	14%	18%	17%	17%	7%	**21%**	39%	55%	72%

Pretoria	3%	4%	11%	20%	22%	3%	**7%**	17%	37%	59%
Cullinan, Rayton & Refilwe	10%	19%	29%	33%	7%	10%	**30%**	59%	92%	99%
Odi	4%	14%	23%	18%	19%	4%	**17%**	40%	58%	77%
Madibeng	3%	15%	19%	22%	16%	3%	**18%**	37%	59%	75%
Mamelodi	2%	5%	21%	23%	25%	2%	**7%**	28%	51%	76%
Akasia	1%	1%	6%	26%	25%	1%	**2%**	8%	34%	60%
Soshanguve	2%	13%	29%	26%	16%	2%	**15%**	44%	70%	86%
Centurion	10%	8%	15%	19%	18%	10%	**19%**	34%	53%	71%
Temba	8%	27%	33%	24%	5%	8%	**35%**	68%	91%	97%
Wallmannsthal	6%	10%	13%	19%	17%	6%	**17%**	30%	49%	66%
Alberton	2%	1%	14%	13%	26%	2%	**4%**	18%	31%	57%
Benoni	11%	17%	32%	25%	11%	11%	**28%**	61%	86%	97%
Boksburg	2%	11%	31%	33%	17%	2%	**13%**	44%	77%	94%
Brakpan	22%	24%	34%	16%	3%	22%	**46%**	80%	96%	99%
Edenvale	2%	16%	25%	30%	17%	2%	**18%**	43%	73%	90%
Germiston	20%	21%	21%	15%	13%	20%	**42%**	63%	78%	92%
Kempton Park	9%	8%	22%	25%	20%	9%	**17%**	39%	64%	84%
Nigel	31%	23%	24%	13%	6%	31%	**54%**	78%	91%	97%
Springs	24%	20%	37%	13%	4%	24%	**44%**	81%	94%	98%
Ladysmith	10%	21%	21%	15%	14%	10%	**31%**	52%	67%	82%

De Aar	2%	30%	61%	7%	0%	2%	**31%**	92%	100%	100%
George	9%	12%	18%	21%	21%	9%	**21%**	40%	61%	82%
Stilbaai	36%	19%	17%	11%	10%	36%	**55%**	72%	83%	92%
Koffiefontein	54%	26%	17%	2%	0%	54%	**81%**	98%	100%	100%
Nelspruit	12%	13%	11%	18%	19%	12%	**24%**	36%	53%	72%
Pietermaritzburg (Vulindlela)	11%	5%	9%	19%	14%	11%	**16%**	25%	44%	58%
Hermanus	2%	6%	35%	29%	18%	2%	**8%**	43%	72%	90%
Hopefield	14%	5%	27%	23%	12%	14%	**19%**	47%	70%	82%
Jacobsbaai	0%	0%	0%	5%	21%	0%	**0%**	0%	5%	26%
Langebaan	12%	9%	12%	31%	26%	12%	**21%**	33%	63%	89%
Paternoster	1%	26%	72%	1%	0%	1%	**27%**	99%	100%	100%
Saldanha	5%	4%	8%	18%	17%	5%	**8%**	17%	35%	52%
St Helena Bay	32%	11%	19%	17%	5%	32%	**43%**	62%	80%	84%
Vredenburg	7%	5%	13%	29%	27%	7%	**13%**	25%	54%	81%
Malmesbury	28%	12%	18%	17%	17%	28%	**40%**	59%	76%	93%
Postmasburg	33%	27%	32%	6%	1%	33%	**60%**	93%	99%	100%
	6.7%	**9.8%**	**16%**	**20%**	**20%**	**7%**	**16%**	**33%**	**53%**	**74%**

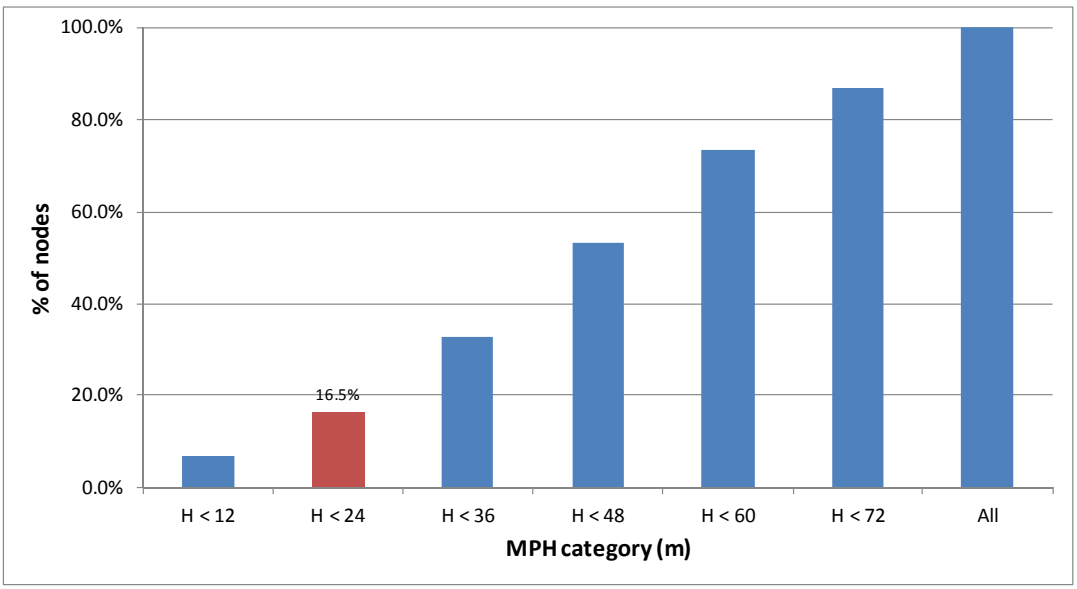
A better understanding of the results in Table 3.4 is obtained when the results are evaluated graphically in terms of relative and cumulative frequency histograms. The relative and cumulative frequency histograms for all the analysed models are included separately in Appendix B. The total relative frequency histogram for all analysed models is indicated in Figure 3.2.



MPH category	< 12	12 - 24	24 - 36	36 - 48	48 - 60	60 - 72	> 72
Number of nodes	36120	52808	88253	109675	110057	71997	70478
% of nodes	6.7%	9.8%	16.4%	20.3%	20.4%	13.3%	13.1%

Figure 3.2: Relative frequency histogram for MPH in all models

The relative frequency histogram corresponds approximately to a normal distribution with most of the nodes in the models analysed experiencing MPH values in the 36 < H < 48 category and the 48 < H < 60 category. Again this is an indication of relatively high minimum pressures experienced in the rest of the system, since the system would have been designed to comply with MPH > 24m at the most critical node. What can be noted further is the relatively high percentage of nodes (13%) with MPH > 72m, which is a clear indication of ineffective pressure management. To simplify management of their pressure zones, some water-providing authorities are of the opinion that fewer pressure zones are easier to operate. Fewer pressure zones imply larger zones which, in turn, imply larger pressure differences between MPH experienced at the most critical node and MPH experienced at the least critical node. The total cumulative frequency histogram for all analysed models is indicated in Figure 3.3.



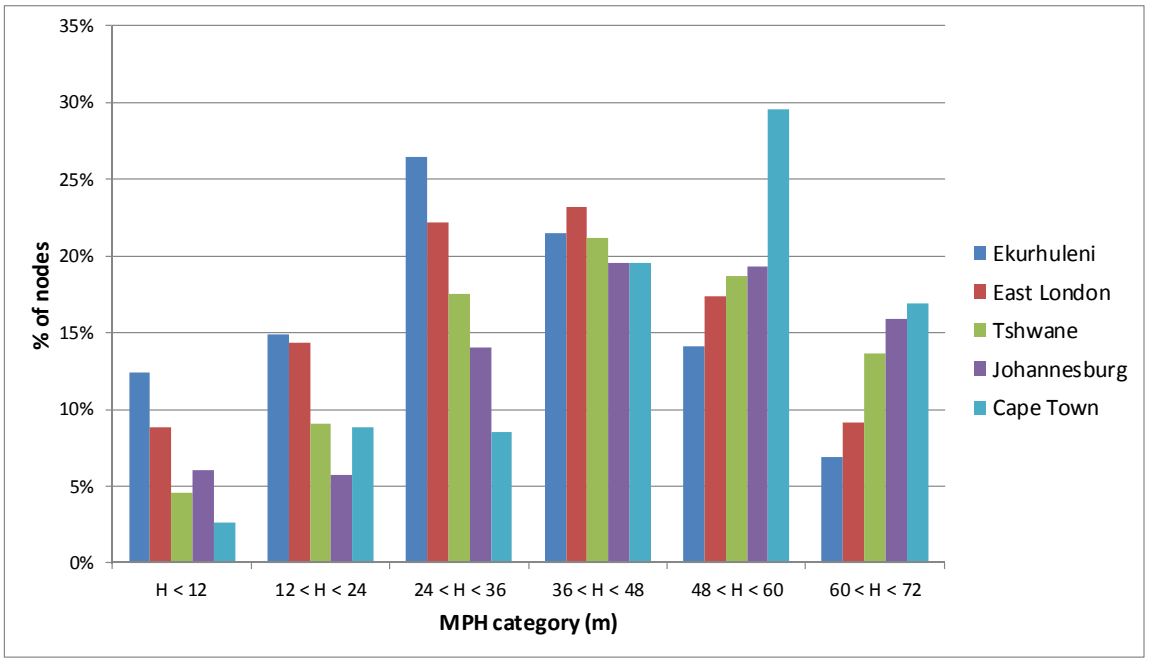
MPH category	H < 12	H < 24	H < 36	H < 48	H < 60	H < 72	All
Number of nodes	36120	88928	177181	286856	396913	468910	539388
% of nodes	6.7%	16.5%	32.8%	53.2%	73.6%	86.9%	100.0%

Figure 3.3: Cumulative frequency histogram for MPH in all models

The histogram in Figure 3.3 indicates that 16.5% of all the nodes analysed in this study experience minimum pressures below the current design criterion. Furthermore, 6.7% of all nodes experience pressures below 12m.

3.2.10 MPH comparison between metropoles

As mentioned earlier, all the South African metropolitan municipalities except for Nelson Mandela (Port Elizabeth) and eThekwini (Durban) were analysed as a part of this study. A comparison of the relative frequency for MPH between the metropoles is indicated in Figure 3.4.



		% Relative frequency of MPH						
	No of nodes	H < 12	12 < H < 24	24 < H < 36	36 < H < 48	48 < H < 60	60 < H < 72	H > 72
Ekurhuleni	80530	12%	15%	26%	22%	14%	7%	4%
East London	29979	9%	14%	22%	23%	17%	9%	5%
Tshwane	113989	5%	9%	18%	21%	19%	14%	15%
Johannesburg	132879	6%	6%	14%	20%	19%	16%	19%
Cape Town	126072	3%	9%	9%	20%	30%	17%	14%

Figure 3.4: Metropoles comparison – Relative frequency of MPH

From Figure 3.4 it can be noted that Ekurhuleni has the highest percentage of nodes with MPH in the lowest pressure category and at the same time also has the lowest percentage of nodes with MPH in the highest pressure category. In contrast Cape Town has the lowest percentage of nodes with MPH in the lowest pressure category whilst having the highest percentage of nodes with MPH in the highest pressure category.

At first glance it appears as if Ekurhuleni is scoring the “worst” in terms of providing pressure to their consumers. However, if defining the 24m < H < 36m category as the most economical category (i.e. providing sufficient pressure in terms of the current design criterion whilst not wasting water, energy and cost in providing pressures that are too high), it can be argued that Ekurhuleni is outperforming all other metropoles.

3.2.11 Discussion

Although 16.5% of all nodes analysed in this study are experiencing pressures below the minimum requirements under the current design criterion, very few low pressure complaints are being received by the water-providing authorities for these affected areas, or at least this is true for the Ekurhuleni Metropolitan Municipality – the metro with the highest percentage of nodes with MPH < 24m. As mentioned earlier, these low pressure areas were investigated for all of the Ekurhuleni models during comprehensive workshops convened at each of the Ekurhuleni service delivery depots. According to the operational staff, no low pressure complaints originated from areas within the 12m < H < 24m category, while some complaints originated from areas within the H < 12m category.

Based on the assumption that the Ekurhuleni customer complaint system is effective and is used by customers to report low pressures, it seems like consumers are accepting these lower pressures without complaints, within certain lower limits of course (certain household appliances will fail to operate sufficiently when the pressure drops to below 10m which will most certainly result in customer complaints). If decreasing the current criterion MPH value of 24m by 12m to MPH > 12m, this will still allow a 2m “safety margin” before the lower limit of 10m set by appliance failures is reached. Furthermore, such a decrease will move 10% of all nodes from a “non-complying” MPH-category into a “complying” MPH-category which implies that where water-providing authorities would have spent capital on upgrading for 16% of their system, they would under the relaxed criterion only have to spend capital on 7% of their system. A “safety margin” of 2m might, in some consumers’ opinion, be too risky given the great variation in variables influencing residual pressures. A lower limit of MPH > 15m is therefore considered more realistic in terms of a possible relaxed criterion. Chapter 5 will present an in-depth study that was performed wherein the exact capital costs required to upgrade a system to comply with the current criterion of MPH > 24m was compared to the capital costs required for a system with a relaxed criterion of MPH > 15m.

4 TIME SIMULATION ANALYSES

This chapter presents the MPH results for a single selected pressure zone within a time simulated hydraulic model (as opposed to steady state analysis). The results were analysed in terms of two aspects: (i) the number of nodes experiencing MPH values of below the design criterion of 24m under peak demand conditions, as well as (ii) the amount of time that each of these nodes were subjected to sub-standard pressures.

4.1 Hydraulic model analysed

Due to the relative complexity of setting up and running a time simulation model, it was decided to choose one of the less complex models (i.e. a model with minimal links controlled by time, pressure and water levels - in other words a model with few pumps switching on or off based on reservoir or tower water levels or control valves opening or closing based on system pressure). The choice of model to be analysed was also influenced by the number of nodes with MPH < 24m obtained from the steady state model analyses. The hourly nodal pressures for each node with MPH < 24m had to be exported over a weekly demand pattern of 168 hours before these results could be statistically analysed.

The residential suburb of Sharon Park in the town of Nigel (Ekurhuleni) comprise a relatively small hydraulic model with low complexity and minimal yet sufficient nodes with MPH < 24m. The complete suburb is one discreet pressure zone being supplied from the Sharon Park water tower and was the obvious choice of model to be analysed during this portion of the study. The Sharon Park tower zone consists of 124 nodes of which 15 nodes (12%) experience MPH < 24m according to the steady state analysis. These 15 nodes were analysed during this portion of the study.

4.2 Methodology

4.2.1 Population of hydraulic model

For the calculation of AADD and population of the hydraulic model, the SWIFT-method as described in Chapter 3 was used by utilising data from the Ekurhuleni Metropolitan Municipality's Venus treasury system. The time simulation implied that peak hour factors were not multiplied by average demand to obtain peak demand output values for the nodes as was done for the steady state analyses. Instead, for the time simulation analysis, hourly unit water demand patterns for a time period of a week were multiplied by the average demand per modelled node, resulting in a weekly demand pattern with one-hour time steps for each modelled node. The water demand pattern chosen was unique for each land. These

unique water demand patterns were extracted from logging results obtained from the City of Tshwane's telemetry system as part of a parallel study by GLS Consulting. These water demand patterns are currently used by GLS Consulting for all time simulations.

The peak hour factors effectively contained within these weekly water demand patterns correspond with the peak hour factors indicated in Table 3.2 that were used for the steady state analysis. The weekly unit water demand patterns per land use are indicated in Figure 4.1. The unit water demand pattern tables are included in Appendix C.

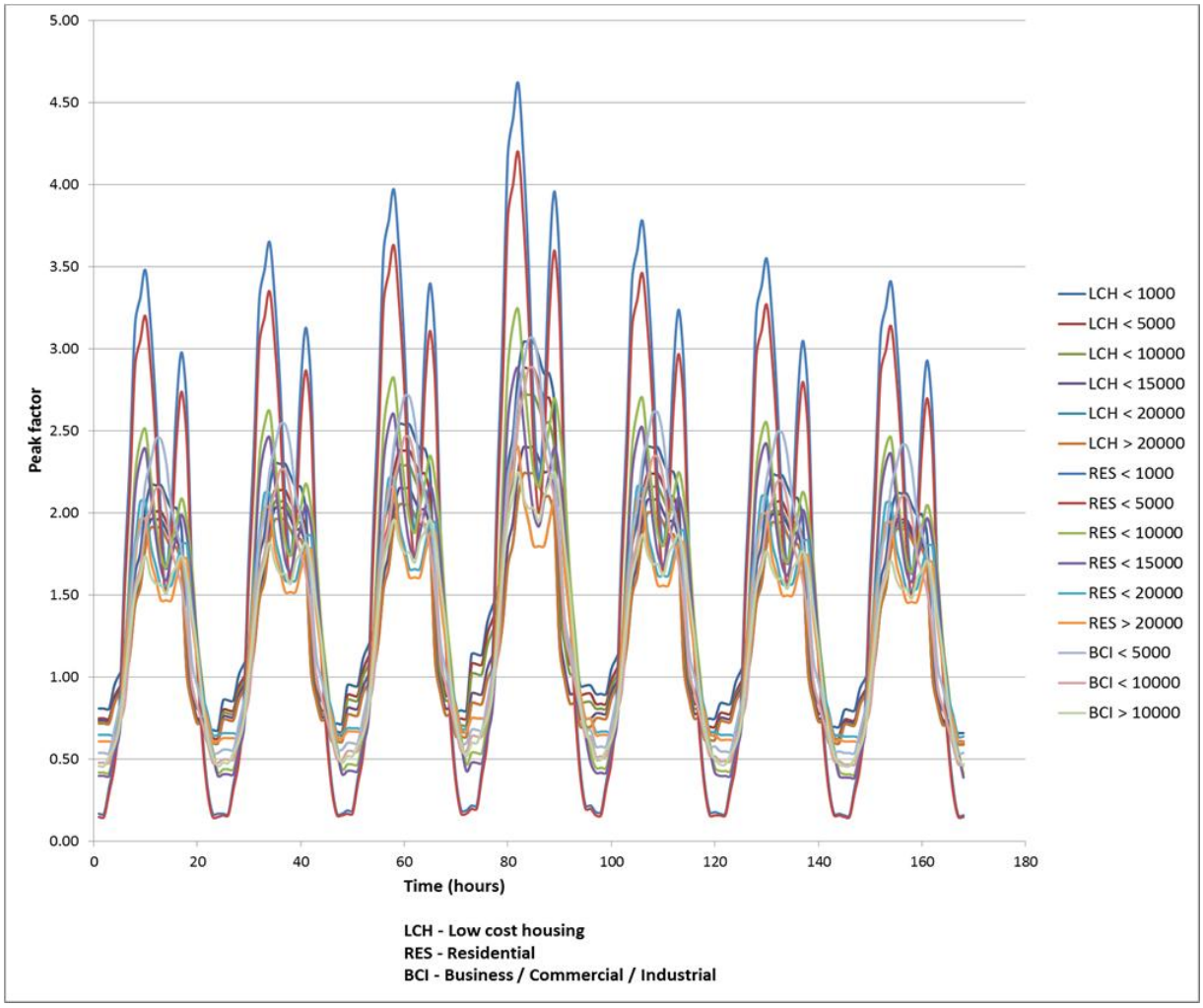


Figure 4.1: Weekly unit water demand patterns for time simulation

A predominant land use type was allocated to each node in the model as per the process described in Chapter 3. The average water demand for each node was then multiplied by the unit water demand pattern governed by the predominant land use of the node to obtain the weekly demand pattern with one-hour time steps for each modelled node.

4.2.2 Statistical analyses

After performing the hydraulic time simulation analysis, the nodal result tables were again exported from WADISO to Microsoft Excel where the statistical analyses were performed. Only the results for nodes which, at some point during the weekly time simulation period, experienced pressures of below 24m were exported. For each node the weekly nodal results were statistically analysed to include minimum and maximum values of H, average H, standard deviation of H and the total amount of time that H was below 24m for the node.

4.2.3 Results

The nodal result tables and the results of the statistical analyses are included for each analysed node (i.e. nodes with MPH < H) in Appendix D. The weekly time graph for H for all analysed nodes with an MPH < 24m at any stage are shown in Figure 4.2.

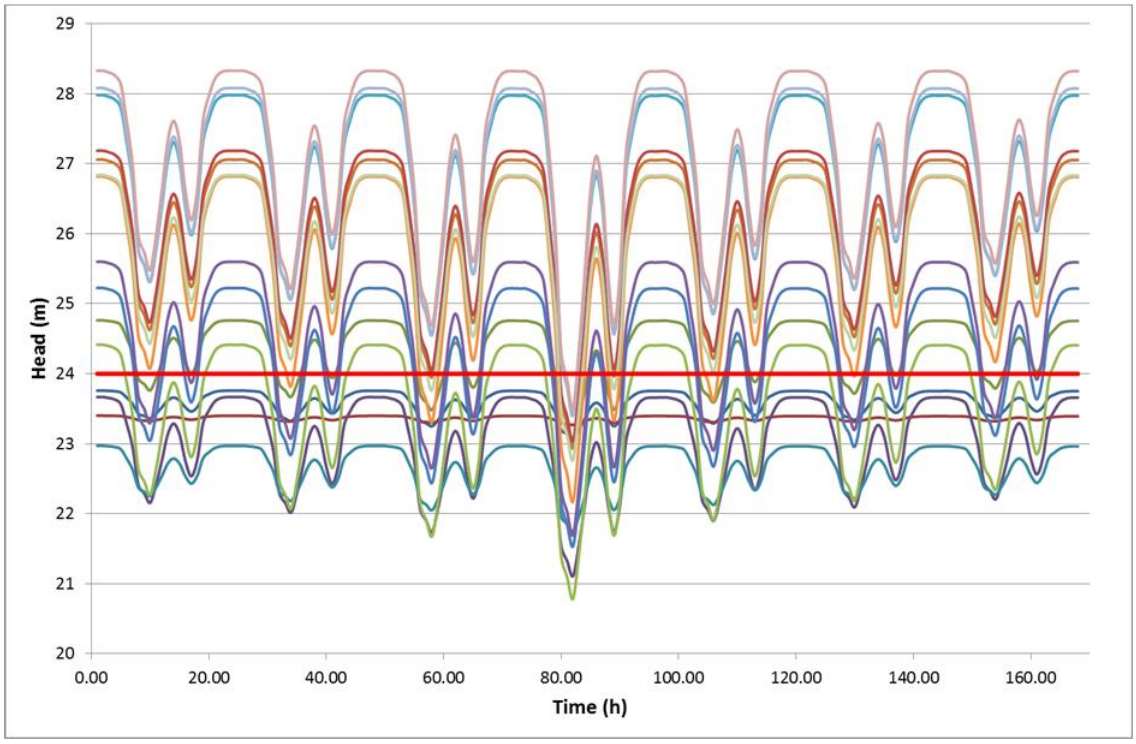


Figure 4.2: Time simulation results for H – Sharon Park tower zone

In Figure 4.2, time 0h represents 12:00am on day one of a seven-day period which is theoretically the time point with the lowest demand and the highest pressure. From the figure it can be noted that 15 nodes in the zone experienced a MPH of below 24m at some point during a typical weekly demand pattern of which four nodes had a maximum H of below 24m. This implies that these four nodes spent the entire week with H < 24m. Incidentally

these four nodes are the nodes with the highest elevation and are also the four nodes nearest to the water tower from where the zone is supplied. The variation in H-values for these nodes is also relatively small over the weekly time period compared to nodes that are further away from the supply source and which are therefore more susceptible to friction losses through the network.

For the rest of the nodes which start the week off with H > 24 and at some point in the week decrease to below 24m, it seems that a significant amount of time is being spent where H > 24m as illustrated in Figure 4.2. A better understanding of the results shown in Figure 4.2 is obtained when the amount of time spent above and below the design standard of 24m (red line on Figure 4.2) is quantified as shown in Table 4.1.

Table 4.1: Summary statistics of time simulated nodes

Sharon Park tower zone								
Node no	General statistics				Hours (h)		% of time	
	Ave H (m)	St dev H (m)	Min H (m)	Max H (m)	H < 24m	H > 24m	H < 24m	H > 24m
1	23.59	0.16	23.08	23.76	168.0	0.0	100.0%	0.0%
2	23.37	0.03	23.27	23.40	168.0	0.0	100.0%	0.0%
3	23.05	0.62	21.11	23.67	168.0	0.0	100.0%	0.0%
4	22.67	0.29	21.75	22.97	168.0	0.0	100.0%	0.0%
5	23.53	0.87	20.79	24.41	99.0	69.0	58.9%	41.1%
6	24.33	0.89	21.53	25.22	60.0	108.0	35.7%	64.3%
7	24.65	0.94	21.69	25.60	45.0	123.0	26.8%	73.2%
8	24.35	0.41	23.07	24.76	40.0	128.0	23.8%	76.2%
9	25.69	1.12	22.17	26.82	13.0	155.0	7.7%	92.3%
10	25.85	0.98	22.77	26.84	6.0	162.0	3.6%	96.4%
11	26.06	0.99	22.94	27.06	6.0	162.0	3.6%	96.4%
12	26.18	1.00	23.04	27.19	4.0	164.0	2.4%	97.6%
13	26.89	1.09	23.46	27.98	2.0	166.0	1.2%	98.8%
14	26.95	1.13	23.40	28.08	2.0	166.0	1.2%	98.8%
15	27.16	1.16	23.51	28.33	2.0	166.0	1.2%	98.8%
					951.0	**1569.0**	**37.7%**	**62.3%**

Table 4.1 includes all nodes within the analysed model with an MPH < 24m (15 of the 124 modelled nodes), i.e. the nodes that are not conforming to the current design criterion. For these non-conforming nodes it is clear from Table 4.1 that the total amount of time spent with H > 24m significantly exceeds the total amount of time spent with H < 24m. What is more remarkable is that more than 50% of non-conforming nodes spend less than 25% of the time in a non-conforming state. Forty percent of non-conforming nodes spend less than 4% of the time not conforming, while 20% of non-conforming nodes spend less than 2% of the time not conforming.

The percentage of non-conforming nodes versus the percentage of time spent in a non-conforming state is summarised in Table 4.2.

Table 4.2: Non-conforming node summary

Percentage of non-conforming nodes	Percentage of time spent with H < 24m
67%	< 36%
60%	< 27%
53%	< 24%
47%	< 8%
40%	< 4%
20%	< 2%

4.2.4 Discussion 1

It has been found that 88% of nodes in the zone analysed conform to the design criterion for 100% of the time. The remaining 12% “non-conforming” nodes actually conform to the criterion for most of the time over a weekly period. Yet this is a zone that, if analysed in steady state as a whole, does not conform to the current design criterion. Considering the fact that theoretical peak factors used for analysis and design (CSIR, 2003) are conservative (Booyens & Haarhoff, 2002), the above finding is exaggerated. If the four nodes close to the reservoir with static heads of below 24m (i.e. nodes for which the maximum head will never increase to above 24m for the entire week) are ignored for the moment, the rest of the zone would conform to the design criterion all of the time, yet it would be considered as a non-conforming zone.

Whether nodes in the zone conform to the criterion or not, the fact remains that for the selected few nodes that theoretically do not conform, this non-conforming state occurs for a very small percentage of the time. Furthermore, if a non-conforming state were to occur in reality (which is unlikely), the consequences of non-conformance are adjudged to be insignificant unless the actual pressures decrease to below 10m (Jacobs & Strijdom, 2009).

Ideally, an optimised model where MPH at the most critical node is exactly equal to 24m is sought to test the compliance of nodes if the design criterion value of MPH > 24m were decreased to, for example, 15m. None of the models in this study met these conditions hence it was decided to set up a theoretical model to test the above hypothesis.

4.2.5 Setting up typical theoretical model

In order to test the compliance of nodes when the design criterion is relaxed from MPH > 24m to MPH > 15m, a typical theoretical hydraulic model was compiled using the WADISO software. The typical model comprises of a zone with an area of 100ha which represents a typical medium density residential suburb with 1000 stands. The network consists of a total of 25 nodes spaced equally in a 250m x 250m grid. Ground elevations were allocated to the nodes to represent a topological area sloping evenly from north to south. The total length of network pipes in the zone is 10km. The zone is supplied via a single dedicated bulk supply pipe from a single reservoir located 1km away from the nearest node in the zone, as indicated on the Figure 4.3.

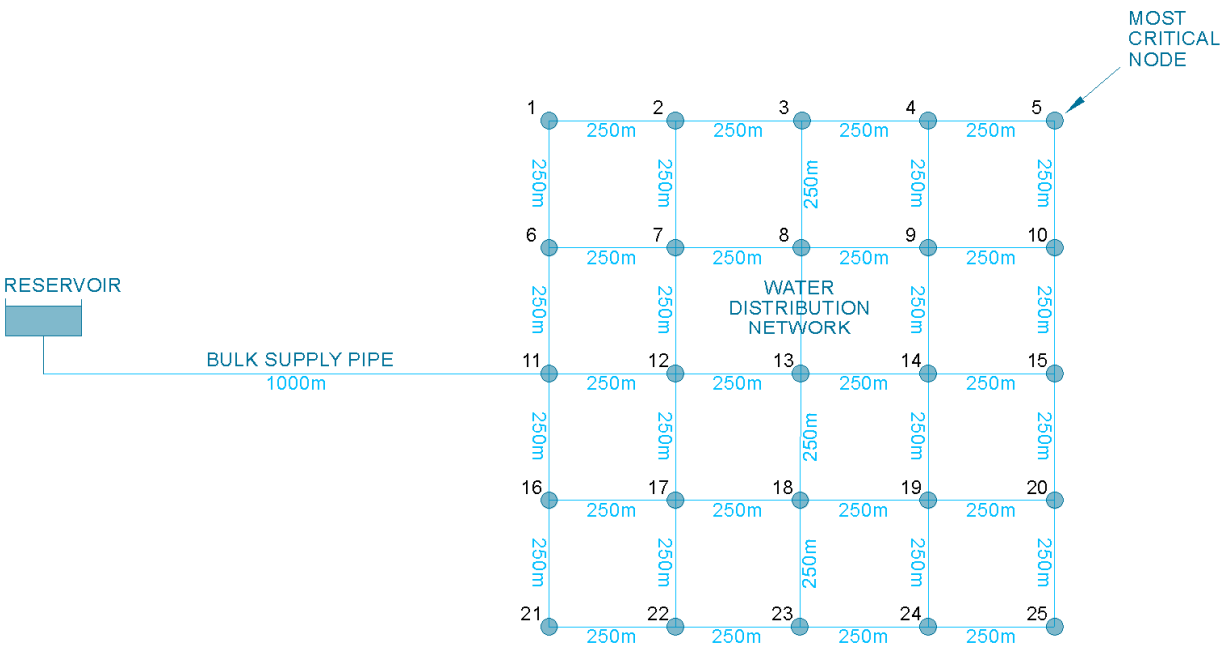


Figure 4.3: Schematic layout of typical theoretical hydraulic model

A total theoretical water demand of 1000 kl/day (1kl/day/stand) was evenly allocated to the nodes. For time simulation analysis purposes the “Res < 1000” weekly water demand pattern (see Figure 4.1) was allocated to all the nodes.

4.2.6 Hydraulic analysis and network optimisation

Firstly, a series of steady state hydraulic analyses were performed (using the theoretical peak hour factor of 4.6 as per Table 3.2) to optimise the design of the bulk supply pipe size and the internal diameters of the network until a minimum pressure head of exactly 24m was reached at the most critical node in the system under theoretical peak demand. As can be expected the most critical node in the system is the node with the highest ground elevation and which is also the furthest away from the supply point (see node no 5 as indicated on Figure 4.3).

Once the steady state model had been set up and optimised, a time simulation analysis using the “Res < 1000” water demand pattern for each node was conducted for a seven-day demand period comprising of one-hour time steps. This analysis is referred to as TimeSim24. The results of the TimeSim24 analysis were then compared to the results of the steady state analysis and it was confirmed that a pressure head of exactly 24m was experienced at the most critical node for exactly one hour (the peak hour within the peak day of the week) for the time simulation. This corresponded correctly to the results of the optimised system on which the steady state analysis was run.

Using the exact same network setup as for TimeSim24, an additional time simulation analysis was then conducted, but with the top water level of the supplying reservoir lowered by exactly 9m, effectively simulating a scenario where the system would experience an MPH of exactly 15m at the most critical node during peak hour demand. This analysis is referred to as TimeSim15.

4.2.7 Statistical analyses

After performing the hydraulic time simulation analysis for TimeSim24 and TimeSim15 the nodal result tables were again exported from WADISO to Microsoft Excel where the statistical analyses were performed. A 168h time-head table was exported for all 25 nodes in the system for both simulations. For each node the weekly nodal results were statistically analysed to include minimum and maximum values of H, average H, standard deviation of H and the total amount of time that H was below 24m for the node. The hydraulic results statistics for TimeSim24 were then compared to those of TimeSim15.

4.2.8 Results

The nodal result tables and the results of the statistical analyses for both simulations are included for each analysed node in Appendix D. The weekly time graph of H for all nodes for TimeSim24 is shown in Figure 4.4.

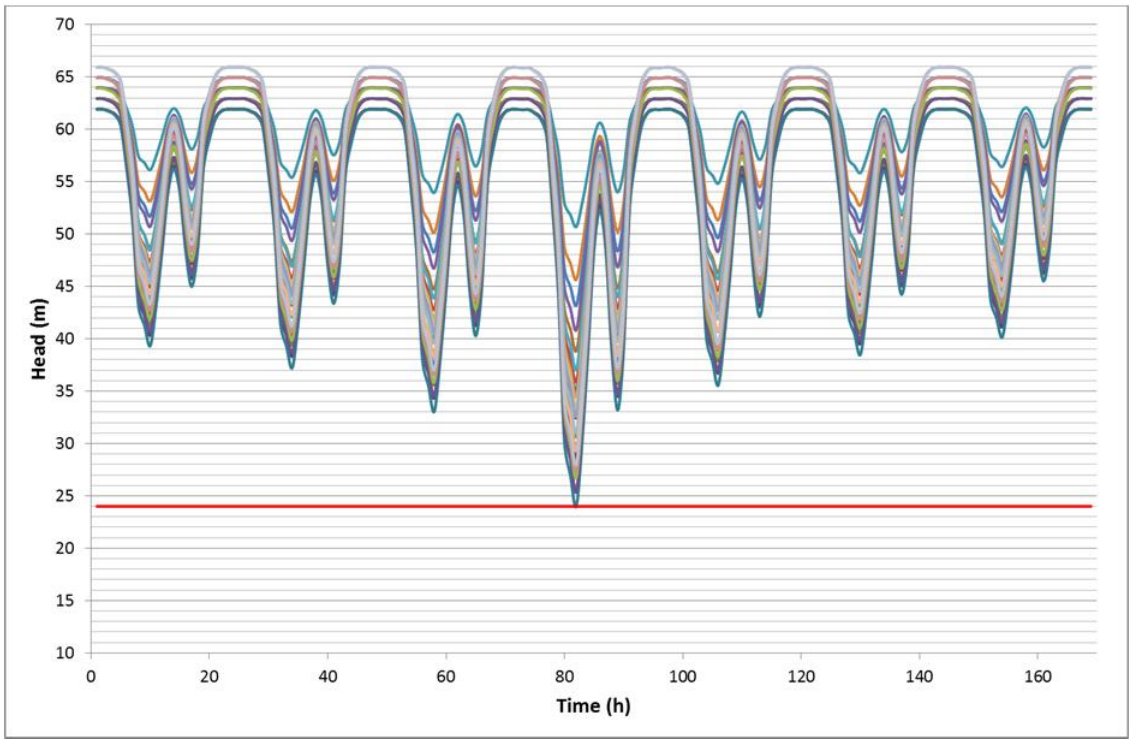


Figure 4.4: Time simulation results for H – TimeSim24

As can be seen from Figure 4.4 that all the nodes in the system experience H > 24m for the entire weekly period with the most critical node in the system experiencing MPH = 24m at the peak hour within the week. This therefore simulates a “perfectly optimised” water distribution network that conforms to the current design criterion. The second analysis simulates a water distribution system that conforms to a “relaxed” design criterion of MPH > 15m. The weekly time graph of H for all nodes for TimeSim15 is shown in Figure 4.5.

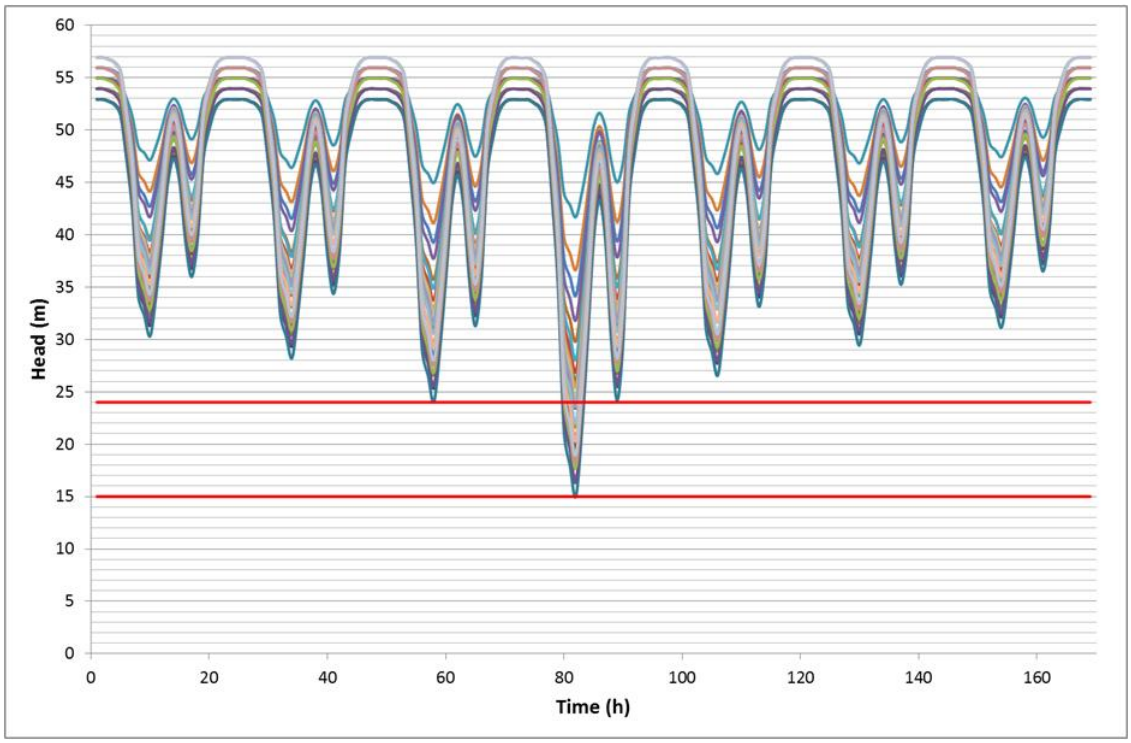


Figure 4.5: Time simulation results for H – TimeSim15

From Figure 4.5, it can be noted that all of the nodes in the system experience H > 15m for the entire weekly period with the most critical node in the system experiencing MPH = 15m at the peak hour within the week. What is interesting, however, is that for this relaxed criterion most of the nodes still experience H > 24m for most of the time. The statistics for both simulations were calculated and are summarised in Table 4.3.

Table 4.3: TimeSim24 vs TimeSim15 comparison

Node nr	TimeSim24				TimeSim15							
	General statistics				General statistics				Hours (h)		% of time	
	Ave H (m)	St dev H (m)	Min H (m)	Max H (m)	Ave H (m)	St dev H (m)	Min H (m)	Max H (m)	H < 24m	H > 24m	H < 24m	H > 24m
1	54.15	7.80	29.54	61.94	45.15	7.80	20.54	52.94	2.0	166.0	1.2%	98.8%
2	53.92	8.02	28.62	61.93	44.92	8.02	19.62	52.93	2.0	166.0	1.2%	98.8%
3	53.55	8.40	27.05	61.93	44.55	8.40	18.05	52.93	4.0	164.0	2.4%	97.6%
4	53.14	8.80	25.38	61.93	44.14	8.80	16.38	52.93	4.0	164.0	2.4%	97.6%
5	52.73	9.20	24.00	61.92	43.73	9.20	15.00	52.92	4.0	164.0	2.4%	97.6%
6	57.15	5.81	38.83	62.95	48.15	5.81	29.83	53.95	0.0	168.0	0.0%	100.0%
7	56.24	6.71	35.05	62.95	47.24	6.71	26.05	53.95	0.0	168.0	0.0%	100.0%
8	55.61	7.34	32.46	62.94	46.61	7.34	23.46	53.94	1.0	167.0	0.6%	99.4%
9	55.18	7.77	30.66	62.94	46.18	7.77	21.66	53.94	1.0	167.0	0.6%	99.4%
10	53.90	9.04	25.39	62.93	44.90	9.04	16.39	53.93	4.0	164.0	2.4%	97.6%
11	60.78	3.20	50.69	63.97	51.78	3.20	41.69	54.97	0.0	168.0	0.0%	100.0%
12	59.56	4.41	45.64	63.96	50.56	4.41	36.64	54.96	0.0	168.0	0.0%	100.0%
13	58.97	5.00	43.19	63.96	49.97	5.00	34.19	54.96	0.0	168.0	0.0%	100.0%
14	57.20	6.75	35.90	63.95	48.20	6.75	26.90	54.95	0.0	168.0	0.0%	100.0%
15	54.99	8.95	26.73	63.93	45.99	8.95	17.73	54.93	2.0	166.0	1.2%	98.8%
16	59.15	5.81	40.83	64.95	50.15	5.81	31.83	55.95	0.0	168.0	0.0%	100.0%
17	58.24	6.71	37.05	64.95	49.24	6.71	28.05	55.95	0.0	168.0	0.0%	100.0%
18	57.61	7.34	34.46	64.94	48.61	7.34	25.46	55.94	0.0	168.0	0.0%	100.0%
19	57.18	7.77	32.66	64.94	48.18	7.77	23.66	55.94	1.0	167.0	0.6%	99.4%
20	55.90	9.04	27.39	64.93	46.90	9.04	18.39	55.93	2.0	166.0	1.2%	98.8%
21	58.15	7.80	33.53	65.94	49.15	7.80	24.53	56.94	0.0	168.0	0.0%	100.0%
22	57.92	8.02	32.62	65.93	48.92	8.02	23.62	56.93	1.0	167.0	0.6%	99.4%
23	57.55	8.40	31.05	65.93	48.55	8.40	22.05	56.93	1.0	167.0	0.6%	99.4%
24	57.14	8.80	29.38	65.93	48.14	8.80	20.38	56.93	2.0	166.0	1.2%	98.8%
25	56.73	9.20	28.01	65.92	47.73	9.20	19.01	56.92	2.0	166.0	1.2%	98.8%
	56.51				**47.51**				**33.0**	**4167.0**	**0.8%**	**99.2%**

Table 4.3 clearly indicates that if the current design criterion of MPH > 24m at the most critical node in a system were relaxed to MPH > 15m, nodes within the system would still conform to the original higher criterion of MPH > 24m for 99.2% of the time. Furthermore, under the relaxed criterion 84% of all nodes will spend less than 2% of the time below the higher H value of 24m. For the TimeSim15 analysis the percentage of nodes versus the percentage of time spent with H < 24m is summarised in Table 4.4.

Table 4.4: TimeSim15 node performance summary

TimeSim15	
Percentage of nodes	Percentage of time spent with H < 24m
100%	< 2.5%
84%	< 2.0%
60%	< 1.0%
40%	< 0.5%

4.2.9 Discussion 2:

A relaxation of the current design criterion of MPH > 24m at the most critical node by 9m to MPH > 15m at the most critical node constitutes a relaxation of approximately 38%. Yet, under such a significant relaxed criterion the nodes in the system still comply with MPH > 24m for most of the time. This is due to the following two factors:

- The fact that the design for the total network is based on the pressures experienced at the most critical node. In water distribution zones with a large elevation difference between the highest and the lowest node, the pressures at the least critical nodes become significantly high to satisfy the minimum design criterion at the most critical node. In the typical theoretical model analysed the elevation difference between the highest and the lowest node was a mere 4m (which suggests a relatively flat residential area). In reality, much more significant elevation differences exist in large distribution zones. Large elevation differences within the same pressure zone causes unnecessary high pressures at the less critical nodes in the system because the pressure criterion at the most critical node must be satisfied.
- The fact that the theoretical peak demand condition so seldom occurs. This is illustrated clearly when considering the most critical node in isolation, thus ignoring elevation difference in the zone. The average head at the most critical node is 55% higher than the minimum head for TimeSim24 and 66% higher for TimeSim15. Also, for both simulations the average nodal head for the zone is significantly higher than the minimum head.

The results suggest that a significant decrease in MPH criterion can be applied without significant effects on nodes in the system in terms of low pressures. The obvious next step is to establish the possible cost saving that the application of such a relaxed criterion could have for water-providing authorities.

5 COST COMPARISON

The advantages of applying a lower MPH criterion include saving on infrastructure cost, saving on energy cost, saving on water use and saving on water losses – all of which contribute to extra revenue for water-providing authorities. This chapter presents the in-depth investigation that was performed to determine the possible saving that application of a decreased MPH criterion will have on infrastructure cost. This was done by firstly compiling a complete comprehensive water master plan for a selected case study town and designing upgrading requirements and new infrastructure in accordance with the current design criterion of MPH > 24m. Secondly the above process was repeated but in accordance with a relaxed criterion of MPH > 15m, after which the cost of required infrastructure was compared between the two scenarios. All other variables remained the same.

5.1 Hydraulic model analysed

The choice of a suitable hydraulic model on which to perform the above investigation was influenced by the following conditions:

- A hydraulic model was sought with reliable data in terms of current infrastructure, water demand figures and hydraulic results.
- The water distribution system should include discrete pressure zones with good variation in terms of average zone pressures.
- The town should include variation in different land use types.
- The water distribution system should be relatively uncomplicated.
- The town should include comprehensive expected future spatial development for which reliable spatial development data is available.

The town of Nigel in the Ekurhuleni Metropolitan Municipality satisfied all the above conditions and was subsequently selected for the cost comparison case study.

5.2 Methodology

5.2.1 Compilation and hydraulic population of the master plan model

The master plan model was compiled by chronologically adding and populating the following modelled entities:

5.2.1.1 Existing system

The existing system includes all modelled entities as per the systems analysed in Chapter 3 and was used as the base from which the rest of the master plan model was compiled. Because the master plan is based on the ultimate future demand scenario, a different approach was required in terms of populating the water demand for the existing system. In the ultimate future demand scenario it is assumed that all existing stands are developed and occupied. Therefore, as opposed to assigning calculated AADDs to stands based on their respective actual measured water meter readings and assigning a water demand of zero to the vacant stands (as for the existing scenario analyses), all existing stands were considered to be fully occupied and a theoretical water demand based on the land use and stand size was assigned to each stand before allocating the stand's water demand to the nearest model node.

5.2.1.2 New infrastructure for existing un-serviced areas

New infrastructure for un-serviced areas includes services to be installed to informal settlements. A complete list of all informal settlements within the metro was obtained from the Ekurhuleni Metropolitan Municipality (EMM). This list was firstly compared to the latest available aerial photography after which certain adjustments were made in terms of whether the informal settlement actually still existed, the size of the settlement and the number of units. Secondly, the adjusted list of areas was judged in terms of suitability for residential development based on topographical and geological conditions. To judge a specific area in terms of suitability for development the informal settlements areas were superimposed graphically onto existing maps indicating areas unsuitable for development such as areas below the 1:100 year flood lines, dolomitic areas and shallow undermined areas. Each informal settlement was then assigned a status of either "to be relocated" or "to be upgraded in-situ".

Schematic internal water pipes were modelled within the informal settlements for which in-situ upgrading was planned. The schematic pipes were then connected to the existing system model at the nearest or most optimum locations. Theoretical water demands based

on the number of informal units were then allocated evenly to the modelled nodes within the informal settlement.

5.2.1.3 New infrastructure for existing serviced areas

A list of existing developed areas to be subdivided and densified was obtained from the EMM. The normal procedure would have been to model schematic pipes within these areas, populate the nodes with theoretical water demands based on the new density and to connect the schematic pipes to the existing surrounding water pipes. However, none of the densification areas received from the EMM was located within the town of Nigel and this procedure was therefore not necessary.

5.2.1.4 New infrastructure for future areas

The Metropolitan Spatial Development Framework (MSDF) is a document compiled by town planners wherein the development within the urban development boundary of a municipality is assessed, optimised and planned. The latest version of this document (EMM, 2011) was obtained from the EMM and was used to identify undeveloped areas earmarked for future development. A map indicating the future development areas and the planned land use for each of the areas was taken from the MSDF document and was used as a background to compile the final adjusted list of future development areas.

As for the informal settlements, maps of the areas unsuitable for development were superimposed onto the above MSDF map and areas below the 1:100 year flood lines, dolomitic areas and shallow undermined areas were subtracted from the original future development areas, resulting in the final adjusted future development areas suitable for development.

Schematic internal water networks for each of the above land parcels were modelled and were connected to the optimum connection point of the existing system model. The optimum connection point is defined as the connection point for which the least amount of capital expenditure is required for an effective connection. For most cases the optimal connection point is incidentally the point nearest to the new land parcel. However, where insufficient capacity is available from the nearest connection point an optimisation analysis is required to determine the optimum connection point. For some instances it proved to be more cost effective during the system layout optimisation not to supply a future development area from the existing system but rather from its own dedicated supply point (i.e. either from a new reservoir/tower or directly from the Rand Water bulk supply system from which the entire EMM is supplied). A theoretical water demand based on the unit water demands from Table

3.1, the size of land parcel and the type of land use was calculated for each land parcel. This theoretical water demand was then allocated evenly to the nodes of the schematic internal water network within the land parcel.

After the existing and future modelled entities were merged and populated with the expected future water demand the model was used to run a steady state hydraulic analysis representing the ultimate future fully developed demand scenario. At this stage no allowance was made for expected future upgrades. As could be expected, the results showed complete failure of the system indicating that the existing system could not supply the ultimate future demand scenario without the implementation of extensive infrastructure upgrading.

5.2.1.5 Upgrading requirements

Upgrading requirements include the required reinforcements to the existing system in order for the entire system to comply with the set design criterion. As mentioned earlier, the purpose of this chapter is to present the two completely separate designs that were performed to determine these upgrading requirements. Firstly, a design was done in order for the system to comply with the current design criterion of MPH > 24m and secondly, the design was re-performed utilising the relaxed criterion of MPH > 15m.

Note that reservoir- and water tower volumes were not analysed as a part of this study as only the top water level (and not the volume) of the reservoir or water tower has a direct impact on the residual pressure experienced within the water network. Therefore the construction of new required reservoirs and towers is not included in the list of upgrading requirements. Upgrading requirements designed for the two analysed scenarios were limited to pipes and pump stations only.

5.2.2 Calculating costs of required upgrading

Once the designs for both scenarios had been completed, a list of upgrading requirements was compiled, including pipe length and diameter (for pipes) as well as pump flow and head (for pump stations). Construction costs for all required upgrading items were calculated by using unit cost functions currently being used by GLS for costing of master plan projects for all their clients. The unit cost functions are revised annually by GLS and are based on the latest available rates obtained from construction tender documents for water infrastructure-related construction projects. These cost functions make provision for construction costs as well as material costs.

For the construction of new pipes the unit cost per meter is dependent on the diameter of the pipe as well as the location of the pipe (high rate for pipes under tarred roads, medium rate

for pipes within the road reserve and low rate for pipes in open undeveloped areas). For each pipe a connection cost based on the pipe diameter is also allocated for each point where the pipe has to connect to the existing system.

For the construction of new pump stations or the upgrading of existing pump stations the unit cost is dependent on the pumped flow and the energy required (which in turn is related to the required head to be pumped). The unit cost functions used to calculate the costs of the required upgrading are included in Appendix E.

For each proposed upgrading project provision was also made in the construction cost for preliminary and general costs associated with setting up a construction project as well as provision for professional fees for project managers, consultants and contingencies. This provision was made by adding 40% to the construction costs calculated by means of the cost functions.

5.2.3 Comparing costs of required upgrading

When the relaxed criterion of MPH > 15m was used to re-design the required system upgrades it was generally found that the required pipe diameter of the upgrading items decreased by one or two pipe sizes from the original diameters required for the current criterion of MPH > 24m. Due to the lower pressure criterion the required pumping head decreased by 9m for all booster pumps in the system.

The list of upgrading requirements, including the respective construction costs for both of the design scenarios is included in Appendix F. The upgrading items were split into bulk system requirements (pump stations and pipes larger than Ø160mm) and upgrading requirements to the reticulation system (pipes smaller or equal to Ø160mm). A summary of the comparison is included in Table 5.1.

Table 5.1: Cost comparison for MPH criteria

Description	Upgrade cost for criteria used				Cost saving
	MPH > 24m		MPH > 15m		
Total for bulk system	R	57 351 937	R	39 339 884	31.4%
Total for reticulation system	R	8 417 751	R	5 074 512	39.7%
Total for Nigel study area	R	65 769 688	R	44 414 396	32.5%

As could be expected, the costs of upgrading to the bulk system significantly exceeded the costs of upgrading to the reticulation system. However, the cost saving when using the relaxed design criterion is of similar magnitude for the bulk and the reticulation system. For the total system the results indicate a cost saving of 32.5% on required upgrading of infrastructure when the relaxed criterion of MPH > 15m is applied instead of MPH > 24m.

5.2.4 Discussion

A cost saving of 32.5% on required infrastructure upgrades under the relaxed criterion of MPH > 15m is considered significant. Furthermore, the above saving includes only the saving on initial capital costs for upgrading and does not take into account the continuous saving on energy costs due to pump stations operating at lower pumping heads. Energy usage required for pumping is directly proportional to the required pumping head. Therefore, if all pump stations operate at a pumping head of 9m less than originally required this will constitute a continuous cost saving on energy. Over the design lifetime of the system this continuous saving will be significant – especially given the current cost of energy and the expected exponential increase in energy costs. Under no circumstances were required pipe diameters decreased for pipes downstream of booster pump stations as this would have increased friction losses resulting in a higher required pumping head.

Should the system have been designed from scratch (i.e. for a new “greenfields” development or the hypothetical case where a new town is planned) according to the MPH > 15m criterion, the initial capital cost saving would be prominent as the existing bulk infrastructure (i.e. existing concrete water towers with fixed top water levels) places minimum requirements of their own on required infrastructure upgrading (i.e. pumps pumping into existing water towers). New required reservoirs and towers would require lower top water levels. This would result in significant savings on the construction costs for especially water towers. Reservoirs can be placed on lower elevations with decreasing effects on the length of the bulk supply pipes. Furthermore, certain sections of the existing infrastructure proved to be oversized and was therefore underutilised when applying the relaxed criterion.

6 CONCLUSION AND FUTURE WORK

6.1 Conclusion

The results of this study suggest that the current design criterion of MPH > 24m at the most critical node in a water distribution system under theoretical peak demand is relatively conservative.

Approximately 16% of modelled nodes analysed as part of this research experienced peak demand pressures below the current design criterion of 24m. Yet, relatively few customer complaints were received in areas where hydraulic models indicate minimum residual pressure heads between 12m and 24m under peak demand. Furthermore, the lower limit placed on system pressure by household appliances is only 10m.

The philosophy of designing for the theoretical peak demand condition leads to a system where many nodes experience pressures in excess of the criteria for most of the time. Based on the hypothetical model evaluated as part of this research, relaxing the criterion from MPH > 24m to MPH > 15m could result in modelled nodes with H > 24m for 99.2% of the time. It should be kept in mind that theoretical peak factors used for analysis and design are known to be conservative.

A significant cost saving on required infrastructure upgrades was evident from the Nigel case study when employing a relaxed design criterion. Although it was not investigated as part of the scope of this study, the cost saving on energy required for pumping, reduced water use and reduced water loss resulting from lower system pressures is also considered to be significant. It is not only the effect on revenue that needs to be considered though. Water is a scarce and valuable resource. South Africa is a water scarce country and intervention is required to conserve South African water resources. Water losses are currently costing the South African economy R7.2 billion per year. Many different methods are currently being applied by water-providing authorities to reduce water loss. Of these methods, pressure reduction has proven to be the method with the biggest immediate impact.

6.2 Possible future work

Based on the finding that the current MPH design criterion is relatively stringent and results in overspending on infrastructure as well as high water use and high water loss, the criterion for MPH could possibly be relaxed. The following alternatives for relaxing the criterion could be considered for trial implementation as part of future research.

6.2.1 Adopting the use of probabilistic peak factors

Designing storm water systems based on different rainfall return periods has become standard practice in the field of hydrology. It is recommended that the same approach be adopted in the design of water distribution systems. Probabilistic peak factors for design have been derived by Booyens and Haarhoff (2002). The use of these peak factors results in a significant reduction in the peak water demand modelled for designing water networks and therefore constitutes a significant relaxation of the design criterion when the MPH value of 24m remains in use. It is recommended that the probabilistic peak factors derived by Booyens and Haarhoff (2002) be incorporated into the design of water networks.

The use of probabilistic peak factors would only be possible if a certain acceptable level of risk is allocated to different types of consumers. A risk of sub-standard pressures at shorter return periods could for example be allocated to residential areas (with segregation between different residential types) and longer return periods for more critical consumers like business, commercial and industrial consumers. The longest return period could be allocated to the most critical consumers like hospitals. Table 6.1 illustrates how the concepts of return periods could be linked to different consumer types and would be used in conjunction with the Booyens and Haarhoff (2002) probabilistic peak factors, indicated on Figure 2.2.

Table 6.1: Return periods for consumer types

Land use	Return period
Residential	
Extra large erf - 1 501m² and larger (Res 1)	1 year
Large sized erf 1001m² - 1500m² (Res 1)	
Medium sized erf 501m² - 1000m² (Res 1)	100 days
Conventional small sized erf up to 500m² (Res 1)	
Cluster housing up to 20 units per hectare (Res 2)	
Cluster housing 21 up to 40 units per hectare (Res 3)	50 days
Cluster housing 41 up to 60 units per hectare (Res 4)	
Cluster housing 61 up to 80 units per hectare (Res 4)	
Cluster housing 81 up to 100 units per ha (Res 5)	10 days
Low cost housing - erf up to 500m²	
Flats (± 50m² per unit)	
Business/commercial	
High water use priority	1 year
Low water use priority	100 days
Industrial	
Industrial dry	10 days
Industrial wet (dependent on pressure)	1 year
Industrial wet (dependent on volume)	100 days
Institutional	
Club buildings	10 days
Club grounds	
Stadium building	
Stadium grounds	
Municipal park buildings	
Municipal park grounds	
Hospital buildings	10 years
Hospital grounds	10 days
Church buildings	50 days
Church grounds	10 days
School, creche, educational buildings	100 days
School, creche, educational grounds	10 days

6.2.2 Relaxing criterion value of 24m

Water-providing authorities may be reluctant to consider the probabilistic peak factor approach due to its complexity. An alternative approach would be to employ the original peak factors used for design, but that the MPH value of 24m be reduced into various H-categories as indicated in Table 6.2.

Table 6.2: Possible criterion for MPH in water networks

MPH criteria under theoretical peak demand	
MPH category	**Description**
H ≤ 12m	Unacceptable pressure head - pressure too low
12m ≤ H ≤ 15m	Low pressure - acceptable for certain low priority land use types
15m ≤ H ≤ 24m	Medium pressure - acceptable for certain medium priority land use types
H > 24m	Acceptable pressure - acceptable for all land use types

It would be possible to segregate Table 6.2 into different land use types as well. Further collaboration with water-providing authorities would be required to compile a more comprehensive table.

Should water-providing authorities be reluctant, in principle, to have different criteria for different land uses, the most simplistic approach would be to use MPH > 15m for all consumers in the system.

Future research could evaluate and refine the alternative methods for relaxing the MPH criteria listed above and test the application of each by means of case studies. It is hoped that the findings from this research would perculate with time and ultimately lead to a less conservative and more accurate yet practical approach to hydraulic modelling. Future research following from this thesis should result in improved potable water services planning.

REFERENCES

Anonymous. 2015. Personal interview held on 8 June 2015 with the chief engineer of the water planning department of a metropolitan municipality who wished to remain anonymous.

Bannister, J., 2011, e-mail, 4 August, Enquiries.Team@ofwat.gsi.gov.uk

Booyens, J.D. 2000. Spitsvloei in munisipale waterverspeidingsnetwerke. Unpublished master's thesis, Randse Afrikaanse Universiteit, Johannesburg, South Africa.

Booyens, J.D. & Haarhoff, J. 2002. Probabilistic peak factors for residential water demand in South Africa. *Paper presented at the Biennial Conference of the Water Institute of Southern Africa (WISA),* 19-23 May 2002, Durban, South Africa.

Browder, G.J. 2007. *Improving the performance of China's urban water utilities.* Washington (DC), USA: The World Bank. World Bank Report No. 40964.

Cassa, A.M. & Van Zyl, J.E. 2014. Predicting the leakage exponents of elastically deforming cracks in pipes. *Procedia Engineering*, 70, 302-310.

Cheung, P.B., Van Zyl, J.E. & Reis, L.F.R. 2005. Extension of EPANET for pressure driven demand modelling in water distribution system. In Savic, D.A., Walters, G.A., Khu, S.T., King, R. (eds.), *CCWI2005 Water Management for the 21st Century,* 311-316. Volume 1. Exeter, United Kingdom: University of Exeter, Centre for Water Systems.

City of Cape Town. 2011. *Integrated master planning of water and sanitation services for the City of Cape Town.* Prepared by GLS Consulting.

City of Gold Coast, Allconnex Water, Queensland Urban Utilities & Unity Water. 2012. *SEQ water and sewerage planning guidelines.* Queensland, Australia: City of Gold Coast, Allconnex Water, Queensland Urban Utilities & Unitywater.

City of Tshwane. 2010. *Water evaluation & planning criterion.* Tshwane, South Africa: City of Tshwane.

Council for Scientific and Industrial Research (CSIR). 2003. *Guidelines for Human Settlement and Design.* A report compiled under the patronage of the Department of Housing by the CSIR. Pretoria, South Africa: CSIR Building and Construction Technology. Report No. BOU/E2001 (Revised version).

Council for Scientific and Industrial Research (CSIR). 2000. *Guidelines for Human Settlement and Design.* A report compiled under the patronage of the Department of Housing by the CSIR. Pretoria, South Africa: CSIR Building and Construction Technology. Report No. BOU/E2001.

Council for Scientific and Industrial Research (CSIR). 1983. *Guidelines for the provision of engineering services and amenities in residential townships.* Compiled for the Department of Community Development by the CSIR.

Crowley, K. 2015. *Water shortages loom for SA: Worst drought in two decades.* Available from: <http://www.biznews.com/undictated/2015/05/20/water-shortages-loom-for-sa-worst-drought-in-two-decades/> [Accessed: 22 July 2015].

Department of Public Utilities. 2011. *Water distribution system design guidelines and standard specifications and details.* Richmond (VA), USA: Department of Public Utilities.

Ekurhuleni Metropolitan Municipality (EMM). 2011. Metropolitan Spatial Development Framework.

Ekurhuleni Metropolitan Municipality (EMM). 2007. *Water and sewer modelling guidelines.* Gauteng, South Africa: Ekurhuleni Metropolitan Municipality.

GLS. 2015. *Master plans*. [Online] Available: IMQS software.

Government of Vietnam. 2006. *Water supply - Distribution system and facilities: Design standard.* Hanoi, Vietnam: Ministry of Construction. Standard No. TCXDVN 33:2006.

Greater Vernon Water. 2013. *Subdivision and development servicing bylaw no. 2650, 2013*. Regional district of North Okanagan, 19.

Hare, A.H. 1989. Aspects on peak flows in water mains*. Institute of Municipal Engineering of Southern Africa (IMIESA),* 14(4), 24-28.

Hes, D. 2013. *'No drop': SA to tackle its water leaks.* Available from: <http://www.southafrica.info/about/sustainable/water-251013.htm#.Vcr_v_mqpBc> [Accessed: 12 August 2015].

Jacobs, H.E. & Fair, K.A. 2012. Evaluating a tool to increase information-processing capacity for consumer water meter data*. SA Journal of Information Management*, 14(1).

Jacobs, H.E. & Strijdom, J.L. 2009. Evaluation of minimum residual pressure as design criterion for South African water distribution systems. *Water SA,* 35(2), 183-191.

Kapelan, Z. 2009. Telephonic interview held on 5 October 2009 with Professor Zoran Kapelan, Professor at the Centre for Water Systems at the University of Exeter with over 20 years of research and consulting experience in various water engineering disciplines in the UK.

Leslie, R. 1957. Water reticulation: Records and standards. *The Transactions of the South African Institution of Civil Engineers,* 7(2), 74-77.

Moult, N. 2015. Personal interview held on the 16th of November 2015 with Mr. N. Moult, engineer at GLS Consulting currently involved in water loss reduction projects for Johannesburg Water.

Myburgh, H.M. & Jacobs, H.E. 2014. Water for firefighting in five South African towns. *Water SA,* 40(1), 11-18.

National Treasury. 2015. *Budget review.* Available from: <http://www.treasury.gov.za/documents/national%20budget/2015/review/FullReview.pdf> [Accessed: 14 August 2015].

Saldarriaga, J.G., Bernal, A. and Ochoa, S. 2009. *Optimized Design of Water Distribution Network Enlargements Using Resilience and Dissipated Power Concepts.*

Snyder, J.K., Deb, A.K., Grablutz, F.M., AWWA Research Foundation, McCammon, S.B., et al. 2002. *Impacts of fire flow on distribution system water quality, design and operation.* Denver (CO), USA: AWWA Research Foundation and American Water Works Association.

South African National Standards (SANS). 2012. *Water supply and drainage for buildings Part 1: Water supply installations for buildings.* Pretoria, South Africa: SABS Standards Division. Standard No. SANS 10252-1.

Strijdom, J.L. 2008. *The effect of minimum pressure head criteria on the required capital expenditure for the water distribution system of Nigel.* Unpublished water networks course assignment report. Stellenbosch University.

The Ohio State University. Undated. *Closed conduit flow: Design of water distribution systems.* Columbus (OH), USA: The Ohio State University.

The Presidency. 2008. *Infrastructure investment.* Available from: <http://www.thepresidency.gov.za/docs/reports/asgisa/infrastructure.pdf> [Accessed: 14 August 2015].

The Water Services Regulation Authority (OFWAT). 2014. *Companies' performance 2013-14.* Available from: <https://www.ofwat.gov.uk/publications/serviceanddelivery#> [Accessed: 14 July 2015].

The Water Services Regulation Authority (OFWAT). 2008. *The Guaranteed Standards Scheme.* (GSS). Available from: <https://www.ofwat.gov.uk/consumerissues/rightsresponsibilities/standards/gud_pro_gss08.pdf> [Accessed: 14 July 2015].

Van Zyl, J.E. & Sheppard, M. 2015. *Pressure management in water distribution systems.* One day course: 18 September 2015.

Van Zyl, J.E. 2014. Theoretical modeling of pressure and leakage in water distribution systems. *Procedia Engineering*, 89, 273-277.

Wagner, J., Shamir, U. & Marks, D. 1988. Water distribution reliability: Simulation methods. *Journal of Water Resources Planning and Management,* 114(3), 276-294.

West Coast District Municipality. 2013. *Water Master Plan.* Prepared by GLS Consulting.

Turner, R.H., Fowler, T.G., Manson, N.J. & Stephenson, D. 1997. *Optimisation of Rand water's distribution system.* Pretoria, South Africa: Water Research Commission. WRC Report No. 488/1/97.

Van Vuuren, S.J. & Van Beek, J.C. 1997. *Her-evaluering van die bestaande riglyne vir stedelike en industríële watervoorsiening gebaseer op gemete waterverbruike. Fase 1: Pretoria voorsieningsgebied.* Pretoria, South Africa: Water Research Commission. WRC Report No. 705/1/97.

Vorster J, Geustyn LC, Loubser BF, Tanner A and Wall K. 1995. A strategy and master plan for water supply, storage and distribution in the East Rand region. *J.S. Afr. Inst. Civ. Eng*. 37 (2) 1-5

Water Services Association of Australia (WSAA). 2007. Desired Standard of Service for Water Supply Networks. Melbourne, Australia: WSAA.

APPENDIX A:
NODAL RESULT TABLES: STEADY STATE ANAYLSES

APPENDIX B:
RELATIVE AND CUMULATIVE FREQUENCY HISTOGRAMS

APPENDIX C:
UNIT WATER DEMAND PATTERN TABLES

APPENDIX D:
NODAL RESULT TABLES: TIME SIMULATION

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DETAILED COST COMPARISON

APPENDIX G:
MAP OF SOUTH AFRICA INDICATING LOCATION OF ANALYSED MODELS