Development of a two-tier prioritisation-algorithm for the replacement of water reticulation pipes

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Declaration

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ABSTRACT

Water pipe replacement in ageing water networks needs to be prioritised within constraints of limited municipal budgets. Relatively higher water pipe failure frequency in a distribution zone could point to a higher replacement priority. Priorities are typically determined based on historically recorded pipe failures, but actual pipe failure data is often not available – especially in developing countries. Pipe failure records may be available for certain zones in a particular system, while no data may be available in other zones of the same system. Replacement priority cannot be limited exclusively to zones with failure data, so a method was devised to spatially extrapolate pipe failures from zones with failure data to other zones where no knowledge of historical failures is available. An algorithm was developed for this purpose to prioritise pipe replacement based on a two-tier structure, comprising physical and hydraulic characteristics. The following model parameters were incorporated: pipe material, diameter, remaining useful life, static pressure, residual pressure and reserve pressure ratio. Actual pipe failure frequency data for a South African study site with 2021 km of pipes and 12802 reported failure events over a period of 180 consecutive months was obtained and used to devise the model. Actual pipe failures were linked to the different model parameters, with all parameter values known per pipe in the case study area. Pipe failure likelihood index values were then calculated for each pipe element in the water network model (as failure/year/meter). Each pipe was then prioritised for replacement in terms of a failure likelihood index, and grouped per water distribution zone. The water distribution zones were ranked for replacement prioritisation. The model was verified by evaluating failure likelihood index values and comparing replacement priority per zone based on actual data to the model results (for those zones with known data). The model was subsequently used to extrapolate the replacement priority to other zones without failure records in the case study area, with acknowledgement of in accuracy due to the lack of model validation. The model results are illustrative and apply to the specific study site – results should not be generalised. The results were represented spatially in GIS format, allowing the user to visually identify the most critical areas for pipe replacement. Future research could involve model validation and possible application beyond the study sample.

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LIST OF ABBREVIATIONS AND ACRONYMS

AADD	Average Annual Daily Demand
AC	Asbestos Cement
CCTV	Closed-Circuit Television
CI	Cast Iron
CSIR	Council for Scientific and Industrial Research
DI	Ductile Iron
GIS	Geographic Information System
GRP	Glass Reinforced Polyester
HDPE	High-Density Polyethylene
HW	Hazen-Williams
IBIS	Integrated Business Information System
LDPE	Low-Density Polyethylene
LEYP	Linear Extended Yule Process
MDPE	Medium Density Polyethylene
mPVC	Modified Polyvinyl Chloride
NHBP	Non-Homogeneous Birth Process
NPV	Net Present Value
oPVC	Biaxially Oriented Polyvinyl Chloride
PE	Polyethylene
P-I	Probability-Impact
PIM	Probability and Impact Matrix
PIP	Probability and Impact Picture
PRP	Pipe Replacement Prioritisation
PVC	Polyvinyl Chloride
RUL	Remaining Useful Life
SAPPMA	Southern African Plastic Pipe Manufacturers Association
SRA	Schedule Risk Analysis
SSI	Schedule Sensitivity Index

ST	Steel
UIM	Uncertainty-Importance Matrix
uPVC	Un-plasticised Polyvinyl Chloride
WALM	Weibull Accelerated Lifetime Model

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1. INTRODUCTION

1.1 Background

Water reticulation pipe failures and deterioration of the water distribution network cause unnecessary stress on municipal budgets, in an economy with existing budget constraints. An optimised relationship between maintenance and replacement strategies is needed, in order to improve the allocation of funds. The water reticulation pipes and network areas evaluated as the most critical under a pipe replacement prioritisation (PRP) analysis, require soonest replacement. Identifying and replacing the most critical pipes, effectively enhances budget spending.

1.2 Research problem and methodology

The research was conducted to address the following question: Which areas or pipes in a water distribution system are the most critical to replace and which would most improve the value of effective budget spending?

As part of the research, an investigation into the causes of water reticulation pipe failures and pipe failure attributes was completed. The attributes were modelled as failure factors and rearranged into an algorithm, where the different failure frequencies were determined. The failure frequency was determined for each pipe, as well as an appropriately calculated failure frequency index. A three-phased likelihood-of-failure allocation process was followed; after which prioritisation was implemented at the end of any of the phases. The three-phase process was implemented on a study area in the City of Tshwane Municipality's water reticulation pressure zone. The study area was used to illustrate the application of the algorithm. A complete hydraulic model of the water distribution system, which contained all the required PRP-data, was available for use during the research study.

The hydraulic modelling software used was able to support a geographic information system (GIS). The water system models with all data and information required for this study were provided for the purpose of this research by the City of Tshwane Municipality on 31 January 2015. The available water distribution model was used as a basis for testing application of the PRP-analysis. The development of the particular PRP-algorithm introduced in the thesis, made use of *Wadiso® (GLS Software), Swift® (GLS Software), Albion® (GLS Software) and Microsoft Excel* software.

1.3 Scope of work

The water reticulation PRP-algorithm developed in the thesis, was implemented in a case study that comprised water reticulation zones south of the Magaliesberg as far as the Constantia Park tower zone, within the City of Tshwane Municipality boundaries. The case study consisted of 2021 km of water reticulation pipeline. The model was last updated in January 2015 (prior to this research study). The model was made available with zero outputs at all nodes.

However, a database with monthly water consumption per consumer in the study area, was also made available for this study. The consumption could be used to derive peak hourly flows (for model node outputs). As part of this study the hydraulic model was populated with the January 2015 water demand billing data and analysed via a steady state analysis to generate the system's hydraulic properties. As part of this research study 12802 reported pipe failures over a period of 15 years were also successfully captured and integrated into the model.

1.4 Limitations

Data availability and integrity was a challenge, in some parts of the system, due to the inadequate record keeping systems of as-built drawings. Over time the data integrity problems were eliminated, when pipe surveys were completed or when pipe replacements occurred, and their as-built drawings were made available. For the thesis case study, the PRP-algorithm implementation required a significant amount of data input, analysing and processing. Data integrity was an important factor necessary to obtain accurate results. The highest possible level of data integrity was maintained throughout the project.

The model developed as part of the research involved a pipe failure frequency analysis ad-on to the hydraulic pipe network model and related GIS. Various other factors were known to affect pipe replacement, but the study was limited to pipe failures.

A few other reasons for replacing pipes may include the improvement of water quality (when there is internal surface degradation), health risks or perceived health risks (removing asbestos cement pipes). Other reasons for replacing pipes may also include planned pipe changes to provide for the demarcation of district metered areas and increasing the hydraulic capacity (replacing the pipe with a new larger pipe). The aspects of water quality, perceived health risks and planned pipe changes as a motivation for pipe replacements are considered to be beyond the scope of this study.

By considering the static water pressure and residual water pressure, the model developed as part of the research study included parameters describing hydraulic capacity. Pipe material, pressure rating, age and diameter were included in the algorithm. Flow velocity as an algorithm parameter was investigated, but excluded from the PRP-analysis as the data gathered from parameter results proofed to be of little consequence. As part of future work the algorithm could be extended by allowing for the inclusion of additional parameters.

2 LITERATURE REVIEW

2.1 Overview

Optimised municipal budget spending, with regard to maintaining, upgrading and replacing existing water distribution infrastructures, has been noted to be important (Giustolisi and Berardi, 2009). Pipe replacements target one of these aspects and greatly influence infrastructure maintenance and upgrade programmes. However, the biggest problem is found with prioritising and optimising pipe replacements. With an absolute pipe replacement priority strategy, room would be allowed for the efficient use of existing budgets, which will decrease the risk of service delivery issues.

First, an understanding of the causes of pipe failure was required (Makar et al., 2001). The understanding needed to include failure behaviour, as well as all the different factors involved during failure events. Secondly, by understanding water pipe failure conditions, research was required in order to fully understand the behaviour of different materials, as well as their mechanical properties and characteristics (Ferrante, 2012). Thirdly, with an understanding of material properties, further research was required to understand internal and external failure conditions, in combination with each water pipe material (Yna, 2013). The investigation supplied sufficient data to develop a tool for calculating the likelihood of pipe failure. Finally, different ways were identified for prioritising the likelihood of pipe failure for developing an appropriate prioritisation algorithm, which ultimately served as a useful water reticulation PRP-tool (Rogers and Grogg, 2006).

2.2 Pipe Failures

Cassa (2005) describe pipe failures as events where water loss occurs through non-maintainable items, which require intervention by repair or replacement of the pipe, fittings, or joints. The pipe failure events disturb the water distribution network lifecycle, as a result of a wide variety of factors. The explanation on pipe failures focuses on two aspects, namely (i) failure types and mechanisms, and (ii) pipe materials.

2.2.1 Types of pipe failure behaviour and characteristics

Pipes can fail in different ways, which can have different consequences. The following failure types exist, as described below (Rizzo, 2010):

- Circumferential crack – Pipe failures where the crack develops around the circumference of the pipe (Rahman et al.,1998). The circumferential crack failure can cause significate leakage and can result in complete rupture when exposed to bending motion.
- Piece blowout – Pipe failures where the internal pressure blows out a piece of wall material (Rajeev et al., 2013). Blow outs are typically caused by reduction of wall thickness at a certain spot. Water pipe corrosion and erosion cause reduction of wall thickness.
- Bell split – Pipe failures where the bell of the pipe initially splits. The crack propagates longitudinally down the length of the pipe and eventually turns towards the pipe circumference near the termination point (Rajani and Abdel-Akher, 2013).
- Spiral failure: Pipe failures where a spiral crack starts off as a circumferential crack and then develops longitudinally in a spiral formation (Bernasovský, 2013).
- Longitudinal crack – Pipe failures where the crack develops down the length of the pipe, after which internal water pressure can cause the top of the pipe to blow or break off (Makar et al., 2001). Water pressure surges and pipe wall corrosion often cause longitudinal cracks.
- Wedge splitting – Pipe failures that develops when a bell crack is split off to relieve bending stresses (Cai et al., 2016).
- Brittle failure – Pipe failures where a longitudinal crack develops due to the inadequate brittle characteristics of materials (Cui et al.,2010).
- Ductile failure – Pipe failures where a type of crack develops after which the material tends to stretch and fail due to its high degree of ductility (Han et al., 2014).

2.2.2 Causes of water pipe failure

According to Scruton (2012), causes of pipe failure could be classified by considering the following categories:

- Manufacturing defects and construction factors.
- Soil movement.
- Extreme temperature changes.
- Tensile and compression failures.
- Material properties.
- Hydraulic factors.

The categories are interlinked and can act as a combination of categories during a failure event, as illustrated in Figure2.1.

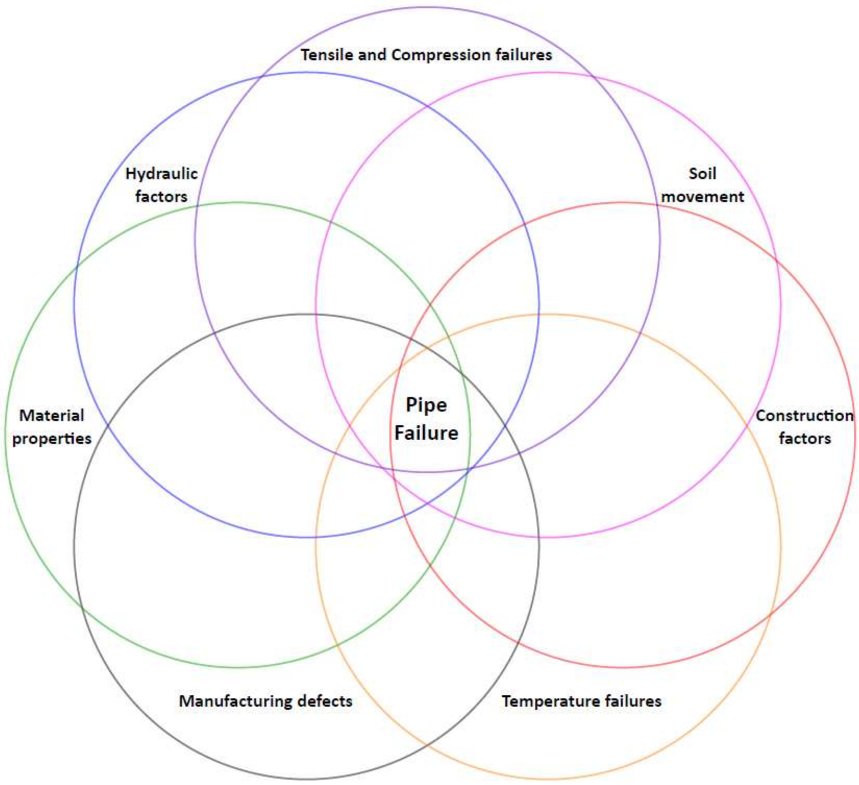

Figure 2.1 Categories of the causes of water pipe failure (Scruton, 2012).

Defects could develop during the pipe manufacturing process (Al-Barqawi and Zayed, 2006). Manufacturing defects could include discrepancies in wall thickness, composition, misshaped structures, shape and poor joint connections, which can all cause pipe failures.

Even pipes with no manufacturing defects could fail after commissioning due to construction negligence (Farshad, 2006). Construction negligence typically comprise of damaging handling methods, misalignment of joints, lack of material protection, undesired construction techniques, incorrect trench dimensions and inappropriate bedding and backfill materials. During construction, multiple factors can contribute to a single defect, which can result in problematic failure events (Farshad, 2006).

Soil movement could also cause pipe failures (Casamichele et al., 2004). Conditions of soil movement include aspects of external loads, internal pressure, water hammer, upward soil pressure, live loads, pipe weight, pipe bedding and backfill soil scouring, as well as moving soil. The conditions of soil movement can cause crushing, compression, tensile failure, longitudinal bending, excessive deflection, buckling, shear fracture and torque on pipes. Hence the importance of selecting appropriate water pipe materials for the soil conditions present.

Pipe failures can be caused by extreme temperature changes (Farmania et al., 2017), which are above the specification for a material. The temperature changes can influence the material structure, which can lead to excessive tensile or compression stresses. Failure events can be triggered more easily than anticipated with the excess tensile or compression stresses present. According to (Cassa, 2005), material tensile or compression stresses can cause pipe failures. The following failure types exist when subdividing material tensile failure:

- Bending failure – Pipe failures caused as a result of bending, which creates ductile or brittle failures.
- Brittle tensile failure – Pipe failures caused due to a sudden failure, which occurs when the material was adequate one moment and failed the next.
- Ductile tensile failure – Pipe failures caused by necking, which occurs when the material surpasses its yield strength and is stretched past its ultimate tensile strength.
- Fatigue failure – Pipe failures caused by cyclic tensile load, which occurs when a small portion of the material is subjected to a load beyond the ultimate tensile strength and generates a crack. This crack develops further every time the stress increases beyond the ultimate tensile strength.

Tensile failures tend to be a common occurrence in pipes: compression failures, on the contrary, tend to be rare (Seica and Packer, 2004). Compression forces on the pipe, push the material past its yield strength, which results in reduction of a cross-sectional area of the pipe, which ultimately causes a compression pipe failure (Seica and Packer, 2004).

Poor selection of pipe material, for the installation of a specified application, can cause pipe failures to occur (Hou et al., 2016). If the pipe material properties are not adequate to handle the internal and external conditions, accelerated material deterioration can occur, which will ultimately lead to a pipe failure.

According to Mylapilli et al. (2015), due to various hydraulic factors, which include head loss, internal pressure thrust, water hammer and internal pipe erosion, failures occur. Internal pipe erosion can occur once soil intrudes the network, under negative pressure conditions.

2.3 Pipe material properties

2.3.1 Criteria for assessment of pipe material performance

The discussion on material behaviour includes mechanical properties, physical properties, system properties and characteristics, pipe assessment and environment interactions. The following principal criteria were identified by Illston and Domone (2001) to assess pipe material performance during construction and in subsequent service:

- Strength – Resistance to internal pressure.
- Stiffness – Resistance to loading and deformation under stress.
- Toughness – Resistance to rapid crack propagation.
- Chemical resistance – Resistance to slow crack growth.
- Water tightness – The ability to prevent any form of leakage through pipe or joint.
- The speed of installation – Time taken to install a pipe, which includes the handling of pipes.
- Environmental impacts – Physical effect the pipe has on the environment, caused either through installation or the fabrication of the material.

2.3.2 Material mechanical properties

All materials have basic mechanical properties, which can be used to assess and characterise the material for a specific application (SAPPMA, 2013). The core material mechanical properties are as follows (SAPPMA, 2013):

- Hardness – Resistance to penetration or indentation.
- Tensile Yield – Maximum stress a material can withstand while stretched before deformation occurs.
- Ultimate Yield – Maximum stress a material can withstand while stretched before breaking.
- Ultimate Elongation – Measurement of the maximum length a material can stretch before breaking, expressed as a percentage of its original length.
- Elastic Modulus – A number that represents a material’s ability to resist deformation under stress.
- Flexural Stress – Maximum stress a material can withstand, before yielding to a flexural test.
- Notched impact– Amount of energy a material absorbs during fracture.
- Thermal stability – Material's resistance to decomposition at high or low temperatures.
- Poissons ratio – Ratio between material elongation, when stretched, and the contraction that occurs in the direction transverse to the direction of stretching.

2.3.3 Material physical properties

According to Fuoss (1955), all materials have certain basic physical properties, which can be used to assess and characterise the suitability of the material for a specific application. The core material physical properties are as follows:

- Density – The material's mass per unit volume.
- Melt flow index – This is a measurement of the ease of flow of a melted material.
- Vicat softening point – The determination of the temperature at which a material that has no definite melting point, such as a plastic, softens beyond some arbitrary predetermined point, tested by depth of penetration.
- Thermal conductivity – A material's ability to conduct heat.
- Flammability – A measurement of the percentage of oxygen needed to support the combustion of the material.

2.3.4 Positive material properties

All materials contain positive properties, depending on those required for their use. The following material properties are evaluated as positive (SAPPMA, 2013):

- High corrosion, chemical and abrasion resistance.
- Lightweight and easy to handle.
- Extended length availability, which reduces the number of joints required.
- High flexibility and toughness.
- Low friction resistance to flow.
- Ability to withstand water hammer.
- Low thermal conductivity.
- Low expansion and contraction coefficient.

2.3.5 Pipe characteristics and aspects

The US Agency for International Development (1982) listed a number of primary pipe properties such as shape, diameter, wall thickness and roughness coefficient. Secondary pipe properties also exist, such as design life, manufacturing, installation and pipe cost. The material properties are briefly reviewed in this section.

Water pipes come in different shapes for various purposes. The shape of a pipe can influence the hydraulic boundaries in water flow and velocity, as well as all pipe maintenance aspects

(Schirber, 2015). As technology and engineering have developed, circular pipes have become the preferred shape to transport water throughout distribution networks.

According to Zeghadnia et al. (2015), circular pipes are the most commonly used pipe shape for transporting water throughout distribution networks. Circular pipes are defined and measured according to their diameter. Pipe diameter consequently has a direct influence on the distribution system’s hydraulic capacity properties, which influence flow, velocity and headloss. All circular pipes consist of an inside diameter and outside diameter, which depends on the wall thickness and pipe material.

The wall thickness of a pipe is directly related to the material and diameter, which highly influences the maximum and operating pressure the water pipe can withstand (US Agency for International Development, 1982). The maximum pressure is referred to as the pressure rating. Wall thickness contributes not only to the pressure rating, but also to the pipe toughness, which is important when pipes are roughly handled or exposed to inappropriate soil conditions (Zhang et al., 2016). Appropriate wall thickness can, therefore, minimise the risk of pipe failure. According to Barlow’s formula, the wall thickness is mainly dependent on diameter and the material characteristics, expressed as follows:

$$P = \frac{2 \times f \times t}{D} \tag{2.1}$$

$P = Safe\ working\ pressure\ (MPa)$

$f = Safe\ working\ stress\ (MPa)$

$t = Pipe\ wall\ thickness\ (mm)$

$D = Pipe\ outside\ diameter\ (mm)$

Pipe roughness is a pipe characteristic, which is dependent on the pipes inner lining material and manufacturing procedures, and which is expressed as a friction coefficient (Nyende-Byakika, 2017).Pipe roughness directly influences energy loss within the pipe, which leads to more expensive upgrades, downstream of the pipe section. The pipe roughness can also promote pipe congestion and ageing over a period, as may be determined by physical pipe inspections (Shahzad James, 2002). Physical pipe inspections have determined that long-term pipe roughness is mostly dependent on water quality and the degree of exposure to flow velocity.

The design life of a pipe is dependent on the material degradation rate (Hancock, 2003). Design life is a valuable property, which can save significant amounts of money when the knowledge is integrated and optimised into maintenance and replacement strategies. The following pipe material design lives have been adopted in the City of Tshwane Municipality (GLS Consulting, 2012), as reflected in Table 2.1:

Table 2.1 Pipe material design life (GLS Consulting, 2012).

Pipe Material	Pipe Material Description	Design life (Years)
CI	Cast Iron	100
DI	Ductile Iron	100
STEEL	Steel	60
AC	Asbestos Cement	40
FC	Fibre Reinforced Cement	40
GRP	Glass Reinforced Plastic	60
HDPE	High-Density Polyethylene	80
mPVC	Modified Polyvinyl Chloride	50
uPVC	Un-plasticized Polyvinyl Chloride	50

Each pipe material is manufactured using a different process, whereby multiple techniques can be applied (Flowtite Technology AS, 2014). Some of the techniques are used to customise pipes for a specific purpose, with specific properties. These techniques contribute to the pipe’s roughness, its length and its quality. As quality is one of the focal points during pipe manufacturing, extensive control checks and qualification testing are done to ensure an acceptable product. According to Yeomans et al. (2012), control checks include visual inspections, and tests for Barcol hardness, wall thickness, length, diameter, hydrostatic leak tightness, stiffness, deflection, axial and circumferential tensile load capacity, as well as overall composition.

Pipe length manufacturing is restricted by the installation’s logistical problems and pipe specifications such as weight, flexibility and shear strength (Kruger, 2013). The length of manufactured pipe sections can greatly influence the risk of pipeline leakage; as the number of pipe joints required, over a long pipeline, increases with shorter length pipes (Hunaidi, 2000).

Flowtite Technology AS (2014) states that, after completion of the manufacturing inspections, the manufactured material undergoes qualification testing of joints, initial ring deflection, long-term ring bending, long-term pressure corrosion, long-term strain corrosion and long-term stiffness. Flowtite Technology also states that pipe installation takes place through direct bury, trenchless, above ground or subaqueous methods. Hermanson and Wagner (2015), states that pipes need to be installed successfully, which requires attention to aspects such as transportation from the manufacturer, material handling, on site storage, installation and project completion, which all need to be implemented with care. According to Flowtite Technology AS (2007), aspects such as trench sizing, pipe bedding, backfill materials, backfill compaction, installation method, pipe defects, pipe joints, thrust restraints and rigid structure connections are all critical during installation to ensure that the installation is successful.

After installation, most pipes tend to undergo maintenance, which is especially necessary if the pipe was incorrectly installed, damaged, badly designed or manufactured. The maintenance events are defined as repair work, inspections or cleaning, (Flowtite Technology AS, 2008).

The total pipe cost includes the pipe material manufacturing cost, as well as the construction costs (US Agency for International Development, 1982). The pipe cost represents Rand per length, which also include the cost of connections, fittings and joints. The overall pipe laying cost covers all costs during construction, up until successful and complete installation.

2.3.6 System properties

Each water distribution network is subjected to various internal and external factors that can have a substantial impact on a pipe failure event. The impacts can either be the cause of the failure event or the consequence thereof.

Water distribution networks are pressurised water systems, which could be segregated into bulk and reticulation sub-systems. The bulk system comprises all infrastructures that supply pressure zones in a water system, which includes a reservoir, tower or direct supply zones. The bulk systems tend to consist of larger diameter pipes. The reticulation system is all the infrastructures which fall under a single isolated pressure zone to supply consumers. Reticulation systems tend to consist of smaller diameter pipes. In the context of South African municipalities, the bulk and reticulation systems are handled separately and fall under different budgets. This research project focuses only on the water reticulation pipe infrastructure, with a case study in South Africa.

Combinations of internal and external factors influence a water distribution network. Internal factors are present as a result of characteristics of the reticulation, while external factors result from non-reticulation-related features. External factors could, for example, include environmental, geological and location aspects. According to Tesfamariam et al. (2006), the external system factors include soil conditions, strategic pipe locations, trench depths and pipe ground cover.

According to the CSIR (2005), the internal factors comprise of the following:

- Water demand and average annual daily demand.
- Peak factors and peak flow.
- Residual pressure.
- Static pressure.
- Flow velocity.
- Water hammer.
- Network redundancy.

Supplying water to consumers is the essence of a water distribution network. According to the CSIR (2005), water demand is the cumulatively calculated water use of an array of different consumers. Although the consumer billing records have been used in earlier studies (Jacobs and Fair, 2012), also to populate peak flows for hydraulic model outputs (Van Zyl et al., 2017), some consumers' billing could be missing. Theoretical water unit demands, calculated from historical water demand and land use datasets, could be assigned in such cases to consumers without billing information (Strijdom and Jacobs, 2016, CSIR, 2005).

Bose et al. (2012) describe peak factors as dimensionless values, which represent a relationship between peak and average consumption. The peak flow in any given pipe is the product of average annual daily demand and the peak factor allocated to that consumer. Calculating a system's peak factors needs to be approached carefully and requires a fair amount of thought. The following factors influence the calculation of the peak factor (Scheepers and Jacobs, 2012):

- Employment trends and practices in the community.
- Gardening activities.
- The number of persons per tap.
- Agricultural activities.
- The number of dwellings.
- Economic status.
- Unauthorised connections.

The residual pressure is the main focus in water distribution networks. The pressure at any point in a water reticulation network, inside a single pressure zone, under any demand scenario, is referred to as the residual pressure. According to Jacobs and Strijdom (2009), the minimum allowable residual pressure, at any moment in a water reticulation network, needs to be 24 m head, while home appliances specify a lower limit of 10 m head. Strijdom and Jacobs (2016) evaluated the residual pressure in South African water distribution systems and found that a successfully designed water reticulation network is isolated and operational under only one pressure zone, via either a reservoir, a tower, a booster pump or pressure reducing valves.

The maximum pressure at any point in a water distribution network, in a single pressure zone, with no demand accounted for, is referred to as the static pressure (Araujo et al., 2006). In South African distribution networks, the maximum static pressure allowed at any point is 90 m head (CSIR, 2005), but should be kept as low as possible, to reduce water loss.

Pipe flow velocity is a function of flow and the internal pipe area (Chadwick et al., 2004). Flow velocity is a critical variable to consider in any network evaluation process (Sitzenfrei et al., 2013). Flow velocity is an indication of flow behaviour, which greatly influences head loss, reticulation design life and water quality. According to the CSIR (2005), the maximum allowable flow velocity for any reticulation pipe in a water network is 1.2 m/s. Some municipalities accept a maximum allowable flow velocity of 2.2 m/s in any water reticulation pipe (GLS Consulting, 2015).

Water hammer is the result of a pressure change caused by a significant variation of flow rate in a water pipe created by a sudden start or stop of water flow (Wang et al., 2014). Due to the severe consequences resulting from water hammer, pipelines need to be designed carefully to take water hammer into account, over and above the effects of residual pressure, static pressure and flow velocity. According to the CSIR (2005), pipe materials with appropriate pressure ratings need to be considered, when accounting for water hammer.

Network redundancy is an important water reticulation design factor, which ensures that the consumer water supply has multiple routes to follow (Gupta et al., 2015). Network redundancy is necessary when the main water branch needs to be repaired, due to some failure. Network redundancy also plays a prominent role in ensuring sufficient supply under fire flow conditions. The consideration of network redundancy applies not only to the water reticulation systems, but also to the bulk systems, the demands of which are determined by the emergency or backup supply during times of maintenance or failure events.

Water pipe exposure to the local geology and soil conditions can have a drastic effect on pipe infrastructure. According to the CSIR (2005), all pipes should be installed with appropriately designed trench dimensions, as well as with appropriate bedding material, with a minimum thickness of 0.1m or 1/6 of the pipe diameter (whichever is the greatest) and adequate backfilling material. These construction considerations are significant, and can have a drastic effect on the life cycle of any pipe which is not correctly installed. The geological conditions in which pipes' installation occurs can severely influence the consequences of pipe failure event. Dearden et al. (2014) states that the following soil conditions develop from leaking pipes and increased ground instability:

- Dispersive soils: Some types of soil materials dissolve in water, which results in great underground cavities, better known as sinkholes. When cavities develop, above ground structures can collapse, which causes considerable damage to infrastructural assets.
- Landslides: An outward-downward gravitational force movement of soil materials along a slope. Water, from leaking pipes, can severely alter soil strength, which can instigate landslide events and cause considerable damage to infrastructural assets.
- Compressible ground: Some geological deposits can contain water-filled pores which, when compressed by infrastructure, can squeeze out the water and cause ground compression. Such events can cause uniform and non-uniform settling, which can damage water reticulation pipe infrastructure.
- Swelling clays: Clay soil can shrink and swell significantly, changing in volume depending on the moisture content. Leaking pipes are, therefore, a major contributor to swelling and may result in uplift or lateral stress on existing water reticulation pipe infrastructure by causing clay to swell, which can cause cracking and distortion. In such cases, oversaturation is also a significant risk and leads to flooding of above ground infrastructure.
- Running sands: A soil condition which occurs when loosely-packed sand, saturated with water, starts to flow into voids. The pressure of water filling spaces between the sand grains reduces the granular contact area, which causes the grains to be carried along. In such cases, the structural integrity of pipe trenches is compromised and this results in unforeseen loads on the underground infrastructure.

Pipes are installed in places with variable installation and maintenance costs (Van Zyl, 2014). The pipe installation locations are identified to include public open spaces, road reserve, underneath a road, underneath a building and above ground.

The strategic location of any pipe can be categorised further according to the following critical consumers (GLS Consulting, 2012):

- Hospital - Critical to save lives.
- Central business development (CBD) area - Critical for the local economy.
- Industrial area - Critical for fire flow.

According to GLS Consulting (2012), the strategic location of any pipe can be sub-categorised, which includes an effect on a pipeline's lifecycle, caused by corrosion potential. The strategic location subcategories are as follows:

- Next to a railway line - all metal pipes need cathodic protection to ensure that the pipeline does not experience electromagnetic corrosion. The electromagnetic corrosion can be caused by prolonged exposure to electromagnetic fields generated from the railway lines.
- Through a wetland - all metal pipes need corrosion protection, to prevent corrosion due to prolonged exposure to moisture in combination with external ground conditions.

Pipes must be designed and installed with adequate backfilling and bedding, as well as the correct excavated trench dimensions to ensure that the pipe structure can handle all the imposed loads (Goyns, 2012). When the trench depths and widths are insufficient, pipes undergo structural failure. The following basic principles can be implemented to calculate the minimum trench depth (CSIR, 2005):

- Road crossings: Pipe diameter + Bedding + 0.8 m.
- Otherwise: Pipe diameter + Bedding + 0.6 m.

Developing a PRP-algorithm require the input of relevant characteristics into the algorithm structure. Choosing the relevant characteristics are based on data availability and integrity (as discussed in Section 4), characteristic relevancy (as discussed in Section 2), logical structuring (as discussed in Section 3) and sensible result interpretation (as discussed in Section 5). Based on the criteria material, design life, diameter, pressure rating, static pressure and the residual pressure characteristics are included in the algorithm structure (as discussed in Section 3).

2.4 Pipe materials

Materials are the focal point of understanding pipe failures (Rodríguez et al., 2014). As this research has shown, there are characteristics of the material, the pipe itself, and both the internal and external systems characteristics, which can form favourable or non-favourable conditions in which a pipe performs. Each one of the conditions highlights a combination of factors, which as a result allocate to each pipe material advantages and disadvantages. According to Yna et al. (2013) the characteristic factors of pipe materials can be assessed, categorised and compared against each other to simulate specific, random or ultimate scenarios. Additional research is required to investigate all the different pipe materials and evaluate their history, advantages and disadvantages.

As the type and quality of pipe materials can vary all over the world, only materials commonly used on South African soil are investigated. According to GLS Consulting (2010) and Shand (2013), the pipe materials illustrated in Table 2.2 are predominant in the South African metropoles.

Table 2.2 Predominant pipe materials in South African metropoles.

Pipe Material	South Africa (GLS Consulting, 2010)	South Africa (Shand, 2013)
Asbestos cement	Yes	Yes
Glass Reinforced Polyester	Yes	Yes
Cast Iron	Yes	Yes
Ductile iron	Yes	Yes
Steel	Yes	Yes
Copper	Yes	No
High-Density Polyethylene	Yes	Yes
Modified Polyvinyl Chloride	Yes	Yes
Oriented Polyvinyl Chloride	Yes	Yes
Un-Plasticised Polyvinyl Chloride	Yes	Yes

A correlation is visible in that both Shand (2013) and GLS Consulting (2010) have found similar material types used in South Africa. Each pipe type has different properties and installation methods, each with their advantages and disadvantages. To be able to fully compare the materials identified as typically used in South Africa, each material requires thorough investigation.

2.4.1 Asbestos cement pipe materials

Asbestos cement (AC) pipes are an older pipe material, which became a common option for water reticulation pipes during the mid-1940s (Williams and Von Aspern, 2010). The materials of which AC pipes are made consist of Portland cement with a 12 % asbestos fibre component, and which also contains water and silica material elements. The AC pipes are formed under pressure and heat, while being cured in an autoclave.

AC pipes have excellent resistance to hydrogen sulphide corrosion, as well as low operating costs due to their low friction factors (Task Committee on Water Pipeline Condition Assessment, 2017). AC pipes were therefore popular during the 1940s-1970s. AC pipes are prevalent in the water reticulation infrastructure of communities or cities that experienced significant growth during that timeframe. Cities with an AC pipe manufacturing facility located nearby generally have a higher percentage of AC pipes than the national average.

Municipalities typically reported that the failure rates for AC pipes are significantly higher than those of any other pipe material (Punurai and Davis, 2017). The irony of the matter is that the predicted failures held against AC pipes seem to be due to the aggressive soil conditions, while these pipes were advertised as optimal for use in aggressive soil conditions. The failure rate of AC pipes increases dramatically with age, especially after their design life expectancy (Punurai and Davis, 2017).

AC is a widely used and easily manufactured water distribution pipe material, whose strength characteristics increase overtime and has the rigidity to support major portions of imposed loads under its own strength with flexible joints that allow for some deflection (Tsakiris and Tsakiris, 2012).

AC pipe material has the characteristic of being brittle, as a result of its tendency to degrade. Degradation depends on the associated water quality and soil condition it is exposed to. With a tendency to degrade, material corrosion occurs around joints, especially if they are not properly protected. AC pipe materials' tendency to corrode causes failures to occur as longitudinal splits, which are associated with general pipe deterioration and broken backs (Water Services Association of Australia, 2012).

AC pipe material is easy to manufacture and had always been perceived as an easy to handle material, but as technology developed and was taken into the twentieth-century, lighter materials became available on the market, which meant that people now regarded AC as difficult to handle (Water Services Association of Australia, 2012). According to Tsakiris and Tsakiris (2012) the main disadvantages associated with AC pipes are as follows:

- The danger of asbestos dust to human health.
- Susceptible to damage due to direct impact.
- Low beam-strength.
- Susceptible to corrosion by certain soils.
- Permeable in certain soil conditions.
- Difficult to locate.
- Complex repair.

2.4.2 Glass-reinforced polyester pipe materials

Glass reinforced polyester (GRP) is a composite water pipe material and has been used on South African soil, since 1992. GRP is a thermoset polymer pipe material, which consists of 70 % glass fibre and 30 % polyester resin, which adds to the pipe's ability to transfer loads, protect the glass fibres and ensure chemical resistance (Mostert, 2011). GRP is a recently developed pipe material, which has only been commonly used since the year 2000, especially in large bulk mains.

GRP is a complex material, which requires advanced manufacturing procedures and testing. Due to this characteristic, an independent inspectorate should be appointed to undertake the factory inspections during and after GRP pipe manufacturing (Flowtite Technology AS, 2014).

Thomas et al. (2014) states that GRP is a superior water pipe material with regards to its hydraulic performance, chemical resistance, UV resistance and corrosion resistance. In addition to the superior characteristics, the material is also associated with low maintenance-cost and easy installation, if correctly done. GRP is regarded as a cost-effective solution for water distribution systems as a pipe material. King et al. (1990) gives the additional advantages associated with GRP pipes as the following:

- Smaller wall thickness.
- Available in long lengths.
- Easy handling and installation with low mass, easy jointing, joint testing and transportation.
- Easy repair if maintenance is required.
- Design flexibility, with up to 2 m diameter and a 32 bar pressure class.
- Surge pressure allowance of + 40 % of nominal pressure.
- Material stiffness manufactured as semi-rigid, as well as flexible.
- Superior hydraulic characteristics.

Kruger (2013) says that GRP pipe material fails because of incorrect installation methods. GRP pipe failures are commonly associated with pipe-wall ruptures, due to pipe or tapping point damage, which occurs during construction procedures. The construction procedures, therefore, need to be monitored by experienced and trained supervision. Professionals need to oversee construction, which could prove challenging, and experts are expensive to come by, if local contractors, inexperienced with GRP, are used in isolated areas. Kruger (2013) gives the additional disadvantages associated with GRP pipes are as follows:

- Joining GRP with other materials poses a difficulty.
- Damage occurs easily in the presence of rock materials.
- Delamination can easily occur when not handled carefully.
- Pipe-ends can be damaged easily.
- Requires extensive testing.

2.4.3 Cast iron pipe materials

Cast iron (CI) is an alloy of iron with a carbon weight content higher than 2 %, as well as a percentage of silicon and manganese (Cassa, 2005). Due to the alloy content, CI tends to be brittle.

Samwel et al. (2012) notes that CI is an old water pipe material, which has been used in water distribution since the early 19th century, but its use has been dying out during the 20th century, when better materials and technologies started to make an appearance. According to Cassa (2005), the main advantages associated with CI are as follows:

- Low flammability.
- High rigidity, which eliminates deflection of pipe walls.
- Long design life.
- Low expansion coefficient.
- High wear resistance.
- High material hardness.

According to Kola (2010), CI is an old water pipe material, which usually fails by cracking and corroding as far as developing holes. Cassa (2005) gives the main disadvantages associated with CI as follows:

- Very brittle, which limits applications.
- High production cost.
- Poor corrosion resistance.
- High margin of error during joint installation.
- High conductivity.

2.4.4 Ductile iron pipe materials

Ductile iron (DI) is a ductile water pipe material, which consists of iron with a 2-4 % carbon content, and with an internal, and often external, polyethylene or cement mortar lining (Shand,2013). The material combination behaves similarly to steel.

DI pipes are usually externally protected with a metallic zinc coating, and a finishing layer of bitumen or synthetic resin that is compatible with the zinc. An additional external polyethylene coating can also be applied where soil characteristics tend to be aggressive, or when pipes are situated close to a live power line of more than 22 kV (Shand, 2013). The manufacturers do not recommend the use of cathodic protection; however, the Department of Water Affairs favours such applications, especially when pipelines are located near power lines and electrified railways. DI as a water pipe material has been used since the 1950’s (Rajani and Kleiner, 2003). DI material is commonly used for large diameter bulk and high-pressure water mains.

DI pipe material is high in strength, which does not decline with time, even if exposed to constant stress. DI pipes also have high ductility and can slightly deform without cracking (Shand, 2013).

According to Mostert (2011), the main advantages associated with DI pipes are as follows:

- Long life expectancy.
- High tensile and impact strength.
- High-pressure rating.
- High ring-bending strength.
- High beam-strength.

DI pipes are heavy and difficult to handle, which results in expensive installation costs, Mostert (2011). DI as a material has poor corrosion resistance with high conductivity and requires an additional internal and external coating or lining, including cathodic protection. The main disadvantages associated with DI pipes are as follows Mostert (2011):

- Not a readily available material and needs to be imported.
- Expensive and its affordability, and thus availability, depend on exchange rate fluctuations.

2.4.5 Steel pipe materials

Steel as a water pipe material, is a metallic material, which consists of an iron and carbon combination, as well as an admixture of manganese, phosphorus, sulphur and silicon. The presence of carbon and manganese contribute to material hardness and tensile strength. However, a high carbon content can cause the material to decrease in durability, toughness and weldability (American water works association, 2004). Mostert (2011) states that steel pipes need internal and external linings or coatings. Internal linings of steel pipes most often consist of centrifugally cast cement mortar. The external coatings consist often of coal tar with a glass fibre felt overwrap, and cement mortar, polyurethane or galvanising.

Steel was used as a water pipe material in water distribution systems and bulk mains in the early 1850s (American water works association, 2004). Since its introduction into water distribution systems, the material has only continued to develop and improve in quality.

Steel pipe material is high in strength, which does not decline with time if exposed to constant stress (Shand, 2013). Steel pipes also have high ductility and can slightly deform without cracking. According to Cassa (2005), the additional advantages associated with steel pipes are as follows:

- Lack of brittleness and resistance to shock.
- Long life expectancy.
- Flexibility and rigidity.

Steel pipes easily corrode if small defects occur in the lining, coating or corrosion protection. If such defects do occur, pitting and perforation are the most common types of failure to develop (Cassa, 2005). If extensive wall thinning develops, wall tearing or ductile ruptures are possible. Steel pipe fittings and joints also tend to corrode if coating proves to be inadequate. Steel materials are sensitive to conductivity characteristics and require protective measures and coatings (Mostert, 2011). Protective measures, and coating, are especially necessary if the pipeline is situated near power lines and or electrified railways, to prevent electromagnetic corrosion.

2.4.6 Copper pipe materials

Copper is an older natural metallic water pipe material that was used in water distribution networks since the late 1940s, after the end of World War II. Copper piping is commonly used for interior plumbing, rather than in water distribution networks.

According to Samwel et al. (2012), copper is favoured as a water pipe material because of its universality and the great number of advantages it possesses. Nesterchuk (2013) states the main advantages associated with copper pipes include high resistance to ultraviolet radiation and corrosion, bacterial growth resistance and the fact it is lightweight, flexible, durable and has a long life expectancy.

According to Nesterchuk (2013), copper as a water pipe material has an application limitation and is used mostly for internal plumbing or gas transportation, rather than in water distribution systems. Nesterchuk considers the main disadvantages associated with copper pipes to include high electrical and thermal conductivity, high cost, available only in small diameters, difficult installation and water quality issues if the water is excessively acidic or alkaline.

2.4.7 High-density polyethylene pipe materials

Three main types of polyethylene (PE) pipe materials exist, namely; high-density polyethylene (HDPE), medium density polyethylene (MDPE) and low-density polyethylene (LDPE) (Samwel et al., 2012). The level of density is an expression of the pressure each of the pipe materials can sustain. For that reason, HDPE is the most commonly used in water distribution systems.

HDPE water pipe material has been used since 1955 in water distribution networks, after being discovered in 1953. Only from 1990 however, was HDPE considered as a preferred pipe material (SAPPMA, 2013).

PE pipes are used over a broad range of applications, which include water distribution systems (SAPPMA, 2013). HDPE is not affected by corrosion or chemicals, with high impact strength and flexibility, which is lightweight and easy to handle. HDPE does require more advanced welding procedures, such as electrofusion, which removes the possibiliity of joint corrosion but does increase the pipe cost. According to SAPPMA (2013), the additional advantages associated with HDPE pipes are as follows:

- Biologically inert against microorganisms and is non-toxic.
- Low friction resistance to flow throughout its useful life.
- Resistance to the effects of ground movement.
- Low installation cost, easy to maintain and a wide range of available sizes.

HDPE water pipe material is commonly associated with failures related to butt-welded joints, electrofusion joints and fittings, which are weak and result in leakage (Samwel et al., 2012). The main disadvantages associated with HDPE pipes are the lack of UV resistance, that it is prone to sagging, stretching and shrinking (Mostert, 2011).

2.4.8 Polyvinyl chloride pipes pipe materials

Polyvinyl chloride (PVC) is a polymeric water pipe material, which was primarily developed pre-World War II and first used for water reticulation systems in the 1950s (Mostert, 2011). Since the 1950s, the material technology has kept on developing and PVC has been commonly used since 1984 as the preferred water reticulation network pipe material (Mulder and Knot, 2001). According to SAPPMA (2013), there are three main types of PVC, namely:

- Un-plasticised polyvinyl chloride (uPVC): Ridged PVC, which is known as the oldest PVC technology. uPVC consists of the first PVC polymer, without the plasticising agents that make PVC flexible.
- Modified polyvinyl chloride (mPVC): A newer PVC, in which the material's ductility and impact resistance have been improved. mPVC is essentially an alloy of the uPVC polymer, which contains several modifying agents that improve the ductility, as well as the impact resistance and crack growth. As a result of the improvements in material characteristics, thinner wall thicknesses, larger internal diameters and increased hydraulic efficiency are possible.
- Biaxially oriented polyvinyl chloride (oPVC): The latest type of PVC material, which consists of the same input materials as mPVC, but undergoes additional molecular orientation procedures, which converts the amorphous polymer structure to a more orientated ordered structure. Due to the ordered, structured orientation of the polymer, the material has more strength and a higher impact and crack resistance than those of its predecessors mPVC and uPVC.

According to Martins et al. (2009), PVC as a water pipe material is not affected by corrosion, and has excellent hydraulic characteristics. The main advantages associated with PVC pipes are as follows (SAPPMA, 2013):

- Resistance to abrasion and scouring.
- Impervious to chemicals found in sewage.
- Not damaged by modern cleaning methods.
- High impact resistance, toughness and durability.
- Lightweight and easy to install.
- Low maintenance and long life expectancy.
- High stiffness.
- High tensile and hoop strength.
- Excellent resistance to creep.
- Does not conduct electricity.

The strength of PVC water pipe material declines over time, when exposed to constant stress (Mostert, 2011). Conflicting PVC deterioration predictions do exist as Folkman (2014) reports on finding minimal deterioration of PVC and validates the long life thereof. PVC is a non-corrosive material, which makes use of special steel fittings in valves and air chamber. The fittings are often neglected and tend to corrode, which compromises the lifetime of the pipeline concerned and causes avoidable failures. More disadvantages do exist, according to Cassa (2005), which states that PVC, a water pipe material commonly associated with mechanical damage. According to Mostert (2011), the additional disadvantages associated with PVC pipes are as follows:

- Backfill is critical in the buried application.
- Maximum effective pipe size used for water distribution network is 500 mm diameter.
- The risk of backyard manufacturers providing sub-standard material.
- Thrust and anchoring blocks are required for installation.
- Prone to sag in supported applications.
- Low UV resistance.
- Prone to creep at steep slopes.

2.5 Prioritisation

A comprehensive understanding of different pipe materials, their relevant aspects, failure modes and system characteristics was gathered. The material comparison allows for full comprehension of the expected behaviour of any pipe and circumstances to which it will react. An understanding of effective assets replacement prioritisation was required.

There are a number of different ways to prioritise water distribution assets for replacement. Johnson (2015) mentions operative, condition-based, proactive or predictive approaches to prioritising.

The operative approach involves a 'find and fix'-approach where an asset is operated continuously throughout its complete useful life, which include operational inspections. The condition-based approach involves a 'find and fix'-approach when assets are approaching the time of failure, which also include operational inspections. The proactive approach involves replacing or rehabilitating an asset before there is a likelihood of failure by regular inspections and assessments of asset condition. The predictive approach involves considering all criteria that will minimise the asset's life-cycle cost by regular assessments of asset condition and projecting their future.

The proactive and predictive prioritisation approaches are both likelihood-based and risk-based approaches, which support decision-making for effective budget spending by prioritising asset replacement and maintenance schemes. According to Hopkinson et al. (2008), proactive and predictive prioritisation of risk is associated with the prioritisation of likelihood and impact assessments. The likelihood and impact assessments are prioritised by ranking the calculated risk, which highlights the items associated with both the high likelihood and high impact of failure. Determining the most significant likelihood and impact factors, in order to understand risks fully, is a complex matter.

In the context of a risk-based PRP-analysis, likelihood and risk both link to the occurrence of pipe failure events. Failure risks can be categorised into the following types (Hopkinson et al., 2008):

- Event risk – Uncertainty concerning an event.
- Variability risk – Uncertainty concerning the final value of an important variable.
- Systemic risk – Uncertainty concerning the combined effect of multiple interdependent factors.
- Ambiguity risk – Uncertainty concerning the underlying understanding, which can be interpreted in different ways.

Prioritising risk adhere to the following assessment procedure:

1. Identify all risks.
2. Develop risk assessment criteria.
3. Evaluate all risks.
4. Evaluate risk interactions.
5. Prioritise risk importance.
6. Implement risk response strategy.

Hopkinson et al.(2008) suggests that the following risk prioritising technique categories can be used for the assessment procedure above:

- Likelihood and impact modelling.
- Multi-attribute modelling.
- Quantitative modelling.

2.5.1 Prioritising risk using likelihood and impact-modelling techniques

The following risk prioritising techniques fall under the likelihood and impact-modelling category (Hopkinson et al., 2008):

- Probability and impact picture (PIP).
- Probability and impact matrix (PIM).
- Summary statistics of likelihood distributions: Expected value.
- Variance and standard deviation.

Dumbravă and Iacob (2013) states that the PIP is a prioritisation technique that offers a flexible format to view and understand the comparison between independent event risks, variability risks and uncertainty risks. Highlighting the risk event uncertainty and plotting the likelihood on the y-axis and the impact on the x-axis on a graph, to form rectangles and thereby achieve a comparison. The rectangles can be prioritised according to the sizes of the rectangular areas.

Ouabouch and Amri (2013) describes a PIM as a prioritisation technique that produces a relative ranking of risk events with the combined product of likelihood and impact, while expressing likelihood as a percentage of likelihood and the impact of one or more dimensions. The PIM matrix is ultimately used to calculate a likelihood-impact (P-I) score for each risk event, which prioritises against all other events. The risk events can also be plotted, for a graphical representation and better viewing and understanding of the prioritisation. The PIM approach can only be used to prioritise independent risk events, which excludes interdependence between risks (Dumbravă, 2013). The PIM prioritisation technique does not allow for the prioritisation of actions, which leads to inappropriate outcomes.

Samanez-Larkin et al. (2011) explains that the expected value prioritisation technique makes use of calculating and ordering each expected risk impact, by multiplying each possible impact by its associated likelihood and summing the results. The expected value approach gives a weighted average impact for each risk, which considers all possible estimated outcomes. For risks with potential impacts that are either adverse or beneficial, the expected impact might be regarded as a relatively simple method of comparing the levels of different sources of risk. By ranking, the calculated expected values prioritisation is achieved.

Drake and Semaan (2011) feels that comparing the expected values of risk event impacts does not give full consideration to the variability of the possible implications. The shortfall is in the expected value approach, addressed by using the risk variance and standard deviation, which is a better measuring tool for outcome uncertainty. By ranking the calculated variance or standard deviation of the likelihood density function, prioritisation is achieved.

2.5.2 Prioritising risk using multi-attribute modelling techniques

Hopkinson et al. (2008) states that the following risk prioritising techniques fall under the multi-attribute modelling category:

- Generalised multi-attribute risk prioritisation.
- Risk prioritisation charts.
- Uncertainty-importance matrix (UIM).
- High-level risk model.

Chang (2016) says the generalised multi-attribute risk prioritisation technique allows for several risk factors to be considered together, for prioritisation. The generalised multi-attribute risk approach can be used to prioritise for either qualitatively defined strategic risks, or specific quantitative risks, by omitting likelihood or replacing it with variability. The generalised multi-attribute risk prioritisation technique requires more effort and thought during the process of prioritisation than the standard PIM approach does. The technique demands more motivation for the specific risk factors used, which results in a more appropriate prioritisation.

Curtis and Carey (2012) discusses the risk prioritisation chart technique, which allows three different dimensions in a single graphical format. The first two dimensions are represented by likelihood and impact respectively, but the third dimension can show any one of a range of factors, such as urgency, response cost, manageability and propinquity. The risk prioritisation chart makes use of plotting both the likelihood and impact on the y-axis, respectively above and below the horizontal x-axis, while the third dimension is plotted on the x-axis (Hopkinson et al., 2008). A sensitivity threshold can also be added to highlight specific relevant values for prioritisation.

The UIM prioritisation technique focuses on the principle that all risks involve characterised uncertainty (Buchmeister et al., 2006). The characterised uncertainty with the greatest importance is allocated the highest priority of risk. This approach is most effective at the beginning of a project when significant numbers of uncertainties exist. By ranking the combinations with the highest levels of both uncertainty and importance, prioritisation is achieved.

Hopkinson et al. (2008) states that for certain needs a user might require a high-level risk model prioritisation approach, which uses information regarding generic risks to evaluate the relative risk exposure of specific projects or areas, as a prioritisation technique. Assigning a weight to risk sources and defining the scale of risk levels assessment are critical parts of the high-level risk process. While there are several different methods for implementing the above, approaches that are not rooted in collegiate experience are unlikely to produce reliable results.

2.5.3 Prioritising Risk using quantitative modelling techniques

The risk prioritisation methods that make use of quantitative modelling techniques are risk and uncertainty based methods that combine the risk consequences and their interdependence to calculate overall risk exposure (Hopkinson et al., 2008).

When prioritising risk with quantitative modelling techniques, the following objectives are essential: To

- gain insight into the importance and relevance of the risk factors involved in prioritisation.
- obtain accurate and unbiased forecast data for results and risk responses.
- gain insight into composite risks, for iterative risk management processes.

Quantitative risk modelling prioritisation techniques are most commonly used for project management applications, but have gained popularity in other applications due to their ability to initiate iterative risk management processes. For successful risk modelling, a holistic approach, where qualitative and quantitative techniques are integrated to develop flexibility into the technique, is required (Hopkinson et al., 2008).

The following risk prioritising techniques fall under the quantitative modelling category (Hopkinson et al., 2008):

- Simple quantitative models.
- Component risk within an activity or cost item.
- Schedule risk analysis (SRA).
- Net present value (NPV) risk model.
- Simple project risk re-estimating model developed form portfolio perspective.
- Monte Carlo output statistics.

Simple quantitative risk model prioritisation techniques provide a useful first-pass or specific risk approach to analysing and prioritising risk. The aim is to provide the minimum viable level of insight with the least effort, to direct the structure to a more complex risk model by displaying the component composite risks with layered cumulative likelihood distributions, in a manner that clarifies the structure of uncertainty components (Feather and Comford, 2003).

Simple type quantitative models provide a useful explanation of some risk principles and risk response prioritisations. However, simple quantitative models are often not the final product for quantitative modelling, but rather a pathway through to models with greater detail or complexity. The simple qualitative process can develop into a more detailed quantitative risk model by adding the following characteristics (Hopkinson et al., 2008):

- Using more complex likelihood density functions.
- Decomposing composite risks to a lower level of definition.
- Structuring models to support choices between different project responses.
- Using probabilistic branching to simulate mutually exclusive possibilities.
- Using conditional branching to simulate the effect of fall-back responses.
- Simulating the effects of feedback loops.
- Layering models to simulate compound risk effects.

Adding detail or complexity to a risk model should be based on prioritised items from previous models of the risk management process (Hopkinson et al., 2008). The most efficient risk management processes will focus on those aspects of risk that make the most difference. A key reason for starting a qualitative risk management process with the simplest possible model is to build sound structures iteratively, thus ensuring that overall risk calculation continues on a rational basis. Without such an approach, detailed risk models can appear to be plausible, despite being irrational and thus incorrect.

According to Jensen et al. (2012), prioritising risk using cost risk modelling was developed with the following objectives in mind: To

- provide an unbiased estimate of the financial contingencies.
- develop a better understanding of risks and improve the prioritisation of financial outcomes.

Hopkinson et al.(2008) says the cost risk model is a flexible structure, which allows different approach combinations to form an appropriate solution. The approach combinations can include:

- Consideration of the duration of the activity, defining a cost item by regarding component risks before building a cost model.
- Development of rectangular histograms with 10 to 20 classes to estimate the effects of each of the composite risks, which is responsible for the model outputs.
- Use of correlation to simulate the effect of underlying interdependencies between risk outcomes.
- The layering of risks to make sense from a dependency perspective.

The SRA is a prioritisation technique for analysing overall project risk (Vanhoucke, 2015). The technique can be used to develop a project strategy by defining objectives that promote realistic targets and identifying the level of contingencies needed to provide confidence in the outputs. Arrange the outputs according to their contribution towards the overall risk (Powell et al.,2016).

The NPV risk models prioritisation technique is appropriate for projects, which involve the associated cost or income over an extended period, and for which benefits can be compared with costs to determine the extent of their economic value (Žižlavský, 2014). Prioritisation is achieved with a sensitivity analysis, which involves starting with the overall risk forecast and then removing one risk at a time, for verification of prioritisation sensitivity.

The Simple project re-estimating model's development from a portfolio perspective approach for prioritisation uses the simple risk model approach. It then, furthermore, estimates the effects of risks on a major cost variable, which is common to all projects. The approach can assess many projects within the context of a single portfolio, by differentiating between sources of risk common to projects (Hopkinson et al., 2008).

The Monte Carlo simulation is the most commonly used process to operate more complex models and calculate the following properties (Cox and Siebert, 2006):

- Cruciality
- Criticality
- The sensitivity index of the schedule

Jones et al. (2006) considers that risks prioritisation consists of listing any of the above properties into descending order. Cruciality is the correlation between values simulated for any element in the model and model output. The cruciality may range from 1 to -1, where 0 indicates an outcome irrelevant to the model's output. Criticality is the percentage of simulation iterations (between 100 % and 0 %), of a risk on the critical path. A value of 0 % indicates that the risk does not affect the overall risk of the project. The criticality of a risk cannot exceed the likelihood of the risk's occurrence. The schedule sensitivity index (SSI) of any risk, in the schedule risk model, is calculated as follows:

$$SSI = \frac{Standard\ deviation\ for\ duration \times Criticality}{Standard\ deviation\ for\ the\ model\ output} \tag{2.3}$$

According to Cox and Siebert(2006), the standard deviations for the SSI calculation should be calculated from all the likelihood distributions, including zero duration values, where risk does not occur. The top-down approach to the development of quantitative risk models is accepted as good practice. The early passes in the approach of risk analysis are concerned with composite risks. As decomposition occurs, individual event risks, variability risks, ambiguity risks, and systematic risks become more important for appropriate prioritisation.

The first step to effective prioritisation is to understand the purpose of prioritisation at the current stage of the project and that the purpose can vary from one stage to another (Hopkinson et al., 2008). The second step is to understand all the elements of risk, not only the effects of individual risk events but also the sources of risk and the interdependence characteristics.

Developing a PRP-algorithm require a method for prioritising asset replacement. Choosing a method for prioritising asset replacement are based on the method adjustability, reliability, accuracy, simplicity and large data handling capacity. The prioritisation method also need to incorporate multiple characteristics, as well as serve as a tool that can be applied over a larger area.

Based on the criteria, a predictive approach with a simple quantitative technique are followed to prioritise asset replacement (as discussed in Section 2.5 and 2.5.3). The simple quantitative technique allow the algorithm to identify assets, which are sort listed for replacement and further prioritised through more complex techniques. The predictive simple quantitative prioritisation technique are included in the algorithm structure, as discussed in Section 3.

With an understanding of the different aspects and prioritisation techniques, an understanding of the existing pipe failure prediction approaches, which have been developed through previous research, is required.

2.6 Existing approaches to pipe failure prioritisation

The prediction of pipe failures for prioritisation of replacement or repair is a new concept, which has been receiving attention during only the past three decades. For South African water distribution network conditions, the concept has not yet been fully optimised or researched. Nevertheless, there are various approaches, each with its properties. The following are the main existing pipe failure prediction approaches (Martins, 2011):

- The Deterministic model.
- Stochastic models, such as the Poisson process model.
- The Weibull proportional hazard model.
- The Yule linear extended model.

2.6.1 The Deterministic model

Deterministic models of failure prediction have been used since the 1980s. These models were some of the first created to predict pipe failures, and which are still commonly used due to their simplicity (Martins, 2011). Palisade (2016) describes a deterministic model as a quantitative risk analysis process, which generates models by direct functions of explanatory variables, via single point estimates. According to Clair and Sinha (2012), the deterministic process model has the following disadvantages: It

- considers only a few outcomes.
- considers all outcomes with the same weighting, with no likelihood calculations.
- ignores interdependence between inputs, which tends to oversimplify the approach and reduce the accuracy of the model.

2.6.2 Stochastic model, such as the Poisson process model

The Poisson process model is a single variate stochastic failure prediction model, which makes use of a failure counting process that starts at zero with independent increments at a Poisson process counting rate (Watson et al., 2004). The Poisson process is based on historical failure data, on which estimates are made to determine a Poisson likelihood of failure. Failures can then be divided into the different pipe characteristics causing them, to determine influential failure predictions. For the Poisson process model, the expected number of failure events is proportional to time and pipe length, which is expressed as the number of failures/km/year. Martins (2011) believes the Poisson process model has the following disadvantages:

- A limited failure history will have a major effect on the results.
- All areas assessed need to have similar system characteristics, in order to generate accurate failure estimations.
- The main variables used as able to influence the failure rate, are limited to pipe material, pipe age, diameter and length.
- Splitting failure categories to smaller sizes can lead to non-significant failure rates.

According to Moglia et al. (2008), the Poisson process represents the behaviour of a straightforward and intuitive prediction model. The Poisson process model is easy to understand and easy to implement. The fact that categories, rather than covariates, define the Poisson process implies that every pipe in the same category has the same failure rate. To better differentiate between pipes, there should be a defined increase in the number of categories.

2.6.3 Weibull accelerated lifetime model

The Weibull accelerated lifetime model (WALM) is a multivariate failure prediction model, which differs from the Poisson process (single variate) in that the time between failures, and not the number of failures, is modelled. The accelerated lifetime model is expressed as the logarithm of time to failure, with a linear combination of covariates and an error term (Davis et al.,2008).

The Weibull distribution consists of simple survival and hazard functions, which simplifies the direct effect of covariates in the hazard functions (Debón et al., 2010). The Weibull distribution model is equivalent to the Cox proportional hazard model, where the covariates act multiplicatively in the hazard function. When conducting a survival analysis for a water distribution network, the time between failures is represented as *right censored*, which makes the time that all pipes have survived without failing, for the observation period, the *right censored time*, as the analysis did not finish with any failure observed. Therefore, inclusion of the right censored times into a likelihood function such as the Weibull survival function is essential. For each pipe, the distribution for the time between failures is estimated. The likelihood of the time surviving is also estimated.

According to Røstum (2000), the WALM has the following disadvantages:

- The convolution is not analytically obtainable and therefore the distribution of the number of failures per time is not derived.
- The number of failures prediction process can become more complex if some of the covariates are dynamic, which can cause the Monte Carlo simulation to enter a never-ending cycle.
- The pipe age covariate is updated only when a failure occurs, which implies that the pipe age will not influence the failure distribution on pipes that have not failed.
- The covariates used, such as length, diameter, age and previous failures, are limiting.

WALM fits the time between failures, rather than being a counting process that allows one to produce a new and different knowledge of pipes and their failures (Røstum, 2000). Differently estimated quantiles of the distribution could give useful information about pipes, such as the service life of pipe materials. However, quantiles of the estimated distribution are far from realistic. Weibull distributions present the disadvantage of not being analytically convoluted and it is therefore not possible to find the distribution of the general number of failures. Therefore, the expected number of failures needs a Monte Carlo simulation process. Nevertheless, the WALM simulation process can give good predictions with a high number of experiments. The WALM combines a great capacity of detecting those pipes with a higher likelihood of failing, which can be translated by the percentage of avoided failures using the model and accurate predictions. Prioritising the pipes more likely to fail among those with no failure history is a more difficult task, especially since the previous failures variable is one of the most significant covariates in the WALM regression.

2.6.4 Linear Extended Yule Process model

The Linear Extended Yule Process (LEYP) failure prediction model makes use of a pure birth (multivariate counting) Yule process. The LEYP process rate represents a linear function of an amount of past values, which are dependent on the pipe age and influenced by covariates. A linear extension of the model maintains a proportion between the process rate and the number of previous events (Le Gat, 2014). The LEYP contains the following important properties:

- The distribution of future individuals depends only on the number of current individuals (Markov property).
- At any time, there is at most one occurrence.
- The number of individuals follows a geometric distribution.

According to Claudio et al. (2014), when applying the pure birth process to failure data, the following boundary conditions are needed:

- The process will start at time zero, and the number of failures at installation will be zero.
- The process rate is free to vary with time.
- The intensity of the counting process is not proportional to the number of previous failures.

When conforming to the boundary conditions, the process takes on the form of a Non-Homogeneous Birth Process (NHBP). The likelihood function of the LEYP is developed through differentiation of the counting process likelihood function and by further induction. The expression of the distribution of the number of failures of a LEYP represents a continuous extended Negative Binomial, with the intensity function based on pipe covariates. For this process, likelihood parameters need to be estimated for the pipe failure likelihood calculation used in prioritisation (Claudio et al., 2014).

According to Le Gat (2014), due to the linear extension of the model, an increasing number of previous failures lead to a higher future failure rate. The relation between past number of failures and the future failure rate is not clear. Therefore, a non-homogeneous birth process, using other functions of the number of previous failures to describe the intensity of the process, could be studied. For instance, a limited function (continuous convergent function or a finite-valued function) could be a good solution. However, considering other functions can increase the complexity of the prediction model.

The nature of LEYP is probably the reason why the model presents a clear tendency to overestimate the number of future failures (Riisnes and Ugarelli, 2017). Nevertheless, LEYP presented an excellent performance when detecting pipes prone to failure. When applied to pipes with no recorded history of failure, the LEYP do not overestimate and the predictions are significantly more accurate.

2.6.5 Comparison of existing approaches to failure prediction

The WALM and LEYP procedures present significant advantages over the predictions of the Poisson process. The use of covariates allows a better understanding of the effects of pipe variables and does not require the division of the failure data, which could be a problem when dealing with small failure datasets (Martins, 2011). The Poisson process, however, has the advantage of being a simple method, easy to understand and to apply.

According to Martins (2011), WALM appeared to be the best of the three model because of its combination of accurate predictions with the ability to detect pipes the more prone to fail. Røstum (2000) however stated that the WALM method overestimates failure predictions and that the Poisson process appears to be a better method for predicting failures.

Martins (2011) regards the past failure variable as essential when predicting future failures. Pipes that have failed before present a higher likelihood of failing in the future. A pipe repair is not the same as a pipe network replacement; in general, a pipe becomes more fragile after a repair than before the failure happened. Another possible reason for failure is that other unknown factors influence the failure rate, such as environmental, traffic, operating pressure conditions or installation characteristics. A complete and reliable failure database is required.

All models require an organised information system with a complete inventory of all pipes being well characterised. Even if there is not an extensive pipe database with many variables, the application of the Poisson process makes useful predictions possible by using only three variables (Martins, 2011). Maintenance records over an extended period of time is not required. What is strictly necessary is to have a complete and up-to-date pipe inventory of all pipes and a reliable rehabilitation database, properly linked to the pipe inventory. The information system should be periodically reviewed to detect and correct possible inconsistencies.

The pipe failure approach presented in this study used various aspects of the WALM, LEYP and Poisson processes. The approach focuses on enabling areas without rehabilitation databases to be assessed together with areas that have reliable rehabilitation databases. The approach require the model to predict optimal failure circumstances, which are defined by the frequency that failure events are occurring at.

Developing a PRP-algorithm through predicting optimal failure circumstances, require an appropriate failure prediction model as input into the algorithm structure. Choosing an appropriate failure prediction model that illustrate the frequency at which failure events are occurring at, are based on model data availability, large data handling, accuracy, reliability, simplicity and sensible result interpretation (as discussed in Section 5). The failure predication model also need to put emphasis on replacement prioritisation, incorporate multiple characteristics, as well as serve as a tool that can be applied over a larger area.

Based on the criteria, the foundation of a Poisson process failure prediction model are followed as input into the algorithm structure (as discussed in Section 2.6.2) to determine the frequency at which failure events are occurring. According to the Poisson process model the frequency of occurrence of failure events is determined and expressed by the number of failures over a specified time period per pipe length in the study area. The predictive simple quantitative prioritisation technique are included in the algorithm structure, as discussed in Section 3.

3. ALGORITHM DEVELOPMENT

3.1 Overview

As mentioned in Section 2.6.5, the development of a PRP-algorithm is based on reliable, verified datasets and the factors necessary to generate accurate prioritisation results. The datasets are used in the following order, to develop the prioritisation algorithm:

- Acquire data.
- Restructure the data, as necessary to perform the analysis.
- Develop the algorithm.
- Apply the algorithm to a case study with high integrity data.
- Verify prioritisation results with existing failure data in the case study site.
- Implement the calibrated algorithm over the entire case study site for prioritisation.

The main prioritisation algorithm objectives should be kept in mind; namely, to ensure that a system assessment includes areas both with and without failure data, which were evaluated on the same specified principles.

3.2 Acquire accurate and reliable data

Some general foundation datasets are required for the PRP-algorithm, including a hydraulic computer model dataset and a failure dataset. Both these datasets need complete data, correct data, no duplicate data and reliable data integrity.

Once the finalised hydraulic computer model and a failure dataset are regarded as reliable, a prioritisation algorithm can be developed and implemented with confidence that the final product is of the highest integrity.

3.3 Conceptual description of algorithm development

One of the main objectives of a PRP-algorithm is to assess a system without failure data, but based on conditions likely to cause failures, which have been derived from existing systems with failure data. To understand the failure conditions, a comprehensive understanding of the life cycle of a pipe is required. Based on the research done, a water system consists of two main tiers namely, physical characteristics (as discussed in Section 2.3.5 and 2.3.6) and system characteristics (as mentioned in Section 2.3.6), as shown in Figure 3.1.

The two tiers are unique, because the first describes the system without water while the second tier involves hydraulic modelling (behaviour of pipes filled with water). The unique aspects are grouped on what is termed the Level 1 characteristic category. Both tiers are expressed through a combination of their relevant characteristics, as shown in Figure 3.1.The two tiers are dependent on each other; to such a degree that one determines how the other reacts to failure conditions. Therefore, the second and third level characteristic categories are introduced as interdependent, as shown in Figure 3.1. Both interdependent levels comprise of unique characteristics, which is relevant to the degree of their interdependence.

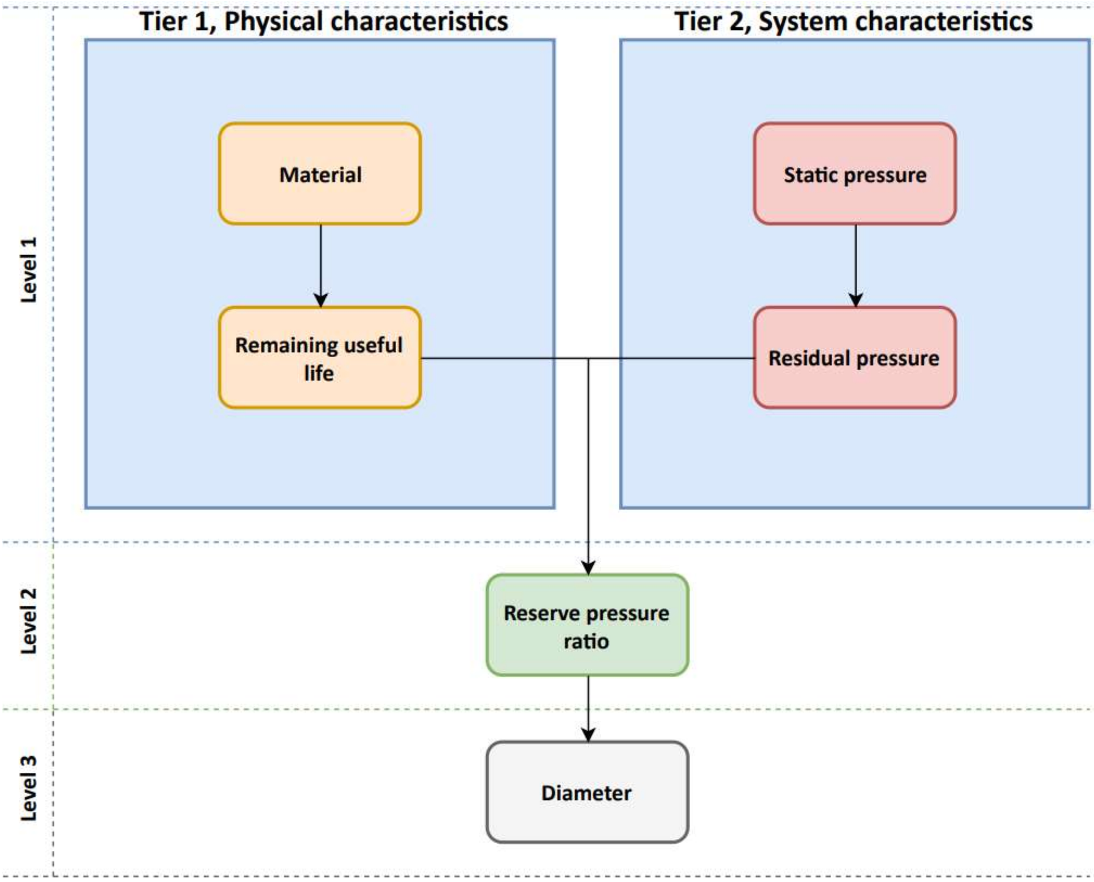

Figure 3.1 Schematic of the two-tier structure, with three levels of characteristics.

The unique physical, system, and interdependent characteristics, for each interdependence level (as shown in Figure 3.1), were expressed as a water pipe failure frequency, as discussed in Section 2.6.5. The pipe failure frequency is expressed in various ways, depending on the available data.

According to Rathnayaka (2016), the standard practice for expressing pipe failure frequency is failures/year/meter, which is similar to the Poisson process (as discussed in Section 2.6.2 and 2.6.5). The interpretation of failure frequency as failures/year/meter was adopted from the Poisson process and used in this study.

3.3.1 Level 1 - Tier 1, physical characteristics

The physical characteristics of the pipe were evaluated by pipe material, as shown in Figure 3.1. The material aspect was selected as the main Tier 1 physical characteristic, as it forms the foundation of the water distribution infrastructure and reflects a strong correlation with pipe failures. The material categories mentioned in Section 2.4 were used as the main assessment criteria (an excerpt of the procedure in the source code is presented in Appendix A).

With pipe material as the main physical characteristic, further differentiation was made to include remaining useful life (RUL), as shown in Figure 3.1. The material's RUL is an important physical system characteristic as it directly relates to installation year and material design life, which are regarded as main asset management components, as discussed in Section 2.3.5. The material's RUL gives insight into a water distribution pipe's life cycle and is highly correlated to material condition and failure occurrences. RUL was calculated for each pipe as explained in Equation 3.1 (GLS Consulting, 2010) (an excerpt of the procedure in the source code is presented in Appendix B and H):

$$RUL = YDL - (YCM - YI) \tag{3.1}$$

$Remaining\ Useful\ Life\ (Year) = RUL$

$Current\ Model\ Year\ (Year) = YCM$

$Installation\ Year\ (Year) = YI$

$Material\ Design\ Life\ (Year) = YDL$

When assessing the failure frequency of materials and RUL (as discussed in Section 3.3), the sum of the two factors, multiplied by the physical characteristic index weight represents the failure frequency index values for the Level 1, Tier 1 physical characteristics. The index weight introduced in Equation 3.2 to Equation 3.13, serves the purpose of making the PRP-algorithm more flexible. Flexibility was added by allowing the user to assign different weightings to contributing characteristics according to the algorithm definition and the user needs. The index weight also serves as a calibration tool once the analysis enters the verification process.

The physical characteristic failure frequency index (referred to as Level 1, Tier 1 failure frequency index) for each pipe, was calculated as explained in Equation 3.2:

$$I_{L1T1f} = (P_{MATff} + P_{RULff}) \times W_{L1T1} \tag{3.2}$$

$Material\ Failure\ Frequency = P_{MATff}$

$RUL\ Failure\ Frequency = P_{RULff}$

$Level\ 1\ Tier\ 1\ Index\ Weight = W_{L1T1}$

$Level\ 1\ Tier\ 1\ Failure\ Frequency\ Index = I_{L1T1f}$

3.3.2 Level 1 - Tier 2, system characteristics

The system characteristics were evaluated by hydraulic conditions. The hydraulic conditions were coordinated and categorised by pressure, which was then subdivided into static pressure and residual pressure.

As mentioned in Section 2.3.6, static pressure is the maximum pressure a pipe might be exposed to and is a direct representation of the relevant pressure zone. The static pressure is regarded as the main system characteristic (as shown in Figure 3.1) for the condition assessment, as it is directly related to the quantity of water lost through leaks and the minimum night flow conditions (which is the reason for pressure management schemes being implemented). For the evaluation of static pressure, the average static pressure over a single pipe was considered. Each pipe, in the hydraulic computer model, consists of a begin and an end node, over which the average static pressure was calculated as explained in Equation 3.3, as advised by Sinske (2017) (an excerpt of the procedure in the source code is presented in Appendix C and H):

$$SP_{AVG} = \frac{SP_{BN}+SP_{EN}}{2} \tag{3.3}$$

$Static\ Pressure\ Begin\ Node\ (m) = SP_{BN}$

$Static\ Pressure\ End\ Node\ (m) = SP_{EN}$

$Average\ Static\ Pressure\ (m) = SP_{AVG}$

With static pressure defined as the main system characteristic, further differentiation was made to include residual pressure. Residual pressure is an indication of pressure in a system with a water demand, as mentioned in Section 2.3.6. The average residual pressure over a single pipe was calculated in the same manner as average static pressure. Residual pressure was added to Tier 2 as a system characteristic (shown in Figure 3.1), as it is an indication of possible stress changes in and on the pipe, which can represent movement on joints, where most leaks occur. Due to the changes in stress, residual pressure can also be directly related to the degeneration rate of the pipe material and, ultimately, to pipe failure. The residual pressure, over a single pipe, was calculated as explained in Equation 3.4, as advised by Sinske (2017) (an excerpt of the procedure in the source code is presented in Appendix D and H):

$$RP_{AVG} = \frac{RP_{BN}+RP_{EN}}{2} \tag{3.4}$$

$Residual\ Pressure\ Begin\ Node\ (m) = RP_{BN}$

$Residual\ Pressure\ End\ Node\ (m) = RP_{EN}$

$Average\ Residual\ Pressure\ (m) = RP_{AVG}$

When assessing the failure frequency for static pressure and residual pressure (as discussed in Section 3.3), the sum of the two factors multiplied by the system characteristic index weight (as discussed in Section 3.3.1), represents the failure frequency index values for Level 1, Tier 2 system characteristics. From the system characteristic failure frequency index, referred to as Level 1, Tier 2 failure frequency index, for each pipe, was calculated as explained in Equation 3.5:

$$I_{L1T2ff} = (P_{ASPff} + P_{ARPff}) \times W_{L1T2} \tag{3.5}$$

$Average\ Static\ Pressure\ Failure\ Frequency = P_{ASPff}$

$Average\ Residual\ Pressure\ Failure\ Frequency = P_{ARPff}$

$Level\ 1\ Tier\ 2\ Index\ Weight = W_{L1T2}$

$Level\ 1\ Tier\ 2\ Failure\ Frequency\ Index = I_{L1T2ff}$

After both Tier 1 and Tier 2 failure frequency index values had been calculated (Equations 3.2 and 3.5), the results were combined and summed to form the Level 1 total failure frequency index for the prioritisation of replacement likelihood, as explained in Equation 3.6. The representation of Level 1 prioritisation is the first replacement likelihood of the total water distribution network calculated for the algorithm. The Level 1 failure frequency index was calculated as explained in Equation 3.6:

$$I_{L1ff} = I_{L1T1f} + I_{L1T2ff} \tag{3.6}$$

$Level\ 1\ Tier\ 1\ Failure\ Frequency\ Index = I_{L1T1ff}$

$Level\ 1\ Tier2\ Failure\ Frequency\ Index = I_{L1T2ff}$

$Level\ 1\ Failure\ Frequency\ Index = I_{L1ff}$

3.3.3 Level 2 and 3, interdependence characteristics

As seen in Figure 3.1, the Tier 1 and Tier 2 characteristics become integrated on Level 2 and Level 3, which are referred to as the interdependence characteristics and form the next phase of prioritisation. The interdependence characteristic on Level 2 are reserve pressure ratio.

Reserve pressure ratio is expressed as the average static pressure, as calculated in Equation 3.3, divided by the pipe material's pressure rating (GLS Consulting, 2015). The reserve pressure ratio is, therefore a representation of the interdependence between the physical pipe material pressure rating characteristics (Tier 1) and the system characteristics of average static pressure (Tier 2). Reserve pressure ratio is a well-balanced interdependence factor, which represents the available pipe pressure capacity during an operation scenario. The reserve pressure ratio gives insight into the pipes' available pressure capacity, which, if exceeded under certain system circumstances can cause physical pipe failures. Reserve pressure ratio was used to represent the interdependence of Level 2 prioritisation as the only contributing characteristic, for this study. The reserve pressure ratio in each pipe was calculated as explained in Equation 3.7 (GLS Consulting, 2015) (an excerpt of the procedure in the source code is presented in Appendix E and H):

$$RPR = \frac{SP_{AVG}}{PR} \tag{3.7}$$

$Reserve\ Pressure\ Ratio = RPR$

$Average\ Static\ Pressure\ (m) = SP_{AVG}$

$Material\ Pressure\ Rating\ (m) = PR$

When assessing the failure frequency for reserve pressure ratio (Level 2), the Level 2 index weight (as discussed in Section 3.3.1) multiplied by the reserve pressure ratio failure frequency, represents the failure frequency index values for the Level 2 interdependence characteristic. The Level 2 failure frequency index, for each pipe, was calculated as explained in Equation 3.8. The Level 1 and Level 2 failure frequency index values were added, to represent the prioritisation of the second likelihood of replacement in the total water distribution network, calculated for the algorithm by method of ranking index values. The Level 2 total failure frequency index for the prioritisation of replacement likelihood was calculated as explained in Equation 3.9:

$$I_{L2ff} = P_{RPRff} \times W_{L2} \tag{3.8}$$

$$I_{TL2ff} = I_{L1ff} + I_{L2ff} \tag{3.9}$$

$Reserve\ Pressure\ Ratio\ Failure\ Frequency = P_{RPRff}$

$Level\ 2\ Index\ Weight = W_{L2}$

$Level\ 2\ Failure\ Frequency\ Index = I_{L2ff}$

$Level\ 2\ Total\ Failure\ Frequency\ Index = I_{TL2ff}$

With Level 2 prioritisation defined, Level 3 prioritisation requires a further interdependent relationship. The pipe diameter was identified as the Level 3 prioritisation interdependent factor, as shown in Figure 3.1.

The pipe diameter has several interdependent relationships, such as with the material (Tier 1, Level 1), residual pressure (Tier 2, Level 1) regarding hydraulic capacity, and reserve pressure ratio (Level 2) regarding material pressure rating (a direct correlation to pipe wall thickness, as discussed in Section 2.3.5). Diameter is dependent on material, as certain diameters are available for certain materials, which influences roughness coefficients, headloss and, most importantly, handling and installation techniques. As a wide range of aspects can go wrong during installation, and which can ultimately lead to pipe failure, diameter and the interdependent relationship with the material is the only correlation indication used between failure frequency and possible construction malfunctioning trends. Pipe diameter forms part of the minimum information required to create any hydraulic computer model and is regarded as widely available information. For the research study, diameter is categorised for every 100 mm as shown in Table 3.1 (an excerpt of the procedure in the source code is presented in Appendix F).

Table 3.1 Selected diameter ranges.

ID	Small diameter ranges (mm)	ID	Large diameter ranges (mm)
1	0-100	8	500-600
2	100-200	9	600-700
3	200-300	10	700-800
4	300-400	11	800-900
5	400-500	12	900-1000

When assessing the failure frequency for diameter (Level 3), the Level 3 index weight (as discussed in Section 3.3.1) multiplied by the diameter failure frequency, represents the failure frequency index values for Level 3 interdependence characteristics. The Level 3 failure frequency index, for each pipe, was calculated as explained in Equation 3.10:

$$I_{L3ff} = P_{Dff} \times W_{L3} \tag{3.10}$$

$Diameter\ Failure\ Frequency = P_{Dff}$

$Level\ 3\ Index\ Weight = W_{L3}$

$Level\ 3\ Failure\ Frequency\ Index = I_{L3ff}$

With all three levels of failure frequency index values calculated, the sum of all 3 levels represents the final failure frequency index values to be used for the prioritisation of pipe replacement likelihood. The final failure frequency index values, were calculated as explained in Equations 3.11, (an excerpt of the procedure in the source code is presented in Appendix G):

$$I_{Fff} = I_{L1ff} + I_{L2ff} + I_{L3ff} \tag{3.11}$$

$Final\ Failure\ Frequency\ Index = I_{Fff}$

The final water reticulation PRP of the final failure frequency index values was achieved by ranking the likelihood index values from high to low.

3.4 Developing the algorithm and performing the analysis

To develop a two-tier algorithm, with three levels of interdependence, for the prioiritsation of replacement of water reticulation pipes, an understanding of various aspects were required (as discussed in Section 2). Firstly an understanding was required to calculate an index of the likelihood of pipe replacement (as discussed in Section 2), which was based on calculated failure frequencies of water reticulation pipes. Secondly an understanding was required in calculating pipe failure frequency (as discussed in Section 3.3), which was calculated in the same manner for all the contributing characteristics (as discussed in Section 3.3) by assigning a failure frequency to corresponding grouped ranges. The failure frequency ranges grouped per contributing characteristic were developed and implemented by integrating the failure dataset with the corresponding water-base hydraulic computer model in the following manner:

A failure dataset was acquired, which geographically linked the relevant pipe (and its contributing characteristic data) to the failure event. The failure dataset was then integrated with the hydraulic computer model, where the number of recorded failure events was assigned to each pipe. Thereafter, a study area was consequently selected that contained high integrity data, for both the failure dataset and the hydraulic computer model. From the newly developed database in the selected study area, the x-axis for each contributing characteristic was categorised into grouped ranges, which ensured that the entire data spectrum related to the relevant contributing characteristic was covered.

The failure frequency for each grouped range of the contributing characteristic was subsequently calculated, as explained in Equation 3.12 (an excerpt of the procedure in the source code is presented in Appendix A to F):

$$FF_i = F_i/T_f/L_i \tag{3.12}$$

$i\ Represent\ a\ single\ x\ axis\ grouped\ range\ in\ a\ spectrum\ for\ a\ contributing\ characteristic$

$Characteristic\ Range\ i\ Total\ Pipe\ Length\ (m) = L_i$

$Characterisic\ Range\ i\ Failure\ Count\ = F_i$

$Failure\ Dataset\ Timespan\ (Year)\ = T_f$

$Characteristic\ Range\ i\ Failure\ Frequency\ (Failure\ Count/Year/Meter) = FF_i$

The calculated failure frequencies were plotted against the contributing characteristic in x-axis ranges. The plotted failure frequency versus contributing characteristic graphs were then evaluated and inspected to ensure that the data was conclusive enough to satisfy sufficient and logical interpretation. If the graphs were not sensible, further data inspection was required to ensure that the data used was of the highest possible integrity. When the graphs were deemed conclusive, the corresponding failure frequencies (for the specific contributing characteristic) were assigned to every single pipe in the water reticulation network, according to the grouped range it correlated with. The failure frequency graphs ultimately describe the pipe's characteristic behaviour under failure conditions within the water distribution network. Once all contributing characteristics, for all pipes in the water reticulation network had been populated (with failure frequency), the corresponding failure frequency index values were calculated as per the predictive approach through the simple quantitative technique (as discussed in Section 2.5.3). Next the final failure frequency index values were calculated, which included the sum of all three interdependence levels of index values for prioritisation, as described in Section 3.3 and explained in Equation 3.13 (an excerpt of the procedure in the source code is presented in Appendix G):

$$\begin{aligned} I_{Fff} \ = \ & (P_{MATff} + P_{RULff}) \times W_{L1T1} \\ & + (P_{ASPff} \times P_{ARPff}) \times W_{L1T2} \\ & + P_{RPRff} \times W_{L2} \\ & + P_{Dff} \times W_{L3} \end{aligned} \tag{3.13}$$

$Material\ Failure\ Frequency = P_{MATff}$

$RUL\ Failure\ Frequency = P_{RULff}$

$Average\ Static\ Pressure\ Failure\ Frequency = P_{ASPff}$

$Average\ Residual\ Pressure\ Failure\ Frequency = P_{ARPff}$

$Reserve\ Pressure\ Ratio\ Failure\ Frequency\ = P_{RPRff}$

$Diameter\ Failure\ Frequency\ = P_{Dff}$

$Level\ 1\ Tier\ 1\ Index\ Weight = W_{L1T1}$

$Level\ 1\ Tier\ 2\ Index\ Weight = W_{L1T2}$

$Level\ 2\ Index\ Weight = W_{L2}$

$Level\ 3\ Index\ Weight = W_{L3}$

$Final\ Failure\ Frequency\ Index = I_{Fff}$

The index weight introduced in Equation 3.13 and Section 3.3.1, ultimately served as a calibration tool for the PRP-algorithm once the analysis entered the verification process. For the research case study no weighting adjustments were implemented and an average weighting of 0.167 was applied to each of the six described contributing characteristic index weightings, throughout the whole analysis and verification process. The final PRP-results for each pipe were grouped into water distribution zones and each zone was prioritised and ranked in order to compare the PRP-results to the prioritised verification failure frequency results. The verification failure frequency results were calculated as the failure count per year per length of pipe for the distribution zones.

The PRP-algorithm was set up on three interdependence levels. The three levels of interdependence were included mainly to give the user additional prioritisation flexibility, which therefore allowed for the elimination of undesired pipes (by characteristics) after each interdependent level's frequency of failure final index values were calculated. The elimination of undesired pipes was guided by the user's needs, determined by the size of budget allocated towards pipe replacement, operational requirements, or to satisfy certain prioritisation needs.

The PRP-procedure started by prioritising Level 1, and eliminating areas or pipes that had low likelihood of failure index values from the algorithm, to adhere to the user needs. Level 2 index values were added to the already prioritised Level 1 index values and once again prioritised with a second elimination process. The analysis continued with Level 3 prioritisation, by adding Level 3 index values to Level 1 and Level 2 prioritised failure likelihood index values (as discussed in Section 3.3). From the final sum of interdependence level index values, the final pipe failure likelihood index values were calculated, ranked and prioritised accordingly. For the research case study no elimination was done, as the elimination for prioritisation was regarded as a user-specific algorithm functionality (an excerpt of the procedure in the source code is presented in Appendix G).

3.5 Verify results with existing data

After generating the first cycle of failure frequency index results in the study area with high integrity data for the PRP-algorithm, the final failure frequency index values per pipe were used, as explained in Equation 3.14 to calculate a index failure count per single pipe.

$$F_{j\_I} = I_{j\_Fff} \times L_j \times T_f \quad (3.14)$$

$j\ Represent\ a\ single\ pipe\ defined\ by\ a\ begin\ and\ end\ node$

$Pipe\ Length\ (m) = L_j$

$Pipe\ Index\ Failure\ Count\ = F_{j\_I}$

$Failure\ Dataset\ Timespan\ (Year)\ = T_f$

$Final\ Failure\ Frequency\ Index\ (Failure\ Count/Year/Meter) = I_{j\_Fff}$

Once an index failure count per single pipe had been calculated the single pipes were grouped into sensible areas (suburbs, wards, regions or discrete water distribution pressure zones). For each sensibly grouped area a new overall failure frequency was calculated in the same manner as explained in Equation 3.12, in Section 3.4, by using the index failure count. After the new overall failure frequencies had been calculated, the results were sorted from high to low and assigned a rank for prioritisation. For the study, water reticulation pipes were grouped according to their respective discrete water distribution pressure zones (discussed in Section 2.3.6).

For the verification and calibration processes of the algorithm results, a second set of failure frequency results (failure frequency verification results) was generated for the study area. The failure frequency verification results were calculated for each of the sensibly grouped areas using their associated total historical failure counts and pipe length. The failure frequency verification results were expressed as the number of failures over a fixed time per pipe length for each of the sensibly grouped areas, which were sorted from high to low and assigned a rank for replacement prioritisation.

Once the verification prioritisation results had been generated, a verification process was initiated against the algorithm results, by comparing both ranked prioritisation results of sensibly grouped areas (sorted by prioritisation rank of algorithm results) in a tabular manner through visual inspection. By using visual inspection, it was easy to identify whether some prioritisation correlation existed between the two sets of results. If a correlation did not exist through visual inspection, the algorithm analysis was redone.

During the second analysis, the interdependence level characteristics x-axis grouped ranges were refined, which ultimately increases the algorithm sensitivity and generates more accurate failure frequency index values for prioritisation. Additionally, the index weightings could be adjusted to generate correlated results. The process was iterated until the desired visual result verification correlation was achieved between the two result sets. Once a desired visual prioritisation result correlation had been achieved, a correlation percentage was calculated for the top ten and top twenty prioritisation results.

The correlation percentages was developed by sorting the sensible grouped areas according to their verification prioritisation rank. The rankings of the top ten prioritised areas for the verification results were added together to calculate the sum of the verification result and in the same manner the sum of the associated algorithm result were calculated. The correlation percentage for the top ten ranks was calculated as the sum of the algorithm results, divided by the sum for the verification results and expressed as a percentage. The same method was followed to calculate the correlation percentage for the top twenty ranks as explained in Equation 3.15 and Equation 3.16 (discussed in Section 5.5).

$$CP_{10} = \frac{SA_{10}}{SV_{10}} \times 100 \tag{3.15}$$

$$CP_{20} = \frac{SA_{20}}{SV_{20}} \times 100 \tag{3.16}$$

$Sum\ of\ Verification\ Result\ Set\ Top\ 10\ Ranks\ per\ Algorithm\ Grouped\ Areas = SV_{10}$

$Sum\ of\ Verification\ Result\ Set\ Top\ 20\ Ranks\ per\ Algorithm\ Grouped\ Areas = SV_{20}$

$Sum\ of\ Algorithm\ Result\ Set\ Top\ 10\ Ranks\ per\ Algorithm\ Grouped\ Areas = SA_{10}$

$Sum\ of\ Algorithm\ Result\ Set\ Top\ 20\ Ranks\ per\ Algorithm\ Grouped\ Areas = SA_{20}$

$Top\ 10\ Correlation\ Pecentage\ (\%) = CP_{10}$

$Top\ 20\ Correlation\ Pecentage\ (\%) = CP_{20}$

Determining the criteria of the correlation percentage was researched, which highlighted the complexity of developing the criteria and its sensitivity. Simundic (2012) states that developing the correlation percentage criteria and determining the sensitivity thereof greatly consist of applying one’s mind regarding the relationship between criteria sensitivity and analysis result accuracy requirements.

A correlation percentage of greater than or equal to 80 % was identified to satisfy calibration needs. As part of further work, the development and interpretation of the correlation percentage criteria and its sensitivity can expanded.

4. CASE STUDY DATA COLLECTION

As part of this study, the PRP-algorithm was implemented on a full-scale case study that comprised of the water reticulation zones south of the Magaliesberg as far as the Constantia Park tower zone, within the City of Tshwane Municipality boundaries (South Africa). The datasets required, with which to conduct the PRP-analysis, consisted of failure and repair logs, or closed-circuit television (CCTV) data, as well as a hydraulic computer model of the water distribution network. The failure and CCTV data represented the system's failure symptoms. The hydraulic computer model of the water distribution network contained all relevant physical pipe data and system data, which were also required for the analysis. The different databases were integrated systematically, as illustrated in Figure 4.1. The datasets were used to populate the model and implement the PRP-algorithm.

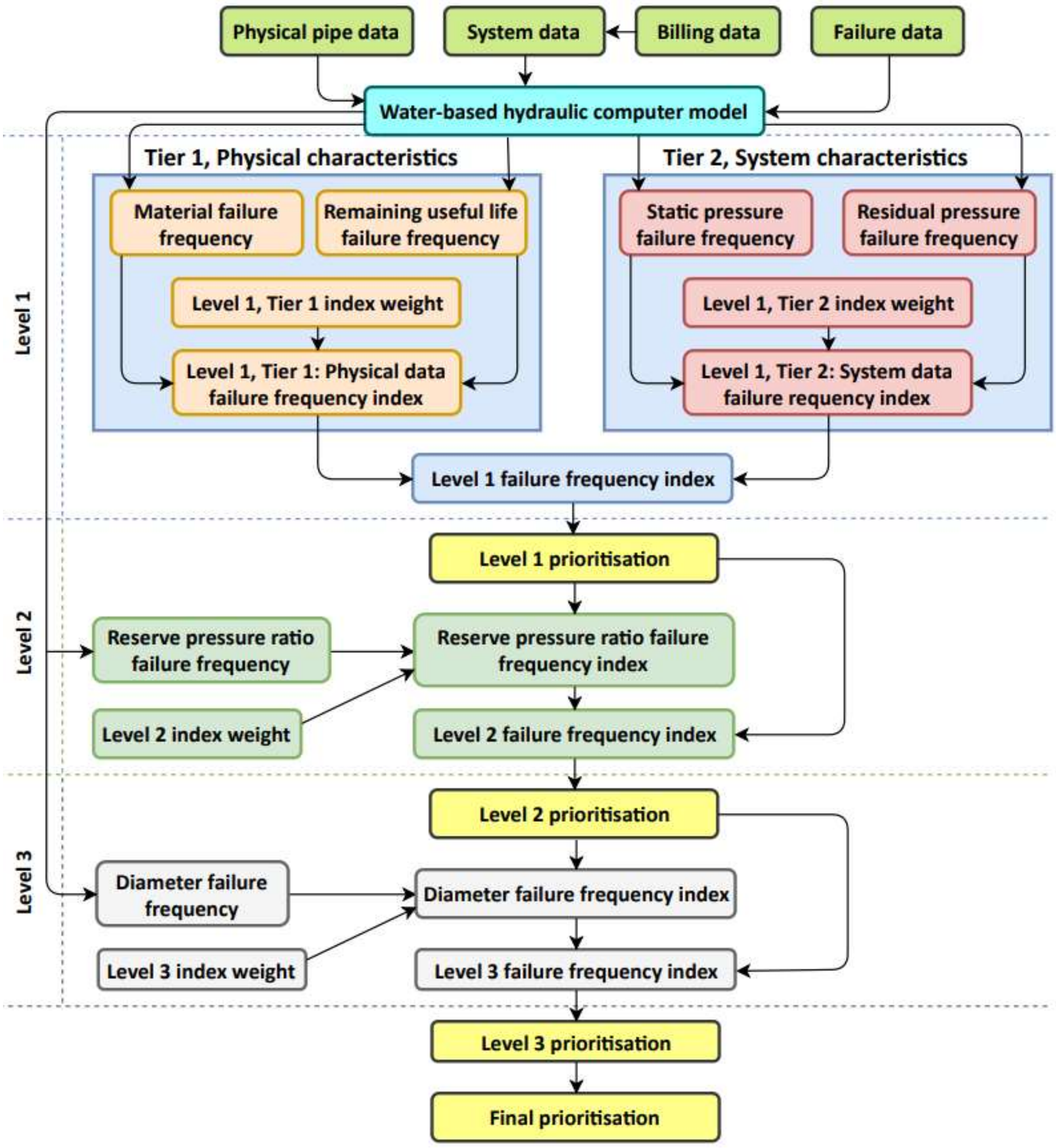

Figure 4.1 Complete data integration process illustration.

4.1 Hydraulic computer model

A hydraulic computer model of a water distribution system typically consists of variously integrated datasets. The integrated datasets are categorised into Tier 1 (Physical pipe data) and Tier 2 (System data). An excerpt from the hydraulic computer model pipe data for the case study model, is presented in Appendix H. According to Loubser (2017) the schematic dataset integration, for a hydraulic computer model, is illustrated in Figure 4.2.

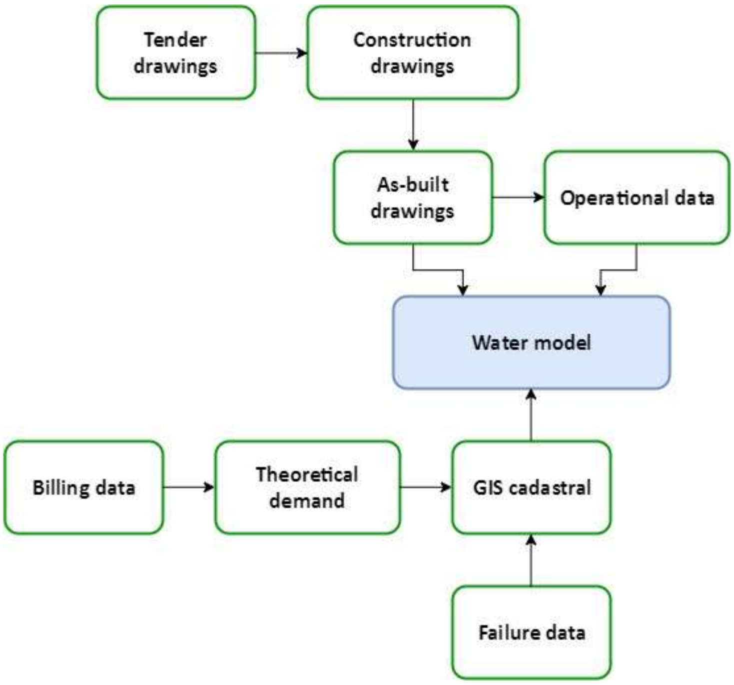

Figure 4.2 Hydraulic model development process.

4.1.1 Tier 1, Physical pipe data

The physical pipe data (from Tier 1) relate to the hydraulic computer model data, captured from 'as-built' drawings and operational data. The as-built drawings and pipe schedules represent the plan of installation and contain the following pipeline information:

- Spatial description, such as coordinates.
- Pipe length - derived from spatial information and construction drawings.
- Pipe diameter.
- Pipe material and pressure class.
- Protection coatings.
- Inlet and outlet connections.
- Pipe joint details.
- Location and type of special fittings such as valves, water meters and system controls.

A comprehensive discussion of the different pipe materials, and the related characteristics was presented in Section 2. Drawing from the knowledge review, a comparison of the pipe material's mechanical and physical properties was made, as shown in Table 4.1.

Table 4.1 Comparison of mechanical and physical properties of pipe materials.

Characteristics	AC	Copper	GRP	CI	DI	Steel	PVC	HDPE	Units
Pipe									
Diameter	50-1000	6-305	25-4000	100-1000	100-1100	300-2500	16-630	16-630	mm
Design life	30-50	40-60	50-60	70-100	70-100	50-70	40-60	60-90	Years
Pressure rating	600-1800	3000-8800	600-3200	3000-6400	3000-6400	1380-24550	600-2500	600-2500	kPa
Mechanical									
Density	1950	8968	1850	7800	7300	7850	1300	958	kg/m^3
Vicat softening point	-	>650	>650	>650	>650	>650	76	67	°C
Thermal conductivity	2.07	385	0	80	79.5	50.2	0.16	0.46	W/(m.K)
Physical									
Hardness	-	70	-	34	33	-	80	63	Shore D
Tensile Yield	17	70	66	130	379	380	25	33	MPa
Ultimate Yield	33	220	343	200	552	414	52	37	MPa
Ultimate Elongation	-	20	1	-	18	22	75	150	%
Elastic Modulus	17	17	17	130	170	200	3.3	0.8	GPa
Flexural Stress	18	40	239	-	330	-	65	20	MPa

Subsequent to the model development all model parameters were populated. Each model parameter was populated with values gathered from the knowledge review, as based on earlier published findings. The pipe material characteristic parameter values, presented in Table 4.1, formed part of the discrete inputs required for all further modelling.

The mechanical and physical material characteristic parameter values presented in Table 4.1 are for comparison purposes, to form a better understanding of pipe material behaviour, as well as the failure thereof. Understanding the mechanical and physical material parameter comparison assist with interpreting and explaining the PRP-algorithm results.

The Wadiso software (discussed in Section 4.3.1) is extensively used in South Africa to capture system information and conduct hydraulic modelling. The following additional information is derived from the available physical system characteristics listed above:

- Pipe inside and outside diameter.
- Pipe roughness coefficient
- Water source locations and data, such as the systems energy grade line, derived from the source top water level.

4.1.2 Tier 2, System data

When a complete water-based hydraulic computer model of the water distribution network exists, the water demands and peak flow rates are populated. For each pipe, data integrity is included (in a comment field), with a unique integrity code, to describe the assets data origin. For example, data could have originated from an informal sketch plan (poor data integrity), or from an "as-built" drawing (reasonable data quality), or from a physical site survey (good data integrity). A steady-state model analysis is subsequently performed to obtain hydraulic results. The steady state analysis is needed to generate the following simulation results, for each pipe in the network:

- Static head – System characteristic (Level 1, Tier 2).
- Dynamic head – System characteristic (Level 1, Tier 2).
- Flow – System characteristic. (Level 1, Tier 2).
- Velocity – Interdependence characteristic (Level 3).
- Head loss – Interdependence characteristic (Level 3).

Hereafter, now understanding the demand measurement measures that apply to the area of concern, the water demand and subsequent peak flows were estimated. The hydraulic computer model was populated with the derived peak flows, based on peak demands.

According to Fair (2017), flow meters are used to measure individual consumer water demands. Flow meters are used at strategic points, for conducting water balance and related calculations. The flow meter strategic points are located in the following areas:

- Bulk inflow meters, at pressure zones.
- Reticulation outflow meters, at pressure zone.
- Consumer meters, at point of consumption.

The bulk water meters are used to verify the consumer meters (Fair, 2017). If the bulk outflow meter and total consumer demand do not correlate, an investigation is required to assure that all consumers are included. Some consumers are not metered, as in the rural areas, for which a theoretical demand is calculated, as discussed in Section 2.3.6. When all consumers have been accounted for, the difference between the bulk outflow meter and total consumer demands is regarded as non-revenue water. Non-revenue water includes illegal connections, broken or uncalibrated meters and leakages. From the metered data, a one-year dataset is required, to successfully calculate the Average Annual Daily Demand (AADD) of each metered consumer.

The peak flow rate, used to populate the hydraulic computer model node output, was based on an analysis of actual consumer demands. Water meter readings formed the basis for the demand analysis, in a process similar to that described by Jacobs and Fair (2012). No non-revenue water would thus be accounted for in the hydraulic model, because consumption records from the billing system exclude real losses. Real losses were determined by conducting a water balance and then including it in the hydraulic model by spreading the total real loss over all nodes equally.

With a completed water-based hydraulic computer model a steady state analysis was performed to generate all the necessary hydraulic system data results and conclude the building of a complete water model.

4.2 Asset condition data

The asset condition data represents the operation condition of assets. The asset condition data was obtained from the City of Tshwane asset management register (Mouton, 2015) and was used in the study for system diagnostic purposes. The asset condition datasets typically comprise visual condition assessment data (CCTV inspections) and pipe failure repair logging data.

4.2.1 Visual condition assessment datasets

Visual condition assessment datasets are commonly referred to as CCTV inspections. These datasets are regarded as the ultimate dataset to use for condition assessment data, but are not always available due to the cost implications. CCTV data is often viewed as a luxury rather than a necessity in water distribution networks.

Datasets from CCTV inspection were found to be readily accessible for sewers, but not for potable water, where CCTV inspections are uncommon. Visual condition assessment datasets were not used in the PRP-algorithm.

4.2.2 Failure repair logging data

For the study area, pipe failure and pipe repair are logged in the commercially available integrated business information system (IBIS). Data is extracted from the IBIS for use in the study. An excerpt from the IBIS data is presented in Appendix I.

The following additional data are available for each repair event:

- Date and job reference number.
- The type of job done.
- Description of the pipe that was worked on.
- Activity cost.
- Stand, street or street corner closest to where the work was done.

The failure repair datasets are exported to Microsoft Excel (as mentioned in Section 4.3.4), and geo-referenced back to the corresponding stands, street or street corner and efficiently linked to the closest pipe in the Wadiso model (as mentioned in Section 4.3.1), using the reported address. The geo-referencing of failure locations to hydraulic model pipe elements was time-consuming, as it requires a manual process.

After linking the failure event to the pipes, a failure description verification process is required. The failure description is matched to the pipe description, which is based on the failure year and diameter. The process is depicted schematically in Figure 4.3.

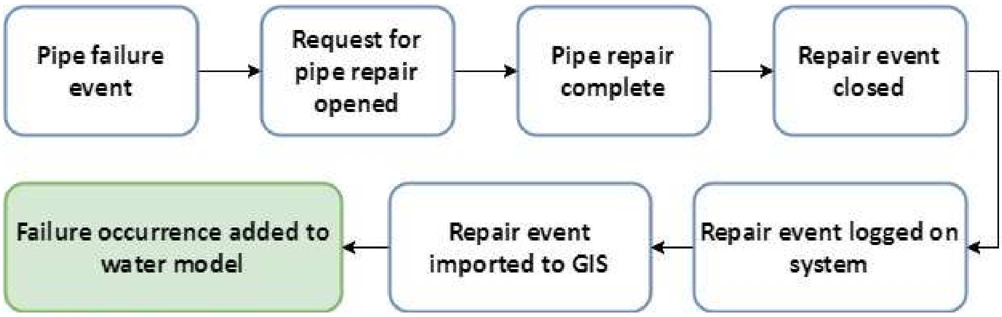

Figure 4.3 Integration of failure data to hydraulic model.

A notable challenge related to the use of failure data is the incorrect capturing of information, such as job type, pipe description, as well as stand and street address. The integrity of the data is compromised by poor data capturing, which leads to incorrect failure allocations.

4.3 Programs required for data collection

The following programs were used for data collection, development, restructuring and the PRP-analysis:

- Wadiso.
- Swift.
- Albion.
- Excel.

4.3.1 Wadiso

Wadiso is a comprehensive computer program for water distribution network design and analysis. The Wadiso program uses a seamless interface to the public domain EPANET program module. The program integrates the following features to allow a graphic display of data and results (GLS Software, 2017):

- Steady state analysis module.
- Extended period simulation module.
- Optimisation module.
- Water quality module.

4.3.2 Swift

Swift is a comprehensive computer program, which allows the user to take the information set of water consumption from a Municipal Treasury database and restructure the information to perform statistical calculations (GLS Software, 2017). The Swift program generates statistical reports, while spatially allocating the data to a cadastral database in GIS (Jacobs and Fair, 2012). Swift can produce the following statistical requirements:

- Water demand management initiatives.
- Water audits.
- Non-revenue water calculations.
- Calculation of water tariffs.
- Water consumption profiles for user-defined categories.
- Water and sewer master plans.
- Identify faulty meter readings.
- Populate the databases of water and sewer model.

4.3.3 Albion

Albion is a 2D CAD and GIS platform, developed for working with large amounts of data (GLS Software, 2017). The computer program makes use of sophisticated database wrapping techniques, which allows the user to work with any information in the same workflow manner as one should when editing a spreadsheet.

4.3.4 Microsoft Excel

IT Business Edge (2017), describes Microsoft Excel as a spreadsheet program, which is included in the Microsoft Office suite applications. The computer program presents the spreadsheets in the form of tables with values arranged in rows and columns, of cells. The cells are mathematically manipulated by using basic or complex arithmetic operations and functions.

4.4 Data used

The dataset used to develop and test the pipe replacement algorithm is from the City of Tshwane Municipality water distribution and reticulation system model of the area studied. The hydraulic model is a GIS-based model, which was analysed as a steady state model, for demand scenario average annual daily demand including non-revenue water (AADD including non-revenue water) without peak factors, for the billing and as-built data of up to January 2015.

The City of Tshwane Municipality study area water distribution and reticulation system is a relatively large system, with data of mixed integrity. The inner reticulation system south of the Magaliesberg up to the Constantia park tower zone boundary (within the City of Tshwane Municipality boundaries) data was assessed and confirmed to be of good integrity. Therefore, this section of water reticulation system was used for the PRP-analysis, which comprised the following 35 pressure zones, with a total of 2021.35 km of water reticulation pipes, to define the study area, as described in Table 4.2. The study area water reticulation pressure zones (discussed in Section 4.4) are shown in Figure 4.4, and the existing water reticulation pipes in the study area in Figure 4.5.

Table 4.2 Pressure zones in the study area.

ID	Pressure zone	ID	Pressure zone	ID	Pressure zone
1	Carina reservoir	13	Iscor reservoir	25	Murrayfield reservoir
2	Constantia Park reservoir	14	Kilner Park reservoir	26	Murrayfield tower
3	Constantia Park tower	15	Koedoesnek HL feeder Direct 1	27	Parkmore HL reservoir
4	Eersterust reservoir	16	Koedoesnek HL reservoir	28	Parkmore LL reservoir
5	Findlay reservoir	17	Koedoesnek LL reservoir	29	Queenswood reservoir
6	Garsfontein Direct 1	18	Lynnwood reservoir	30	Salvokop reservoir
7	Garsfontein Direct 3	19	Magalies reservoir	31	Suiderberg reservoir
8	Heights HL Direct 3	20	Meintjieskop reservoir	32	Waterkloof East reservoir
9	Hercules East BPT	21	Moreleta reservoir	33	Waterkloof reservoir
10	Hercules East reservoir	22	Moreleta tower	34	Waverley HL reservoir
11	Hercules West reservoir	23	Muckleneuck reservoir	35	Waverley LL reservoir
12	Hospital reservoir	24	Muckleneuck tower		

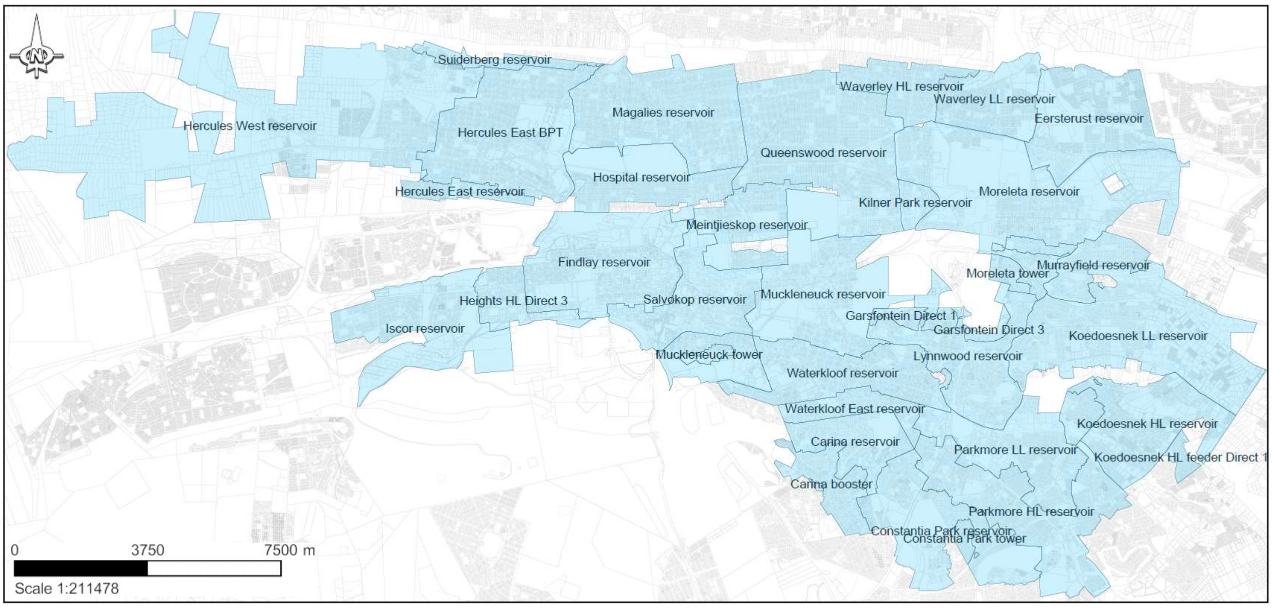

Figure 4.4 Pressure zones in the study area over, a cadastral layout, as presented in Appendix J.

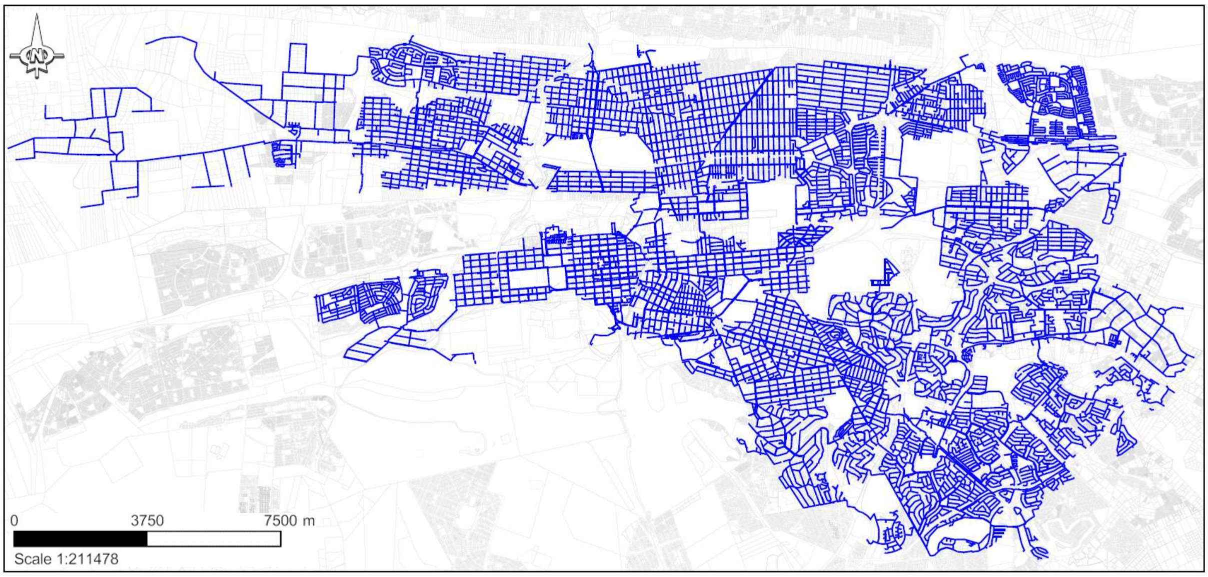

Figure 4.5 Existing water reticulation pipes in the study area, as presented in Appendix K.

No CCTV data was available. However, there was a complete dataset of failure and repair loggings, obtained from the IBIS dataset. The IBIS dataset referred to is the City of Tshwane Municipality's failure and repair logging system. The failure logging data was exported, from the IBIS, for the period, 01/01/2000 to 31/12/2014, which represented a 15-year dataset. The dataset was edited to match 12802 failure events, which represented a 65% success rate for failure events geo-referenced and linked to the hydraulic computer model in the study area.

The verified and linked failure events are illustrated in Figure 4.6. The failure events illustrated in Figure 4.6 were categorised according to the year the failure event occurred, represented by uniquely coloured dots. The failure events were geo-spacially captured into the relevant water reticulation pressure zone on the exact location of occurrence

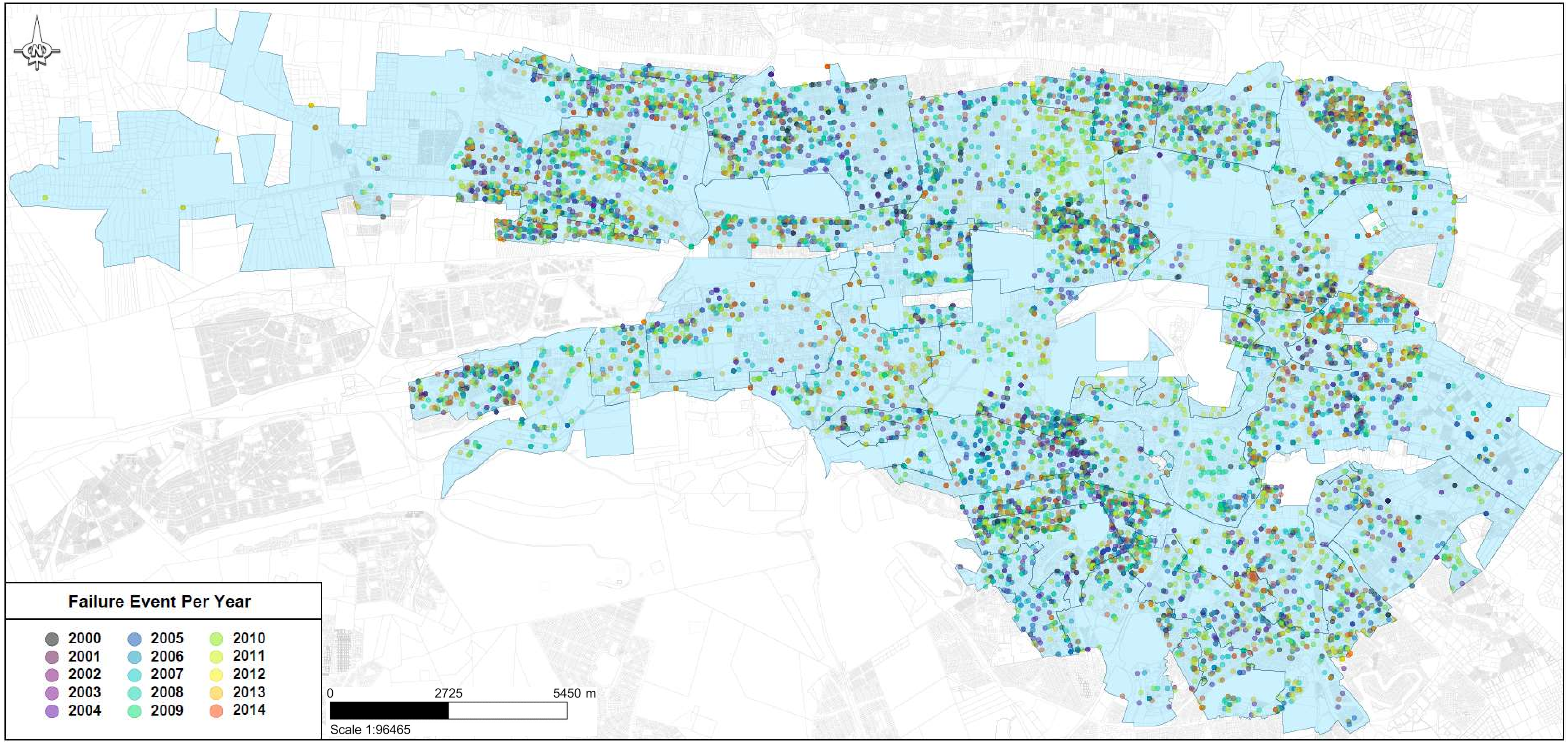

Figure 4.6 Failure points matched to the study area, as captured from IBIS.

5. CASE STUDY RESULTS

The PRP-methodology, as described in Section 3, was implemented on the current City of Tshwane Municipality water reticulation distribution model, as outlined in Section 4.4. The following results were obtained:

- Failure frequency graph results for all identified network characteristics.
- Level one pipe replacement prioritisation.
- Level two pipe replacement prioritisation.
- Level three pipe replacement prioritisation.
- Final pipe replacement prioritisation.
- Pipe replacement prioritisation verification.

5.1 Failure frequency graph results

The first step of the analysis was to connect the corresponding failure logging data to each pipe and to determine the pipe failure frequency, per pipe. The following failure frequency graphes were derived, based on the pipe failure frequency for the following water distribution characteristics:

- Material.
- Remaining useful life (RUL).
- Static pressure.
- Residual pressure.
- Reserve pressure ratio.
- Diameter.

Figure 5.1 shows the graph generated, which represents the calculated failure frequency of each of the different pipe materials used within the study area. In Figure 5.1 the x-axis ranges were grouped according to the different pipe materials.

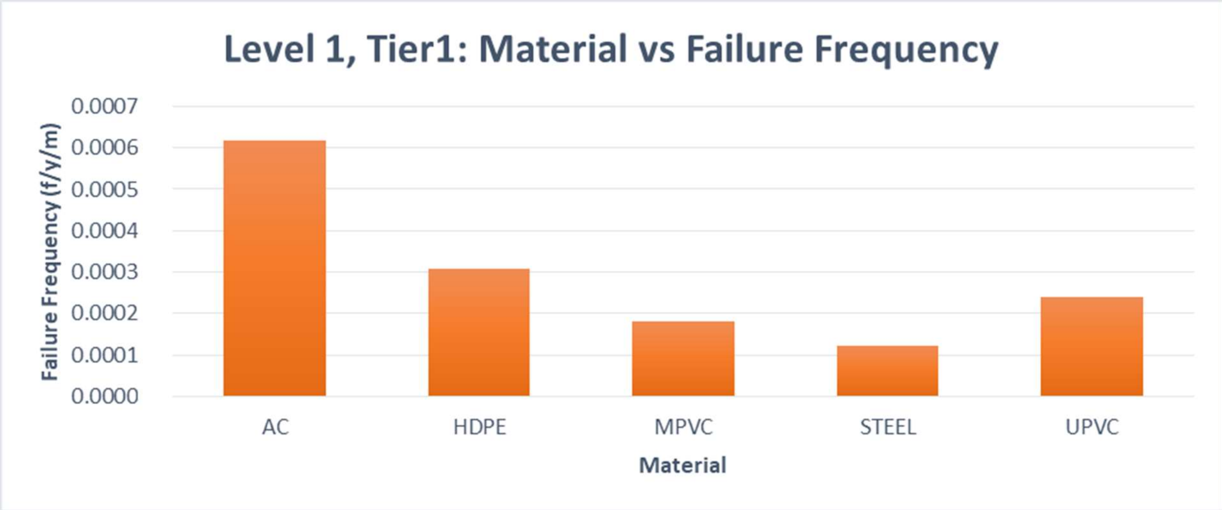

Figure 5.1 Level 1, Tier 1: Failure frequency of water reticulation pipe material.

AC, as a water reticulation pipe material, was calculated to have the highest frequency of failure, as illustrated in Figure 5.1. The high value can be explained by the fact that AC pipe material is still in use in the study area, although it is regarded as an old technology pipe material, which is starting to fail due to the material reaching its life cycle end.

The calculated failure frequency for a pipe's remaining useful life was then developed, ranged and plotted as illustrated in Figure 5.2. In Figure 5.2 the x-axis was grouped by fifteen year ranges.

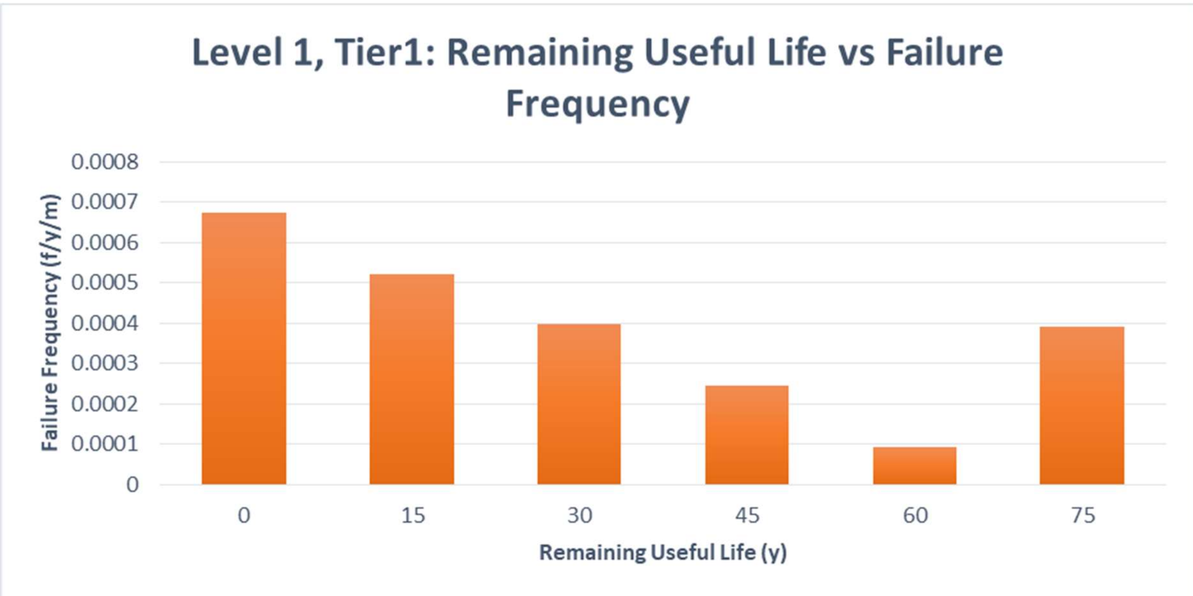

Figure 5.2 Level 1, Tier 1: Failure frequency for remaining useful life of water reticulation pipes.

Pipes with a remaining useful life of 0 years was calculated to have the highest failure frequency, as illustrated in Figure 5.2. The high value indicated that pipes had reached the end of their material life cycle and started to fail. An interesting failure frequency 'bath type' curve was formed for the range, from newly installed pipes to old pipes. According to Trifunović (2013) the 'bath type' curve for failure frequency per pipe's remaining useful life in water distribution zones was considered typical, as illustrated in Figure 5.3.

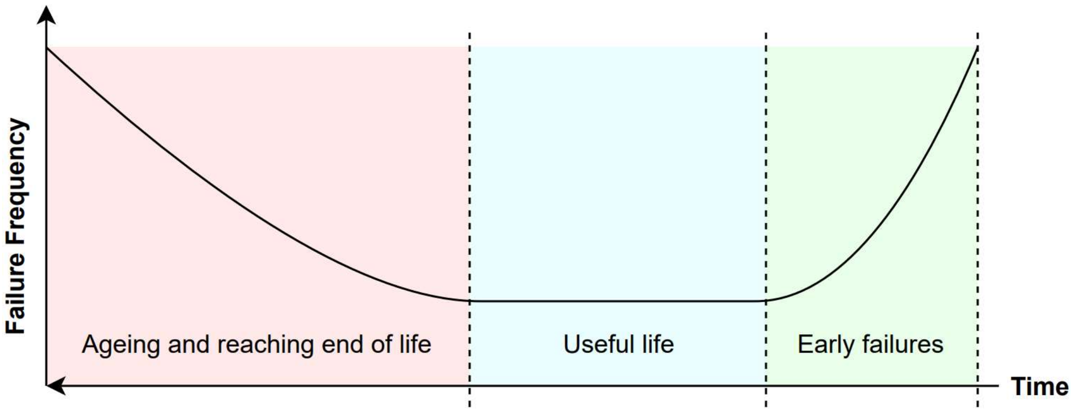

Figure 5.3 'Bath type' curve representing failure frequency for remaining useful life of pipes (Trifunović, 2013).

The 'bath type' trend assisted in understanding the life cycle of a pipe, which indicated that after a pipe installation, the pipe settles into its environment, during which time leaks and failures start to develop. The settling period was considered as over, once minimal new leaks or failures developed. During the steady period, pipe deterioration takes place gradually, up until total pipe failure occurs, indicating the end of the pipe's life cycle.

The calculated failure frequency for static pressure was then developed, ranged and plotted as illustrated in Figure 5.4. In Figure 5.4 x-axis ranges were grouped by every 15 m static pressure.

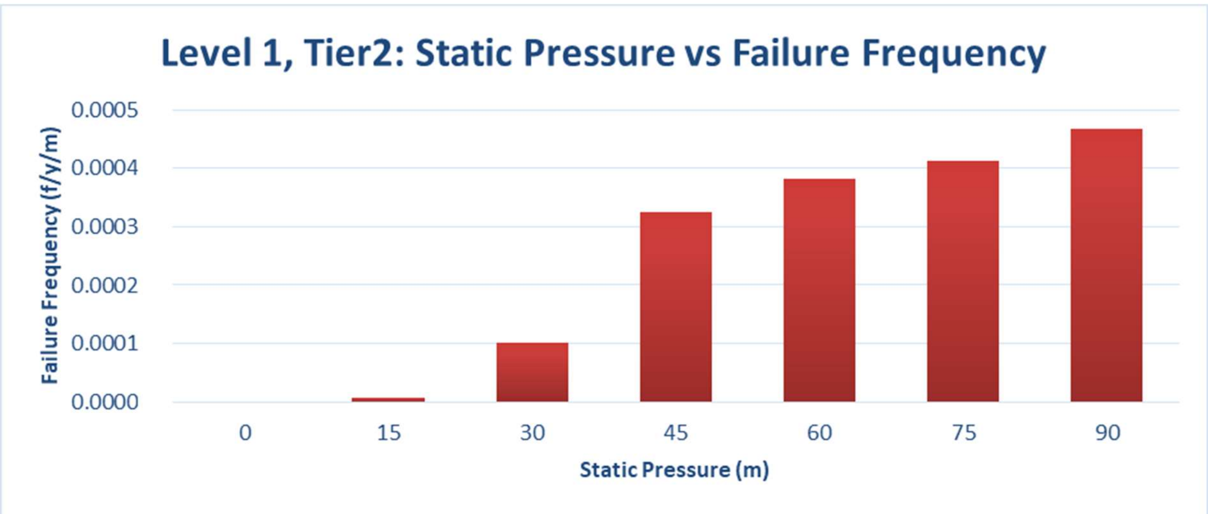

Figure 5.4 Level 1, Tier 2: Failure frequency for static pressure in water reticulation pipes.

The static pressure of between 75 and 90 m was calculated to result in the highest failure frequency, as illustrated in Figure 5.4. The graphed failure frequency for static pressure confirmed that pipes with a higher static pressure were more likely to be subjected to a failure event. The trend also highlighted that replacing areas exposed to high static pressure was not the best option, as pipe failures will keep on occurring until pressure management has been implemented as a solution.

The calculated failure frequency for residual pressure was then developed, ranged and plotted as illustrated in Figure 5.5. In Figure 5.5 the x-axis ranges were grouped by every 15 m residual pressure.

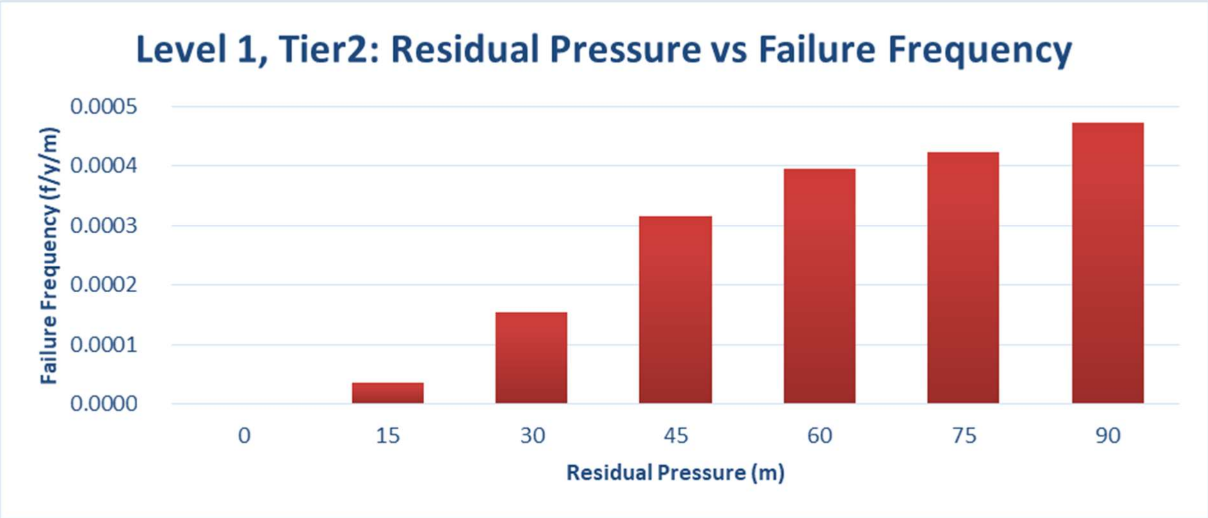

Figure 5.5 Level 1, Tier 2: Failure frequency for residual pressure in water reticulation pipes.

The residual pressure of between 75 and 90 m was calculated to have resulted in the highest failure frequency, as illustrated in Figure 5.5. The graphed failure frequency for residual pressure confirmed that pipes with a higher residual pressure were more likely to be subjected to a failure event. The trend also highlighted that replacing areas exposed to high residual pressure was not the best option, as pipe failures will keep on developing until pressure management has been implemented as a solution. The trend illustrated in Figure 5.5 was similar to the trend illustrated in Figure 5.4, which confirms that residual pressure is dependent on the static pressure characteristic of a pipe and, as a combination, represents the system hydraulic characteristic of the algorithm, as discussed in Section 3.

The calculated failure frequency for reserve pressure ratio was then developed, ranged and plotted as illustrated in Figure 5.6. In Figure 5.6 the x-axis ranges were grouped by the reserve pressure ratio value of 0.2.

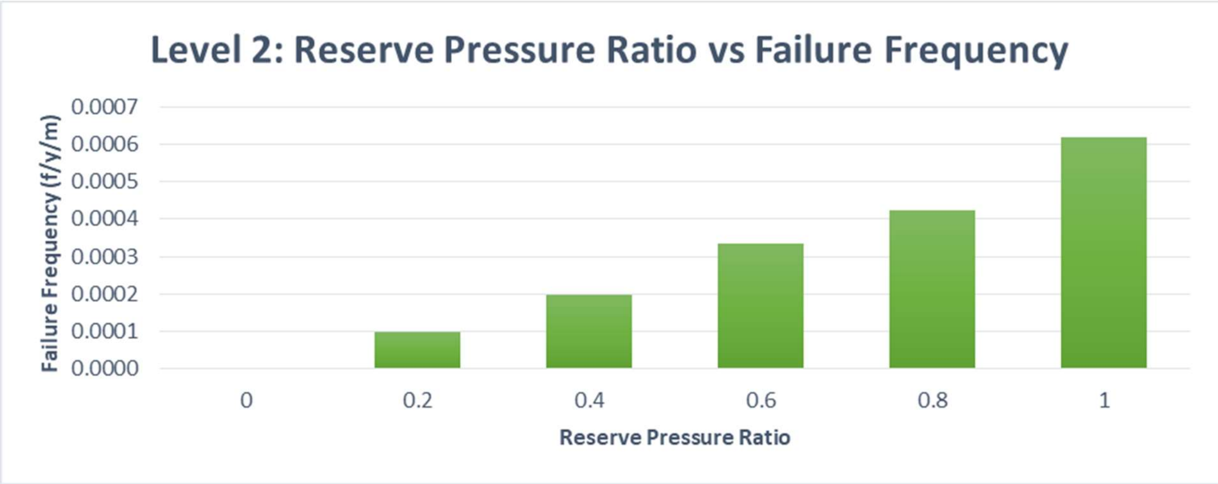

Figure 5.6 Level 2: Failure frequency for reserve pressure ratio in water reticulation pipes.

The reserve pressure ratio of between 0.8 and 1 was calculated to have the highest failure frequency value, as illustrated in Figure 5.6. The graphed failure frequency for reserve pressure ratio confirmed that pipes with a higher reserve pressure ratio were more likely to be subjected to a failure event. The trend highlighted that an underdesigned water reticulation network, regarding material selection and available system flow capacity, was more likely to experience failures.

The failure frequency for diameter was then calculated and plotted as illustrated in Figures 5.7 and 5.8. In Figures 5.7 and 5.8 the x-axis ranges were grouped by every 100 mm diameter. The differentiation between Figure 5.7 and Figure 5.8 was small and large diameter pipes, which implies that the general range of plastic pipes in the study area would be between 0 and 400 mm. With the limited range of diameter available in plastic material pipes, joints between different pipe materials, as well as smaller to larger diameters, were required.

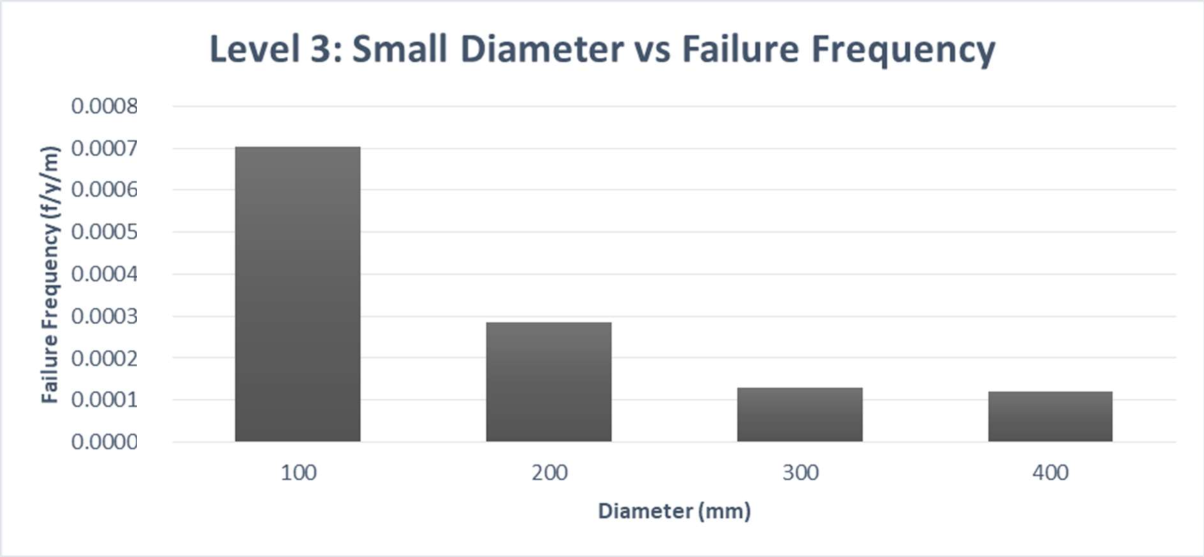

Figure 5.7 Level 3: Failure frequency according to water reticulation pipe diameter (small).

The relatively small pipe diameters of between 0 and 100 mm was calculated to have the highest level of failure frequency, as illustrated in Figure 5.7. The graphed failure frequency for small diameter pipes decreases as the diameter increases, which confirms the findings of Rathnayaka (2016).

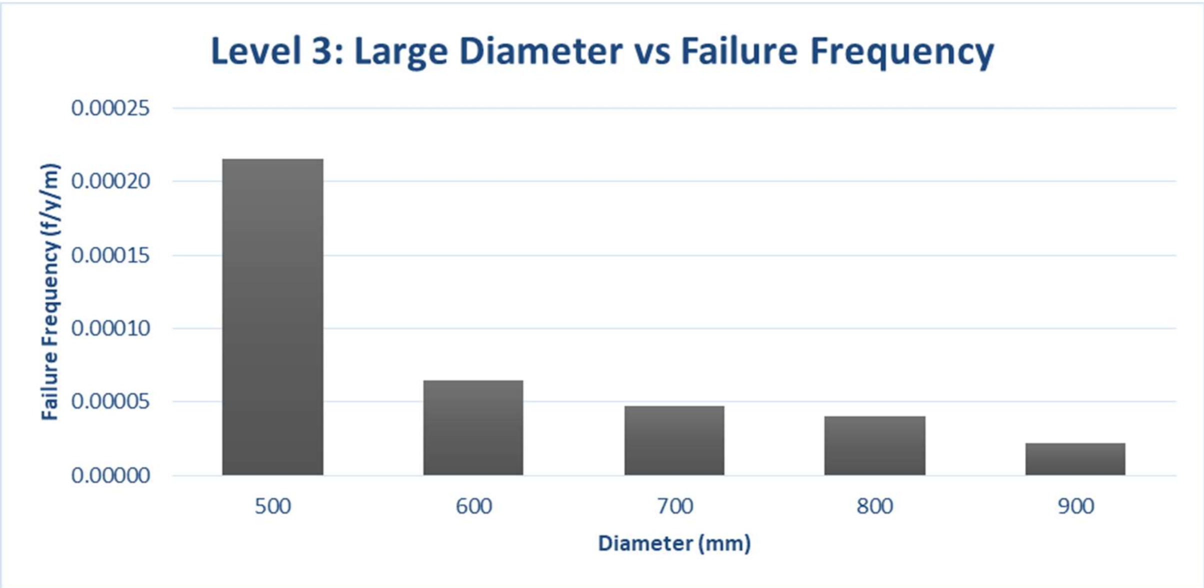

Figure 5.8 Level 3: Failure frequency according to water reticulation pipe diameter (large).

The diameter of between 400 and 500 mm was calculated to have the highest failure frequency value for large diameter pipes, as illustrated in Figure 5.8. The graphed failure frequency for large diameter pipes decreases as the diameter increases. The high failure frequency for the diameter of 500 mm pipes was explained as being due to the high possibility of failures occuring between joints of different pipe materials, which confirms the findings of Rathnayaka (2016).

5.2 Level 1; tier 1 and tier 2 prioritisation results

Once all the failure frequency graphs had been developed, the failure frequencies were matched onto each pipe in the study area, as described in Section 3.3. The prioritisation calculations were grouped into water distribution zones to represent prioritisation. The water distribution zone failure frequency index results were generated for Level 1, Tier 1, which consists of the sum of material and remaining useful life failure frequencies, multiplied by the Level 1, Tier 1 index weight of 0.167, shown in Figure 5.9. An index weight of 0.167 was used for all algorithm index weightings, to ensure an evenly distributed failure frequency index over all six water distribution characteristics, for the first round of algorithm calibration, as discussed in Section 3.3 and 3.4.

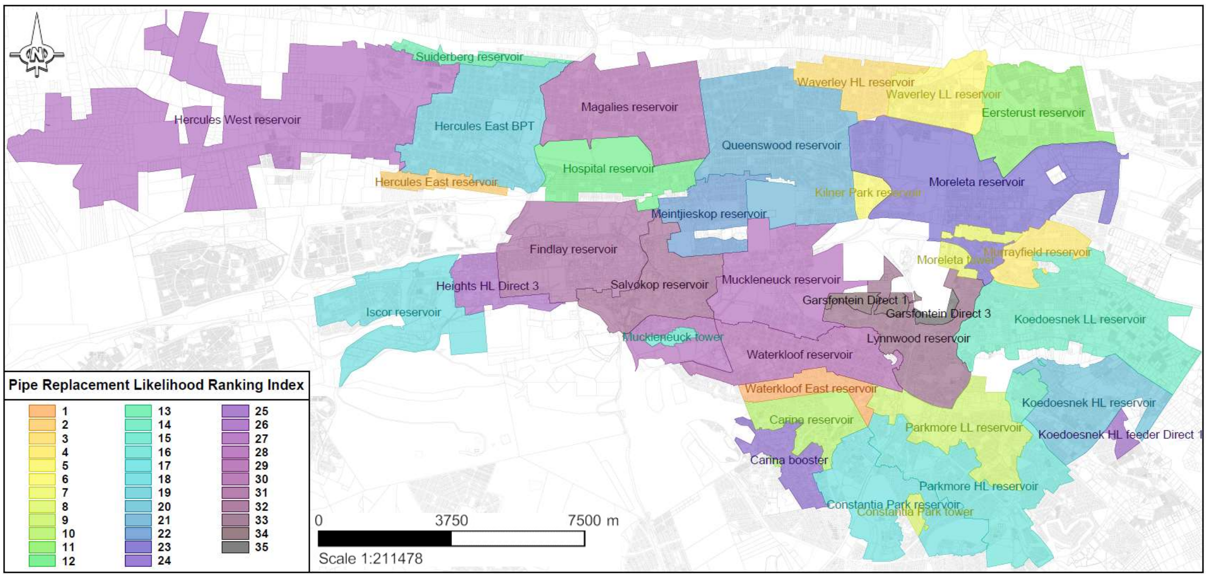

Figure 5.9 Level 1, tier 1 pipe failure frequency index prioritisation for the study area.

The failure frequency index results, giving the likelihood of pipe replacement for the area were based on the physical characteristics of water distribution, as explained in Section 3.3.1, and illustrated in Figure 5.9, which verified that areas were ranked with highest (orange) to the lowest (deep purple) on the failure frequency index. The water distribution zone of Waterkloof East reservoir, Hercules East reservoir and Waverley HL reservoir were identified as the areas with the highest failure frequency index for likelihood of water pipe replacement, regarding physical characteristics. The high failure frequency index for the likelihood of pipe replacement (orange) was an indication of the presence of significant amounts of AC pipe material with a remaining useful life of 0 years.

The water distribution zone failure frequency index results were generated for Level 1, Tier 2, which results consisted of the sum of the failure frequencies caused by static pressure and residual pressure, multiplied by the Level 1, Tier 2 index weight of 0.167, as shown in Figure 5.10.

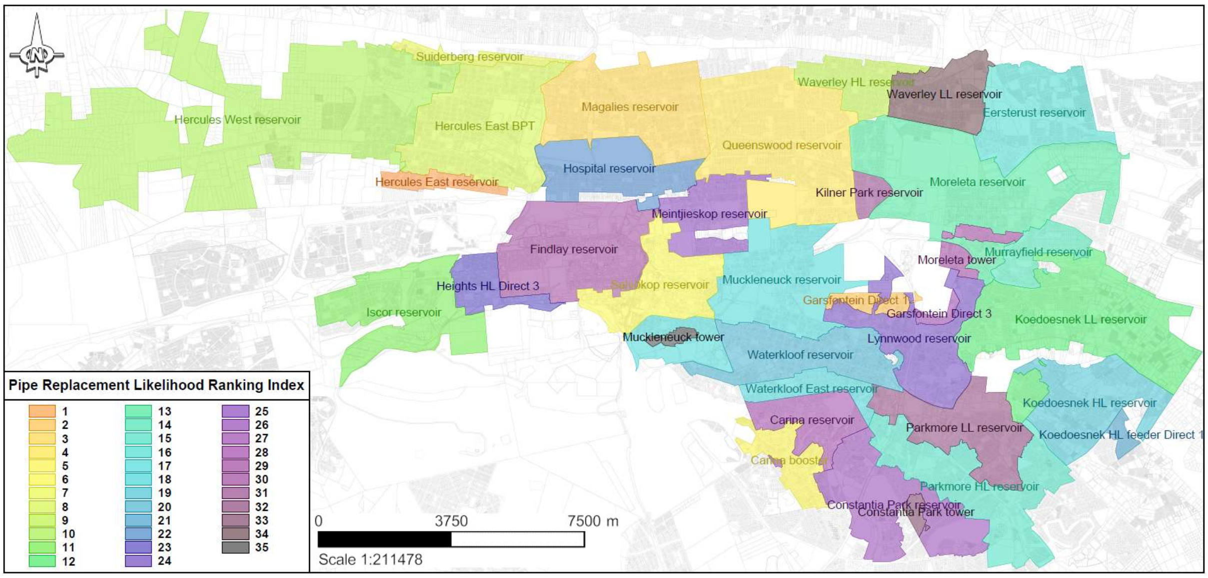

Figure 5.10 Level 1, tier 2 pipe failure frequency index prioritisation for the study area.

The failure frequency index results, giving the likelihood of pipe replacement for the area were based on water distribution system characteristics as explained in Section 3.3.2, and illustrated in Figure 5.10, which verified that areas were correctly ranked with highest (orange) to lowest (deep purple) failure frequency index. The water distribution zone of Hercules East reservoir, Garsfontein Direct 1 and Magalies reservoir were identified as the areas with the highest likelihood for water pipe replacement according to the failure frequency index, regarding system characteristics. The high failure frequency index for the likelihood of pipe replacement (orange) was an indication of distribution zones with a prevailing static pressure of between 75 and 90 m and vast areas with residual pressure of between 75 and 90 m.

The results for Tier 1 and Tier 2 were combined and summed to generate the Level 1 failure frequency index results, which were ranked for prioritisation. Level 1 prioritisation allows item identification for elimination from the prioritisation process, which supplied the user with the flexibility to support a specified outcome need. No items were removed, as final verification of prioritisation first needed to be completed. The water distribution zone's failure frequency index results for the likelihood of pipe replacement were generated for Level 1 prioritisation, as shown in Figure 5.11.

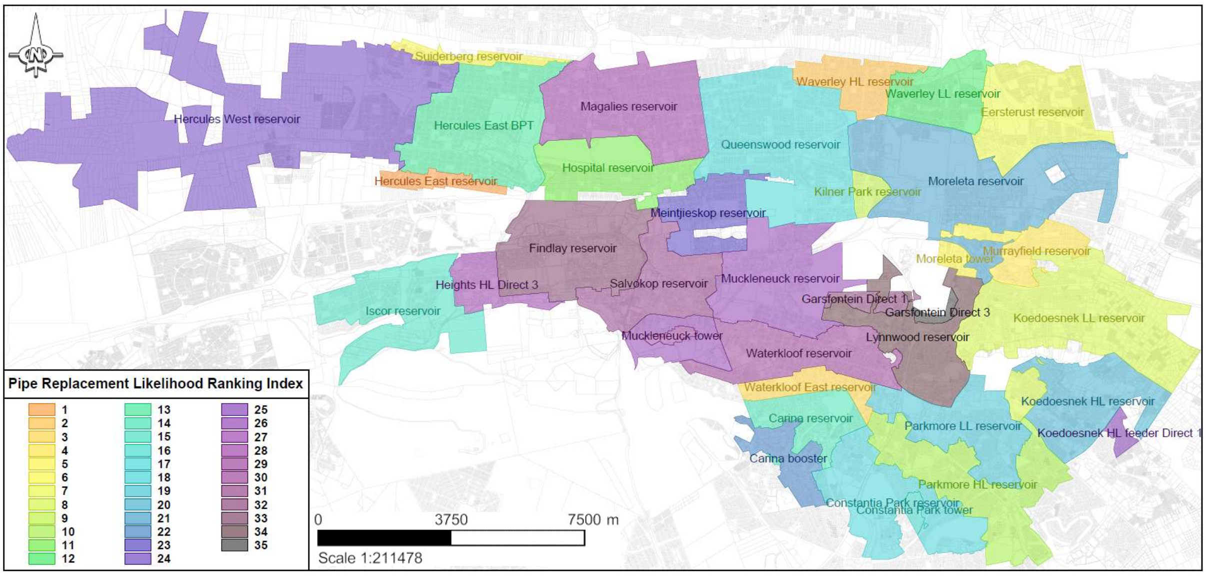

Figure 5.11 Level 1 pipe failure frequency index prioritisation for the study area.

The failure frequency index results for the likelihood of pipe replacement for the area were based on physical and system characteristics of water distribution, as explained in Section 3.3.2. As illustrated in Figure 5.11, this has verified that areas are correctly ranked with highest (orange) to lowest (deep purple) on the failure frequency index. The water distribution zone of Hercules East reservoir, Waverley HL reservoir and Waterkloof East reservoir were identified as the areas with the greatest likelihood of failure and of water pipe replacement on the failure frequency index, regarding their combined physical and system characteristics. The high failure frequency index rating for the likelihood of pipe replacement (orange) was an indication of distribution zones with large amounts of AC pipe material with a remaining useful life of 0 years, a prevailing static pressure of between 75 and 90 m and vast areas with residual pressure of between 75 and 90 m.

5.3 Level 2 prioritisation results

Completing Level 1 of PRP introduced Level 2 of prioritisation. Reserve pressure ratio failure frequency index values were added to the Level 1 study area prioritisation algorithm. The water distribution zone failure frequency index results for the likelihood of pipe replacement were generated for Level 2 prioritisation, shown in Figure 5.12.

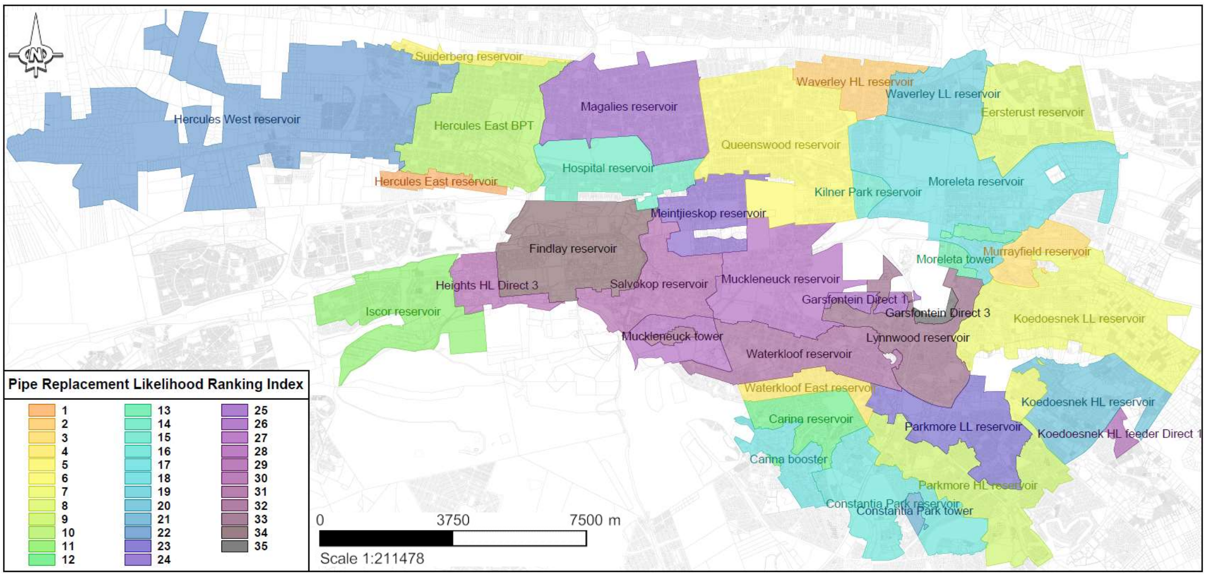

Figure 5.12 Level 2 pipe failure frequency index prioritisation for the study area.

The failure frequency index results for the likelihood of pipe replacement for the area were based on water distribution interdependence characteristics as explained in Section 3.3.3. As illustrated in Figure 5.12, it verified that areas were correctly ranked with highest (orange) to lowest (deep purple) failure frequency index. The water distribution zone of Hercules East reservoir, Waverley HL reservoir and Murrayfield reservoir were identified as the areas with the highest likelihood for water pipe replacement, according to the failure frequency index, regarding combined physical and system characteristics, as well as the added component of reserve pressure ratio. The high failure frequency index rating, indicating the likelihood of pipe replacement (orange) was an indication of distribution zones with large amounts of AC pipe material with a remaining useful life of 0 years, a prevailing static pressure of between 75 and 90 m with pressure residual pressure of between 75 and 90 m, as well as a dominant reserve pressure ratio of between 0.8 and 1.

5.4 Level 3 and final prioritisation results

Level 3 of prioritisation was subsequently introduced. Failure frequency index values of diameter were added to the Level 2 study area prioritisation algorithm. The water distribution zone failure frequency index results for the likelihood of pipe replacement were generated for Level 3 prioritisation, shown in Figure 5.13.

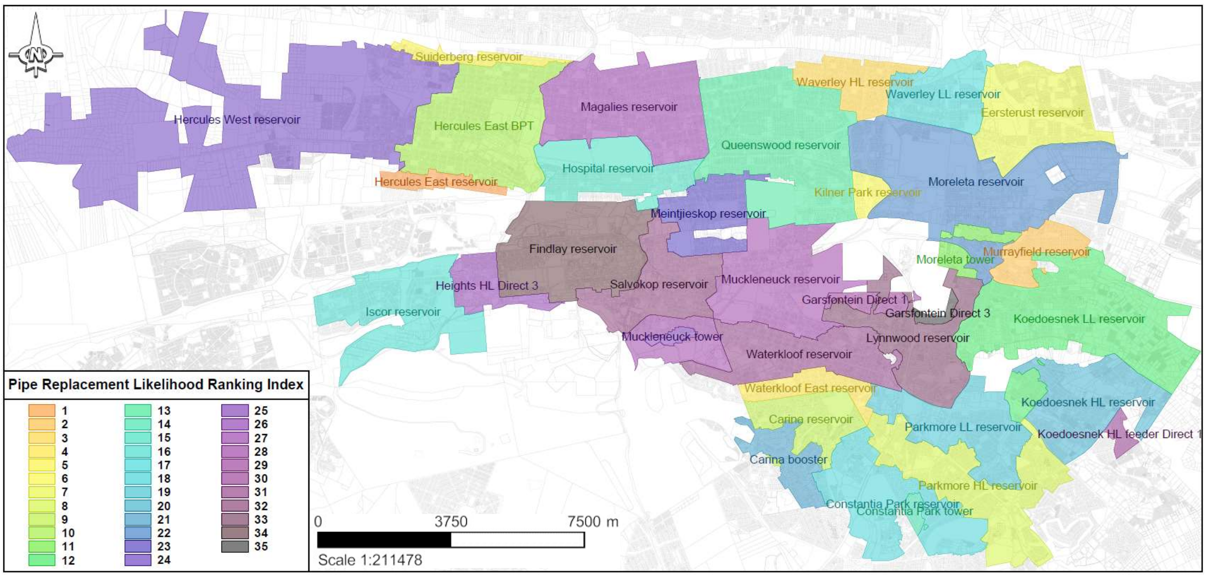

Figure 5.13 Level 3 and Final pipe failure frequency index prioritisation for the study area.

The failure frequency index results for the likelihood of pipe replacement for the area were based on water distribution interdependence characteristics as explained in Section 3.3.3. As illustrated in Figure 5.13, it can be verified that areas are correctly ranked with highest (orange) to lowest (deep purple) failure frequency index ranking. The water distribution zone of Hercules East reservoir, Murrayfield reservoir and Waverley HL reservoir were identified as the areas with the highest failure frequency index likelihood for water pipe replacement, regarding combined physical and system characteristics, as well as the added components of reserve pressure ratio and pipe diameter. The high failure frequency index ranking for the likelihood of pipe replacement (orange) was an indication of distribution zones with large amounts of AC pipe material with a remaining useful life of 0 years and a prevailing static pressure of between 75 and 90 m with residual pressure of between 75 and 90 m. The high failure frequency index (orange) is also an indication of distribution zones with a dominant reserve pressure ratio of between 0.8 and 1, and pipe diameters of between 0 and 100 mm.

5.5 Prioritisation verification results

The PRP-results from the final algorithm, grouped into prioritised water distribution zones, were ranked against the prioritised verification failure frequency results. The verification failure frequency results were generated from failure count per year per length of pipe for the distribution zones, as discussed in Section 3.5. The final prioritisation verification procedure (as discussed in Section 3.5) produced the results as shown in Table 5.1.

Table 5.1 Verification of algorithm results and correlation with failure frequency results.

Water distribution zone	Prioritisation		Water distribution zone	Prioritisation	
	Algorithm rank	Verification rank		Algorithm rank	Verification rank
Hercules East reservoir	1	2	Parkmore LL reservoir	19	18
Murrayfield reservoir	2	3	Koedoesnek HL reservoir	20	32
Waverley HL reservoir	3	4	Carina booster	21	17
Waterkloof East reservoir	4	5	Moreleta reservoir	22	21
Suiderberg reservoir	5	9	Meintjieskop reservoir	23	13
Kilner Park reservoir	6	1	Hercules West reservoir	24	29
Eersterust reservoir	7	8	Muckleneuck tower	25	30
Parkmore HL reservoir	8	16	Heights HL Direct 3	26	23
Carina reservoir	9	6	Magalies reservoir	27	24
Hercules East BPT	10	10	Muckleneuck reservoir	28	26
Moreleta tower	11	7	Garsfontein Direct 1	29	33
Koedoesnek LL reservoir	12	14	Koedoesnek HL feeder Direct 1	30	28
Queenswood reservoir	13	15	Waterkloof reservoir	31	22
Constantia Park tower	14	20	Salvokop reservoir	32	27
Hospital reservoir	15	11	Lynnwood reservoir	33	34
Iscor reservoir	16	19	Findlay reservoir	34	31
Waverley LL reservoir	17	12	Garsfontein Direct 3	35	35
Constantia Park reservoir	18	25			

The prioritisation verification for the top ten ranked water distribution zones resulted in an 95% correlation and for the top twenty, a 97% correlation, calculated as discussed in Section 3.5. Based on the satisfactory verification results, the PRP-algorithm was implemented on the water reticulation system of the entire City of Tshwane Municipality for analysis. The expanded implementation area included regions both with and without failure logging data.

Although the prioritisation verification results were satisfactory for the top ten and twenty ranked water distribution zones, a number of conflicting comparisons did exist. The conflicting comparisons required a more critical analysis for the water distribution zones of Constantia Park reservoir, Constantia Park tower, Parkmore HL reservoir, Kilner Park reservoir Mientjieskop reservoir and Waverley LL reservoir.

The critical analysis for Constantia Park reservoir received a verification rank of 25 and an algorithm rank of 18, Constantia Park tower received a verification rank of 20 and an algorithm rank of 14 and Parkmore HL reservoir received a verification rank of 16 and an algorithm rank of 8. Constantia Park reservoir, Constantia Park tower and Parkmore HL reservoir indicated an algorithm overestimation of failure frequency for material, remaining useful life and diameter.

The critical analysis for Kilner Park reservoir received a verification rank of 1 and an algorithm rank of 6, Meintjieskop reservoir received a verification rank of 13 and an algorithm rank of 23 and Waverley LL reservoir received a verification rank of 12 and an algorithm rank of 17. Kilner Park reservoir, Meintjieskop and Waverley LL reservoir indicated an algorithm underestimation of failure frequency for static pressure, residual pressure and reserve pressure ratio.

The over- and underestimation of failure frequencies that were identified in the critical analysis could be corrected by refined characteristics x-axis grouped ranges and adjusted index weightings (as discussed in Section 3.5). The algorithm was representative for areas in the City of Tshwane and was therefore recommended to be developed and tested in other parts of South Africa.

6. CONCLUSION

6.1 Discussion

Results were generated with the two-tier PRP-algorithm for the selected water reticulation distribution zones, in the City of Tshwane Municipality study area. The actual pipe failure frequency data (for the study area), was subsequently compared to the theoretical results in order to validate the algorithm. The algorithm required a significant amount of data processing, analysis and research. The algorithm was used to produce a pipe replacement priority in line with the research objectives.

The algorithm followed a multi-level structured method of a ranked pipe failure frequency index for the likelihood of prioritisation. The algorithm made use of a predictive approach through the simple quantitative prioritisation technique, based on a Poisson process model, as the failure prediction input. The selected pipes or grouped areas with the highest failure frequency index were ranked as top priority for likelihood of replacement.

The algorithm makes use of a two-tier multi-level structured prioritisation approach, developed through the system characteristics, physical characteristics and interdependence between the two tiers. Each of the characteristics used in the algorithm presented failure frequency values spatially. The algorithm was a representation of failure occurrences, rather than using failure occurrences as the main characteristic (physical, system or interdependence). This approach allows areas both with and without failure data to be evaluated in the same analysis. The overall network analysis presents the opportunity to optimise return on investment, with all relevant assets included.

The algorithm was developed to focus on pipe replacement, and not on pipe maintenance, by focusing on structured characteristics for the likelihood of failure. Preventing confusion between replacement and maintenance schemes, allows for effective budget planning with regard to reticulation pipe replacement.

All the main objectives were achieved in developing a water reticulation PRP-algorithm, which can be used as a tool for informed decision-making. Although the algorithm was sufficient, regarding the case study area, the evaluation of results highlighted key findings and shortcomings, which require additional research.

6.2 Further Research

Further research is required to optimise the failure frequency index weightings of physical, system and interdependence characteristics, to improve verification of the algorithm. Developing an optimised way of calculating the index weighting will add to the algorithm's sensitivity and ultimately improve the accuracy of the output results of the algorithm.

Current methods for reporting failures and capturing the details of failures on a GIS could be standardised and improved. Optimised failure reporting ensures that datasets are accurate, by allocating failures to the correct pipe, which significantly improves the integrity of the data. The research needs to focus on a cost-effective approach, which ensures improved data and methods of correlation integrity. Various information system software is on the market, such as IBIS, Smart Citizen and IMQS Maintenance Manager.

The PRP-algorithm could be adjusted to develop a water reticulation pipe rehabilitation or maintenance prioritisation algorithm. The maintenance prioritisation algorithm must operate parallel to the PRP-algorithm to ensure further optimisation regarding return on investment of assets. Gathering a better understanding of the condition of assets in monetary terms, promotes better budget planning and spending.

The level of evaluating the PRP-algorithm results, developed in the study, could be improved. Different ways of evaluation can be researched to ultimately form a more calibrated algorithm and a better understanding of the results.

Given that the PRP-algorithm focuses only on water reticulation networks, further research can be done to expand the algorithm to include water bulk supply and sewer drainage systems. Creating a PRP-algorithm tailored for water reticulation, water bulk supply and sewer drainage networks, would cover the total spectrum of pipe replacement prioritisation.

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APPENDIX A: MATERIAL FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

Material

Row	Material	Link Number	Pipe Length	Failure Count
10	AC	316312	155	15
11	STEEL	349665	195	32
12	AC	347596	80	41
13	AC	308671	100	40
14	AC	300721	100	38
15	AC	304383	70	36
16	AC	312418	60	35
17	AC	341125	105	34
18	UPVC	316124	290	14
19	AC	300064	155	24
20	AC	313248	145	14
21	AC	340373	60	26
22	UPVC	343713	75	22
23	AC	325456	165	25
24	UPVC	346492	60	24
25	AC	341656	60	4
26	UPVC	313517	100	8
27	AC	317993	35	21
28	AC	328682	245	13
29	AC	301793	170	20
30	UPVC	328461	130	10
31	AC	323961	115	11
32	UPVC	315079	160	1
33	UPVC	345796	100	17
34	AC	340284	105	17
35	AC	338791	80	17
36	HDPE	313942	120	17
37	UPVC	345803	10	16
38	AC	323963	115	11
39	AC	323922	225	12
40	AC	323969	260	11

Row Labels	Sum of Failure Count	Sum of Pipe Length
AC	9295	1006910
HDPE	131	28430
MPVC	7	2575
STEEL	142	81260
UPVC	3227	902175
(blank)		
Grand Total	**12802**	**2021350**

Andre-Hugo van Zyl:
=PIVOT TABLE (MATERIAL!$B$9:$E$26063)

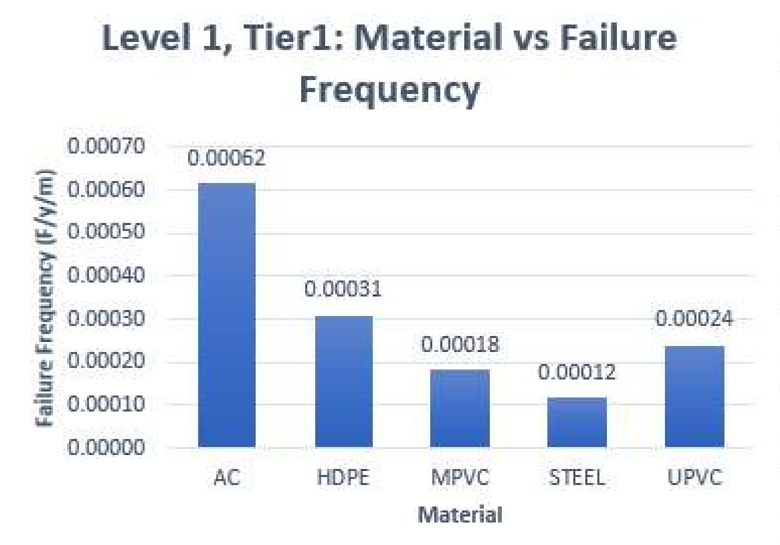
Andre-Hugo van Zyl:
Failure data set period in years

Andre-Hugo van Zyl:
=N10/$L$9/O10

15	Material	Failure Count	Pipe Length	Failure Frequency (Failures/Year/Meter pipe)
	AC	9295	1006910	0.00062
	HDPE	131	28430	0.00031
	MPVC	7	2575	0.00018
	STEEL	142	81260	0.00012
	UPVC	3227	902175	0.00024
	Grand Total	**12802**	**2021350**	**0.00145879**

Andre-Hugo van Zyl:
Vlookup K from Pivot table $I$9:$K$16

Andre-Hugo van Zyl:
Vlookup J from Pivot table $I$9:$K$16



System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX B: REMAINING USEFUL LIFE FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

Remaining Usefull Life

Material	RUL
AC	40
HDPE	80
MPVC	50
STEEL	60
UPVC	50

15 2015 15

Material	Link Number	Pipe Length	Failure Count	Installation Year	RUL Rounded
AC	314627	185	0	1902.00	0.00
AC	313415	75	0	1903.00	0.00
AC	313516	190	0	1903.00	0.00
AC	369208	70	0	1903.00	0.00
AC	308341	50	0	1904.00	0.00
AC	308342	105	1	1904.00	0.00
AC	308343	70	0	1904.00	0.00
AC	308344	10	0	1904.00	0.00
AC	308345	65	0	1904.00	0.00
AC	308346	80	1	1904.00	0.00
AC	308347	60	0	1904.00	0.00
AC	308348	55	0	1904.00	0.00
AC	308351	50	0	1904.00	0.00
AC	308352	90	4	1904.00	0.00
AC	308354	60	2	1904.00	0.00
AC	308361	75	1	1904.00	0.00
AC	308371	80	0	1904.00	0.00
AC	308373	60	0	1904.00	0.00
AC	308374	90	0	1904.00	0.00
AC	308376	90	0	1904.00	0.00
AC	308377	90	0	1904.00	0.00
AC	308380	95	0	1904.00	0.00
AC	308384	85	2	1904.00	0.00
AC	308385	55	1	1904.00	0.00
AC	308390	20	0	1904.00	0.00
AC	308394	15	0	1904.00	0.00
AC	308395	70	0	1904.00	0.00
AC	308396	125	0	1904.00	0.00
AC	308397	85	0	1904.00	0.00
AC	308398	65	0	1904.00	0.00
AC	308401	70	0	1904.00	0.00

Row Labels	Sum of Failure Count	Sum of Pipe Length
0	5437	538670
15	2152	274875
30	2326	389630
45	2691	734835
60	91	65385
75	105	17955
(blank)		
Grand Total	12802	2021350

RUL	Failure Frequency (Failures/Year/Meter pipe)	Failure Count	Pipe Length
0	0.000672892	5437	538670
15	0.000521934	2152	274875
30	0.000397984	2326	389630
45	0.000244136	2691	734835
60	9.27838E-05	91	65385
75	0.000389864	105	17955
Grand Total		12802	2021350

Andre-Hugo van Zyl: Failure data set period in years

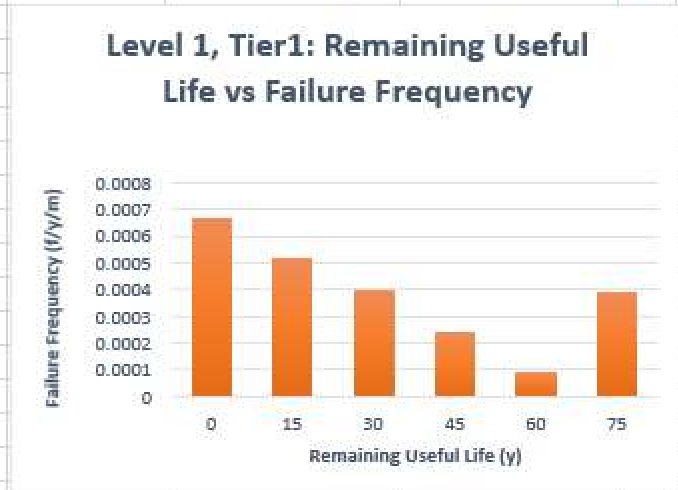
Andre-Hugo van Zyl: Remaining usefull life

Andre-Hugo van Zyl: =P11/$N$9/Q11

Andre-Hugo van Zyl: Vlookup L from Pivot table $J$10:$L$18

Andre-Hugo van Zyl: Vlookup K from Pivot table $J$10:$L$18

Andre-Hugo van Zyl: =PIVOT TABLE (RUL!$B$10:$H$26064)



... Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX C: STATIC PRESSURE FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

	A	B	E	F	H
1		**Static pressure**			**15**
2		**Link Number**	**Sum of Pipe Length**	**Failure count**	**Avg Static Pressure Rounded**
3		21514	5	0	15
4		21518	5	0	15
5		21695	20	0	15
6		21696	40	0	30
7		21741	30	0	15
8		21863	10	0	30
9		21874	35	0	15
10		21884	65	0	15
11		21892	15	0	15
12		21923	15	0	15
13		22216	15	0	15
14		22241	35	0	15
15		22284	70	0	15
16		22292	10	0	15
17		22293	65	0	15
18		22294	5	0	15
19		22301	30	0	15
20		22302	25	0	15
21		22303	5	0	15
22		22312	5	0	15
23		22313	90	0	15
24		22316	5	0	15
25		22317	90	0	15
26		22321	5	0	15
27		22323	5	0	15
28		22324	5	0	15
29		22326	5	0	15
30		22327	15	0	15
31		22330	175	0	15
32		22331	10	0	15
33		22332	50	0	15
34		22333	35	0	15

	J	K	L
2	**Row Labels**	**Sum of Failure count**	**Sum of Sum of Pipe Length**
3	0	0	45
4	15	1	11185
5	30	42	27440
6	45	734	150780
7	60	1824	317800
8	75	3102	500555
9	90	7099	1013545
10	**Grand Total**	**12802**	**2021350**

	N	O	P	Q	R
2	**15**	**Static pressure**	**Failure Frequency (Failures/Year/Meter pipe)**	**Failure Count**	**Pipe Length**
3		0	0.00000	0	45
4		15	0.00001	1	11185
5		30	0.00010	42	27440
6		45	0.00032	734	150780
7		60	0.00038	1824	317800
8		75	0.00041	3102	500555
9		90	0.00047	7099	1013545
10		**Grand Total**		**12802**	**2021350**

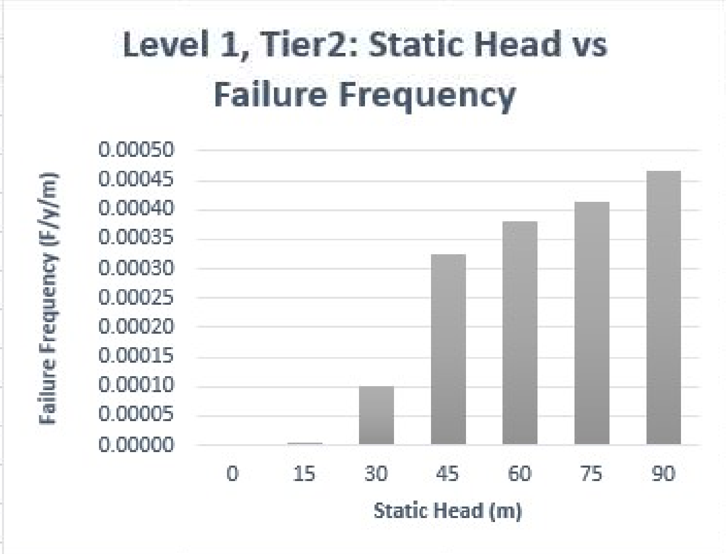
Andre-Hugo van Zyl:
Failure data set period in years

Andre-Hugo van Zyl:
=Q3/R3/$N$2

Andre-Hugo van Zyl:
Vlookup L from Pivot table $J$2:$L$10

Andre-Hugo van Zyl:
Vlookup K from Pivot table $J$2:$L$10

Andre-Hugo van Zyl:
=PIVOT TABLE (ZONE!$B$2:$H$25882)



System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | **ZONE** | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX D: RESIDUAL PRESSURE FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

Residual Pressure 15

Link Number	Sum of Pipe Length	Failure count	Avg Residual Pressure Rounded
21514	5	0	15
21518	5	0	15
21695	20	0	15
21696	40	0	15
21741	30	0	15
21863	10	0	30
21874	35	0	15
21884	65	0	15
21892	15	0	15
21923	15	0	0
22216	15	0	15
22241	35	0	15
22284	70	0	15
22292	10	0	15
22293	65	0	0
22294	5	0	15
22301	30	0	15
22302	25	0	15
22303	5	0	15
22312	5	0	15
22313	90	0	15
22316	5	0	15
22317	90	0	15
22321	5	0	15
22323	5	0	15
22324	5	0	15
22326	5	0	15
22327	15	0	0
22330	175	0	15
22331	10	0	15
22332	50	0	15

Row Labels	Sum of Failure count	Sum of Sum of Pipe Length2
0	0	2750
15	7	15120
30	82	36240
45	835	176270
60	2059	347050
75	3328	524865
90	6491	919055
Grand Total	**12802**	**2021350**

Andre-Hugo van Zyl: PIVOT TABLE (AADD!$B$2:$H$25882)

15

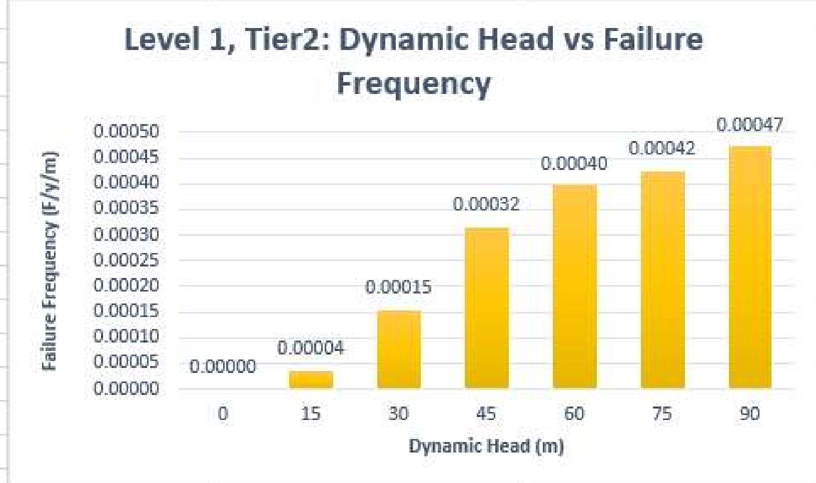
Residual Pressure	Failure Frequency (Failures/Year/Meter pipe)	Failure Count	Pipe Length
0	0.00000	0	2750
15	0.00004	8	15120
30	0.00015	84	36240
45	0.00032	835	176270
60	0.00040	2061	347050
75	0.00042	3335	524865
90	0.00047	6510	919055
Grand Total		**12802**	**2021350**

Andre-Hugo van Zyl: Failure data set period in years

Andre-Hugo van Zyl: =P3/Q3/$L$2

Andre-Hugo van Zyl: Vlookup L from Pivot table $J$2:$L$10

Andre-Hugo van Zyl: Vlookup K from Pivot table $J$2:$L$10



System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX E: RESERVE PRESSURE RATIO FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

Reserve Pressure Ratio 0.2

Link Number	Pipe Length	Sum of ResPrRatio	Failure count	ResPRRatio Roundup
22374	15	0	0	0
24176	20	0	0	0
25369	10	0	0	0
21514	5	0.0328399	0	0.2
21518	5	0.0363137	0	0.2
21741	30	0.0367722	0	0.2
21884	65	0.0109462	0	0.2
21892	15	0.0211863	0	0.2
21923	15	0.0073595	0	0.2
22284	70	0.0060494	0	0.2
22292	10	0.0068067	0	0.2
22293	65	0.002719	0	0.2
22294	5	0.0085229	0	0.2
22301	30	0.0212833	0	0.2
22302	25	0.0150131	0	0.2
22303	5	0.0144575	0	0.2
22312	5	0.0167124	0	0.2
22313	90	0.0231709	0	0.2
22316	5	0.0156242	0	0.2
22317	90	0.0226994	0	0.2
22321	5	0.0175806	0	0.2
22323	5	0.0146895	0	0.2
22324	5	0.0140583	0	0.2
22326	5	0.0159667	0	0.2
22327	15	0.0041765	0	0.2
22331	10	0.0149442	0	0.2
22332	50	0.0113007	0	0.2
22333	35	0.0098039	0	0.2
22491	5	0.0081726	0	0.2
22492	5	0.0078477	0	0.2
22493	5	0.0083756	0	0.2

Andre-Hugo van Zyl: PIVOT TABLE (PRESSURE_RATIO!$B$2:$G$25882)

Row Labels	Sum of Failure count	Pipe Length
0	0	45
0.2	100	69920
0.4	676	228360
0.6	2481	494150
0.8	3996	630930
1	5549	597945
Grand Total	12802	2021350

15

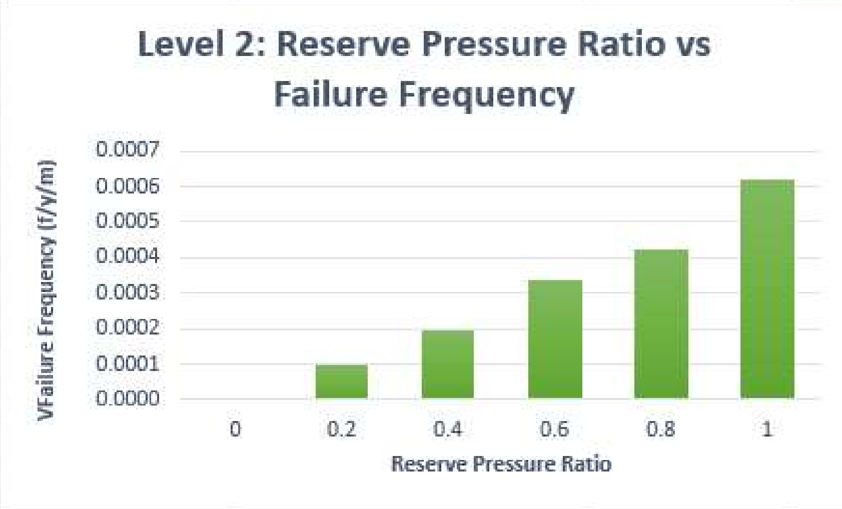
Andre-Hugo van Zyl: Failure data set period in years

Andre-Hugo van Zyl: =P3/Q3/$L$2

Andre-Hugo van Zyl: Vlookup K from Pivot table $I$2:$K$9

Andre-Hugo van Zyl: Vlookup J from Pivot table $I$2:$K$9

Reserve Pressure Ratio	Failure Frequency (Failures/Year/Meter pipe)	Failure count	Pipe Length
0	0.0000	0	45
0.2	0.0001	103	69920
0.4	0.0002	676	228360
0.6	0.0003	2485	494150
0.8	0.0004	4006	630930
1	0.0006	5563	597945
Grand Total		12802	2021350



System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX F: DIAMETER FAILURE FREQUENCY SOURCE CODE CALCULATIONS EXAMPLE

Diameter 100

Link number	Pipe Length	Diameter Rounded	Failure Count
316902	145	100	0
317439	50	100	0
338951	40	100	0
339475	55	100	0
339737	80	100	0
339995	70	100	0
340053	70	100	0
340105	100	100	0
340117	45	100	0
340815	60	100	0
366543	70	100	0
366544	70	100	0
303068	15	100	0
304212	30	100	0
304431	15	100	0
308802	70	100	0
308813	35	100	0
312847	140	100	0
314495	90	100	0
314678	55	100	0
314739	20	100	0
314747	100	100	0
314804	115	100	0
314883	5	100	0
314907	170	100	0
314976	80	100	0
314977	160	100	0
315110	20	100	0
315199	90	100	0
317200	35	100	0
335204	5	100	0
335205	70	100	0

Row Labels	Sum of Failure Count	Sum of Pipe Length
100	8142	770380
200	4143	972655
300	310	160485
400	120	65600
500	63	19515
600	12	12330
700	9	12675
800	1	1670
900	2	6040
Grand Total	12802	2021350

Andre-Hugo van Zyl:
PIVOT TABLE (DIAMETER!$B$2:$F$25882)

Andre-Hugo van Zyl:
Failure data set period in years

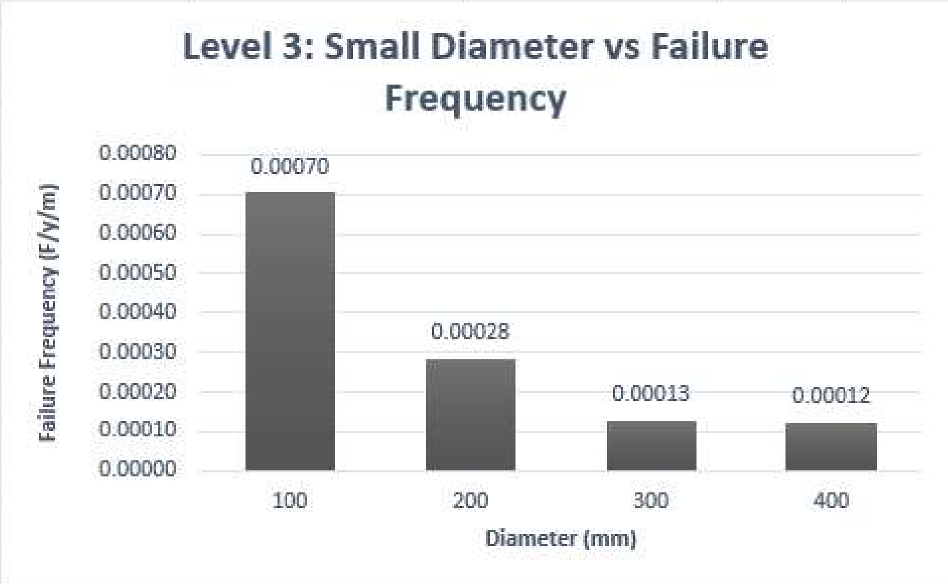
15

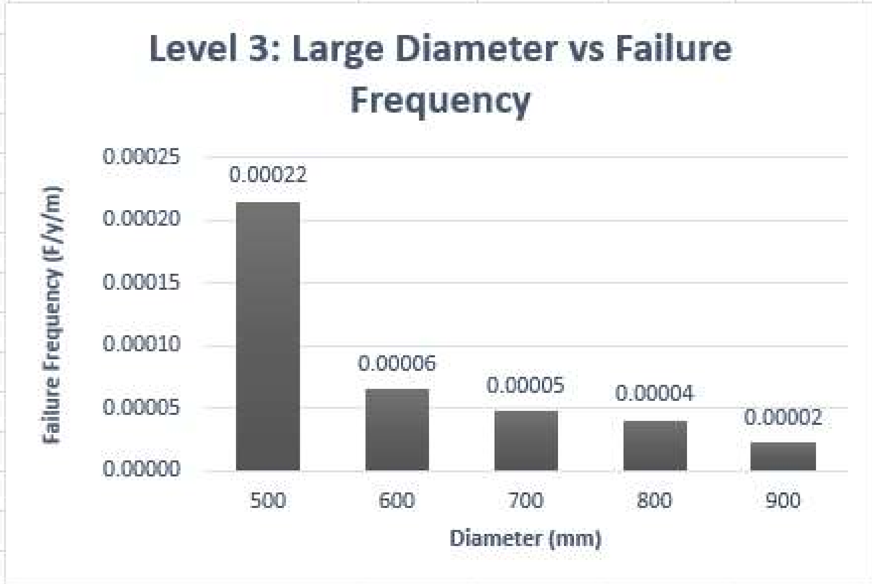
Diameter	Failure Frequency (Failures/Year/Meter pipe)	Failure Count	Pipe Length
100	0.00070	8142	770380
200	0.00028	4143	972655
300	0.00013	310	160485
400	0.00012	120	65600
500	0.00022	63	19515
600	0.00006	12	12330
700	0.00005	9	12675
800	0.00004	1	1670
900	0.00002	2	6040
Grand Total		12802	2021350

Andre-Hugo van Zyl:
=N3/$K$2/O3

Andre-Hugo van Zyl:
Vlookup J from Pivot table $H$2:$J$12

Andre-Hugo van Zyl:
Vlookup I from Pivot table $H$2:$J$12





System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX G: PIPE REPLACEMENT PRIORITISATION ALGORITHM SOURCE CODE EXAMPLE

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X
1	Zone	User Link No	Pipe Length	Pressure Rating	Installation Year	Material	Material FF	RUL Rounded	RUL FF	Level1_1 FF_Index	Static Pressure Rounded	Static Pressure FF	Residual Pressure Rounded	Residual Pressure FF	Level1_2 FF Index	Level_1 FF_Index	Reserve Pressure Ratio Rounded	Reserve Pressure Ratio FF	Level_2 FF_Index	Diameter Rounded	Diameter_FF	Level_3 FF_Index	Final FF_Index	New Failure Count
2	Findlay reservoir	349813	155	1800	1997	AC	0.00062	########	0.0004	0.00017	45.00000	0.00032	45.00000	0.00032	0.00011	0.00028	0.40000	0.00020	0.00003	200	0.00028	0.00005	0.00036	0.83005
3	Moreleta reservoir	[illegible]	[illegible]	[illegible]	[illegible]	AC	0.00062	0.00000	0.0007	0.00022	90.00000	0.00047	90.00000	0.00047	0.00016	0.00037	0.60000	0.00034	0.00006	200	0.00028	0.00005	0.00048	1.10825
4	Findlay reservoir	[illegible]	[illegible]	[illegible]	[illegible]	STEEL	0.00012	########	0.0001	0.00003	45.00000	0.00032	45.00000	0.00032	0.00011	0.00014	0.20000	0.00010	0.00002	500	0.00022	0.00004	0.00019	0.55850
5	Waterkloof reservoir	[illegible]	[illegible]	[illegible]	[illegible]	UPVC	0.00024	########	0.0002	0.00008	60.00000	0.00038	60.00000	0.00040	0.00013	0.00021	0.60000	0.00034	0.00006	200	0.00028	0.00005	0.00031	0.61176
6	Muckleneuck reservoir	[illegible]	[illegible]	[illegible]	[illegible]	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	0.80000	0.00042	0.00007	200	0.00028	0.00005	0.00036	0.85239
7	Hercules East BPT	[illegible]	[illegible]	[illegible]	[illegible]	AC	0.00062	########	0.0004	0.00017	75.00000	0.00041	75.00000	0.00042	0.00014	0.00031	0.80000	0.00042	0.00007	100	0.00070	0.00012	0.00050	0.78357
8	Koedoesnek HL reservoir	300721	100	900	1986	AC	0.00062	########	0.0005	0.00019	60.00000	0.00038	60.00000	0.00040	0.00013	0.00032	0.80000	0.00042	0.00007	100	0.00070	0.00012	0.00051	0.75991
9	Lynnwood reservoir	[illegible]	[illegible]	[illegible]	1963	AC	0.00062	0.00000	0.0007	0.00022	90.00000	0.00047	90.00000	0.00047	0.00016	0.00037	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00059	0.53484
10	Hercules East BPT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	########	0.0004	0.00017	90.00000	0.00047	90.00000	0.00047	0.00016	0.00033	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00055	0.49266
11	Moreleta reservoir	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	########	0.0004	0.00011	90.00000	0.00047	90.00000	0.00047	0.00016	0.00026	0.80000	0.00042	0.00007	200	0.00028	0.00005	0.00038	1.66016
12	Queenswood reservoir	[illegible]	[illegible]	[illegible]	1950	AC	0.00062	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.00016	0.00037	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00059	0.62398
13	Hospital reservoir	[illegible]	[illegible]	[illegible]	1960	AC	0.00062	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.00013	0.00035	0.60000	0.00034	0.00006	100	0.00070	0.00012	0.00052	0.62400
14	Waverley LL reservoir	308671	100	900	1904	AC	0.00062	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.00011	0.00032	0.60000	0.00034	0.00006	100	0.00070	0.00012	0.00050	0.74545
15	Salvokop reservoir	343219	75	1200	2006	UPVC	0.00024	########	0.0002	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	200	0.00028	[illegible]	[illegible]	0.39044
16	Meintjieskop reservoir	313517	100	1200	2005	UPVC	0.00024	########	0.0002	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	200	0.00028	[illegible]	[illegible]	0.47058
17	Salvokop reservoir	343713	75	1200	2004	UPVC	0.00024	########	0.0002	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.80000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	Constantia Park reservoir	325456	165	900	1985	AC	0.00062	########	0.0005	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.00000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
19	Meintjieskop reservoir	313248	145	900	2004	AC	0.00062	########	0.0004	0.00017	75.00000	0.00041	[illegible]	[illegible]	[illegible]	[illegible]	0.80000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	Magalies reservoir	346492	60	1200	2004	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	[illegible]	[illegible]	[illegible]	0.00024	0.80000	0.00042	0.00007	200	[illegible]	[illegible]	[illegible]	[illegible]
21	Meintjieskop reservoir	313047	90	1200	2005	UPVC	0.00024	########	0.0002	0.00008	75.00000	0.00041	75.00000	0.00042	0.00014	0.00022	0.60000	0.00034	0.00006	200	[illegible]	[illegible]	[illegible]	[illegible]
22	Kilner Park reservoir	300064	155	900	1988	AC	0.00062	########	0.0005	0.00019	60.00000	0.00038	60.00000	0.00040	0.00013	0.00032	0.60000	0.00034	0.00006	100	[illegible]	[illegible]	[illegible]	[illegible]
23	Murrayfield reservoir	317993	35	900	1970	AC	0.00062	0.00000	0.0007	0.00022	60.00000	0.00038	60.00000	0.00040	0.00013	0.00035	0.80000	0.00042	0.00007	100	[illegible]	[illegible]	[illegible]	[illegible]
24	Magalies reservoir	345796	100	1200	2003	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	1.00000	0.00062	0.00010	200	0.00028	0.00005	0.00039	0.58198
25	Heights HL Direct 3	341656	60	1800	2006	AC	0.00062	########	0.0002	0.00014	60.00000	0.00038	60.00000	0.00040	0.00013	0.00027	0.40000	0.00020	0.00003	200	0.00028	0.00005	0.00035	0.31821
26	Waterkloof reservoir	328653	55	1200	1949	AC	0.00062	0.00000	0.0007	0.00022	75.00000	0.00041	75.00000	0.00042	0.00014	0.00036	0.60000	0.00034	0.00006	200	0.00028	0.00005	0.00046	0.37917
27	Hercules East BPT	340284	105	1800	2004	AC	0.00062	########	0.0004	0.00017	90.00000	0.00047	90.00000	0.00047	0.00016	0.00033	0.60000	0.00034	0.00006	200	0.00028	0.00005	0.00043	0.67693
28	Salvokop reservoir	343007	5	1200	2008	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	1.00000	0.00062	0.00010	200	0.00028	0.00005	0.00039	0.02910
29	Muckleneuck reservoir	313942	120	1200	1988	HDPE	0.00031	########	0.0002	0.00009	90.00000	0.00047	90.00000	0.00047	0.00016	0.00025	0.80000	0.00042	0.00007	100	0.00070	0.00012	0.00044	0.78610
30	Magalies reservoir	345803	10	1200	2004	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	1.00000	0.00062	0.00010	200	0.00028	0.00005	0.00039	0.05820
31	Carina reservoir	323963	115	900	1949	AC	0.00062	0.00000	0.0007	0.00022	90.00000	0.00047	90.00000	0.00047	0.00016	0.00037	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00059	1.02511
32	Salvokop reservoir	343907	90	1200	2004	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	0.80000	0.00042	0.00007	300	0.00013	0.00002	0.00033	0.44455
33	Hercules East reservoir	338791	80	900	1940	AC	0.00062	0.00000	0.0007	0.00022	90.00000	0.00047	90.00000	0.00047	0.00016	0.00037	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00059	0.71312
34	Heights HL Direct 3	341913	60	1200	2008	UPVC	0.00024	########	0.0002	0.00008	75.00000	0.00041	75.00000	0.00042	0.00014	0.00022	0.80000	0.00042	0.00007	200	0.00028	0.00005	0.00034	0.30428
35	Hercules West reservoir	366396	95	1200	2007	UPVC	0.00024	########	0.0002	0.00008	75.00000	0.00041	75.00000	0.00042	0.00014	0.00022	0.60000	0.00034	0.00006	200	0.00028	0.00005	0.00032	0.46088
36	Muckleneuck reservoir	314360	90	1200	2007	UPVC	0.00024	########	0.0002	0.00008	90.00000	0.00047	90.00000	0.00047	0.00016	0.00024	0.80000	0.00042	0.00007	400	0.00012	0.00002	0.00033	0.44302
37	Magalies reservoir	344551	60	1200	2000	UPVC	0.00024	########	0.0002	0.00008	45.00000	0.00032	45.00000	0.00032	0.00011	0.00019	0.40000	0.00020	0.00003	200	0.00028	0.00005	0.00027	0.24093
38	Kilner Park reservoir	300043	90	900	1988	AC	0.00062	########	0.0005	0.00019	60.00000	0.00038	60.00000	0.00040	0.00013	0.00032	0.60000	0.00034	0.00006	100	0.00070	0.00012	0.00049	0.66411
39	Carina reservoir	323922	225	900	1949	AC	0.00062	0.00000	0.0007	0.00022	90.00000	0.00047	90.00000	0.00047	0.00016	0.00037	1.00000	0.00062	0.00010	100	0.00070	0.00012	0.00059	2.00565

Andre-Hugo van Zyl:
=VLOOKUP(F2,MATERIAL!$M$9:$P$15,4,FALSE)

Andre-Hugo van Zyl:
=VLOOKUP($B2,RUL!$C$10:$I$26064,6,FALSE)

Andre-Hugo van Zyl:
=IF(H2=RUL!$N$11,RUL!$O$11,IF(H2=RUL!$N$12,RUL!$O$12,IF(H2=RUL!$N$13,RUL!$O$13,IF(H2=RUL!$N$14,RUL!$O$14,IF(H2=RUL!$N$15,RUL!$O$15,RUL!$O$16)))))

Andre-Hugo van Zyl:
=(G2+I2)/6

Andre-Hugo van Zyl:
=VLOOKUP($B2,ZONE!$B$2:$H$26056,7,FALSE)

Andre-Hugo van Zyl:
=IF(K2=ZONE!$O$3,ZONE!$P$3,IF(K2=ZONE!$O$4,ZONE!$P$4,IF(K2=ZONE!$O$5,ZONE!$P$5,IF(K2=ZONE!$O$6,ZONE!$P$6,IF(K2=ZONE!$O$7,ZONE!$P$7,IF(K2=ZONE!$O$8,ZONE!$P$8,ZONE!$P$9))))))

Andre-Hugo van Zyl:
=VLOOKUP($B2,AADD!$B$2:$H$25882,7,FALSE)

Andre-Hugo van Zyl:
=IF(M2=AADD!$N$3,AADD!$O$3,IF(M2=AADD!$N$4,AADD!$O$4,IF(M2=AADD!$N$5,AADD!$O$5,IF(M2=AADD!$N$6,AADD!$O$6,IF(M2=AADD!$N$7,AADD!$O$7,IF(M2=AADD!$N$8,AADD!$O$8,AADD!$O$9))))))

Andre-Hugo van Zyl:
=(L2+N2)/6

Andre-Hugo van Zyl:
=O2+J2

Andre-Hugo van Zyl:
=VLOOKUP($B2,PRESSURE_RATIO!$B$2:$G$25882,6,FALSE)

Andre-Hugo van Zyl:
=IF(Q2=PRESSURE_RATIO!$N$3,PRESSURE_RATIO!$O$3,IF(Q2=PRESSURE_RATIO!$N$4,PRESSURE_RATIO!$O$4,IF(Q2=PRESSURE_RATIO!$N$5,PRESSURE_RATIO!$O$5,IF(Q2=PRESSURE_RATIO!$N$6,PRESSURE_RATIO!$O$6,IF(Q2=PRESSURE_RATIO!$N$7,PRESSURE_RATIO!$O$7,PRESSURE_RATIO!$O$8)))))

Andre-Hugo van Zyl:
=R2/6

Andre-Hugo van Zyl:
=W2*15*C2

Andre-Hugo van Zyl:
=V2+S2+P2

Andre-Hugo van Zyl:
=U2/6

Andre-Hugo van Zyl:
=IF(T2=DIAMETER!$L$3,DIAMETER!$M$3,IF(T2=DIAMETER!$L$4,DIAMETER!$M$4,IF(T2=DIAMETER!$L$5,DIAMETER!$M$5,IF(T2=DIAMETER!$L$6,DIAMETER!$M$6,IF(T2=DIAMETER!$L$7,DIAMETER!$M$7,IF(T2=DIAMETER!$L$8,DIAMETER!$M$8,IF(T2=DIAMETER!$L$9,DIAMETER!$M$9,DIAMETER!$M$10)))))))

Andre-Hugo van Zyl:
=VLOOKUP($B2,DIAMETER!$B$2:$F$25882,4,FALSE)

... | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX H: WATER HYDRAULIC COMPUTER MODEL PIPE DATA EXAMPLE

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
1	User Link Number	Zone	Link Type	Begin Node Number	End Node Number	Pipe Length (m)	Begin Static Head (m)	End Static Head (m)	Begin Residual Head (m)	End Residual Head (m)	Material	Pressure Rating (m)	Nominal Diameter (mm)	Year	RUL Ceiling (Year)	Avg Static Head Ceiling (m)	Avg Residual Head Ceiling (m)	Reserve Pressure Ratio Ceiling	Diameter Ceiling (mm)
2	23352	Carina booster	PIPE	23346	23359	5	65	65	65	65	STEEL	638	150	1992	0	75	75	0.2	200
3	23355	Carina booster	PIPE	23359	23369	5	65	65	65	65	AC	180	300	1992	0	75	75	0.4	300
4	23369	Carina booster	PIPE	23369	23352	25	65	65	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	75	75	0.4	300
5	23379	Carina booster	PIPE	23379	23369	5	85	65	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	75	75	0.2	200
6	26754	Carina booster	PIPE	26755	26756	5	85	85	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	90	90	0.2	200
7	26755	Carina booster	PIPE	26757	26758	5	85	85	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	200
8	26756	Carina booster	PIPE	26759	26760	5	85	85	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	200
9	26763	Carina booster	PIPE	26765	26766	5	85	85	86	86	STEEL	743	150	2011	[illegible]	[illegible]	[illegible]	[illegible]	200
10	26767	Carina booster	PIPE	26769	26770	5	85	85	86	86	STEEL	743	150	2011	0	90	[illegible]	[illegible]	[illegible]
11	26769	Carina booster	PIPE	26771	26772	5	85	85	86	86	STEEL	743	150	2011	0	90	[illegible]	[illegible]	[illegible]
12	26772	Carina booster	PIPE	26766	26770	5	85	85	86	86	STEEL	743	200	2011	0	90	[illegible]	[illegible]	[illegible]
13	26775	Carina booster	PIPE	26770	26772	5	85	85	86	86	STEEL	743	200	2011	0	90	90	0.2	200
14	26777	Carina booster	PIPE	26772	23358	5	85	85	86	86	STEEL	743	200	2011	0	90	90	0.2	200
15	324098	Carina booster	PIPE	324135	324074	200	95	101	95	101	UPVC	120	110	1992	0	105	105	1	200
16	324116	Carina booster	PIPE	324161	324142	160	100	96	100	96	UPVC	120	110	1992	0	105	105	1	200
17	324126	Carina booster	PIPE	324116	324177	160	94	77	94	77	UPVC	120	110	1992	0	90	90	0.8	200
18	324127	Carina booster	PIPE	324134	324125	185	80	85	80	85	UPVC	120	110	1992	0	90	90	0.8	200
19	324133	Carina booster	PIPE	324153	324156	215	96	87	96	87	UPVC	120	110	1992	0	105	105	0.8	200
20	324134	Carina booster	PIPE	324144	324134	55	80	80	80	80	UPVC	120	110	1992	0	90	90	0.8	200
21	324135	Carina booster	PIPE	324135	324136	150	95	101	95	101	UPVC	120	110	1992	0	105	105	1	200
22	324136	Carina booster	PIPE	324148	324135	105	88	95	88	95	UPVC	120	110	1992	0	105	105	0.8	200
23	324141	Carina booster	PIPE	324136	324141	145	101	109	101	109	UPVC	120	110	1992	0	105	105	1	200
24	324142	Carina booster	PIPE	324142	324153	5	96	96	96	96	UPVC	120	110	1992	0	105	105	0.8	200
25	324144	Carina booster	PIPE	324144	324146	60	80	75	80	75	UPVC	120	110	1992	0	90	90	0.8	200
26	324145	Carina booster	PIPE	324146	324165	55	75	71	75	71	UPVC	120	110	1992	0	75	75	0.8	200
27	324146	Carina booster	PIPE	324147	324146	25	75	75	75	75	AC	90	100	1941	45	75	75	1	100
28	324148	Carina booster	PIPE	324149	324148	140	82	88	82	88	AC	90	75	1992	0	90	90	1	100
29	324149	Carina booster	PIPE	324253	324149	185	76	82	76	82	AC	90	75	1992	0	90	90	1	100
30	324151	Carina booster	PIPE	324151	324148	30	87	88	87	88	UPVC	120	110	1992	0	90	90	0.8	200
31	324153	Carina booster	PIPE	324153	324071	250	96	95	96	95	UPVC	120	110	1992	0	105	105	0.8	200
32	324155	Carina booster	PIPE	324157	324156	160	79	87	79	87	UPVC	120	110	1992	0	90	90	0.8	200
33	324156	Carina booster	PIPE	324151	324156	5	87	87	87	87	UPVC	120	110	1992	0	90	90	0.8	200
34	324157	Carina booster	PIPE	324187	324151	50	84	87	84	87	UPVC	120	110	1992	0	90	90	0.8	200
35	324164	Carina booster	PIPE	324177	324157	150	77	79	77	79	UPVC	120	110	1992	0	90	90	0.8	200
36	324165	Carina booster	PIPE	324165	324186	165	71	69	71	69	UPVC	120	110	1992	0	75	75	0.6	200
37	324166	Carina booster	PIPE	324168	324165	30	71	71	71	71	AC	90	100	1941	45	75	75	0.8	100
38	324167	Carina booster	PIPE	324167	324168	35	72	71	72	71	UPVC	120	110	1987	0	75	75	0.6	200
39	324168	Carina booster	PIPE	324169	324167	115	75	72	75	72	UPVC	120	110	1987	0	75	75	0.8	200
40	324174	Carina booster	PIPE	324194	324136	140	100	101	100	101	UPVC	120	110	1992	0	105	105	1	200
41	324177	Carina booster	PIPE	324178	324177	75	76	77	76	77	UPVC	120	110	1992	0	90	90	0.8	200

Andre-Hugo van Zyl:
=IF(CEILING(2015-(N2+VLOOKUP(K2,$U$4:$V$9,2,FALSE)),15)<0,0,CEILING(2015-(N2+VLOOKUP(K2,$U$4:$V$9,2,FALSE)),15))

Andre-Hugo van Zyl:
=CEILING((G2+H2)/2,15)

Andre-Hugo van Zyl:
=CEILING((I2+J2)/2,15)

Andre-Hugo van Zyl:
=CEILING((((G2+H2)/2)/L2),0.2)

Andre-Hugo van Zyl:
=CEILING(M2,100)

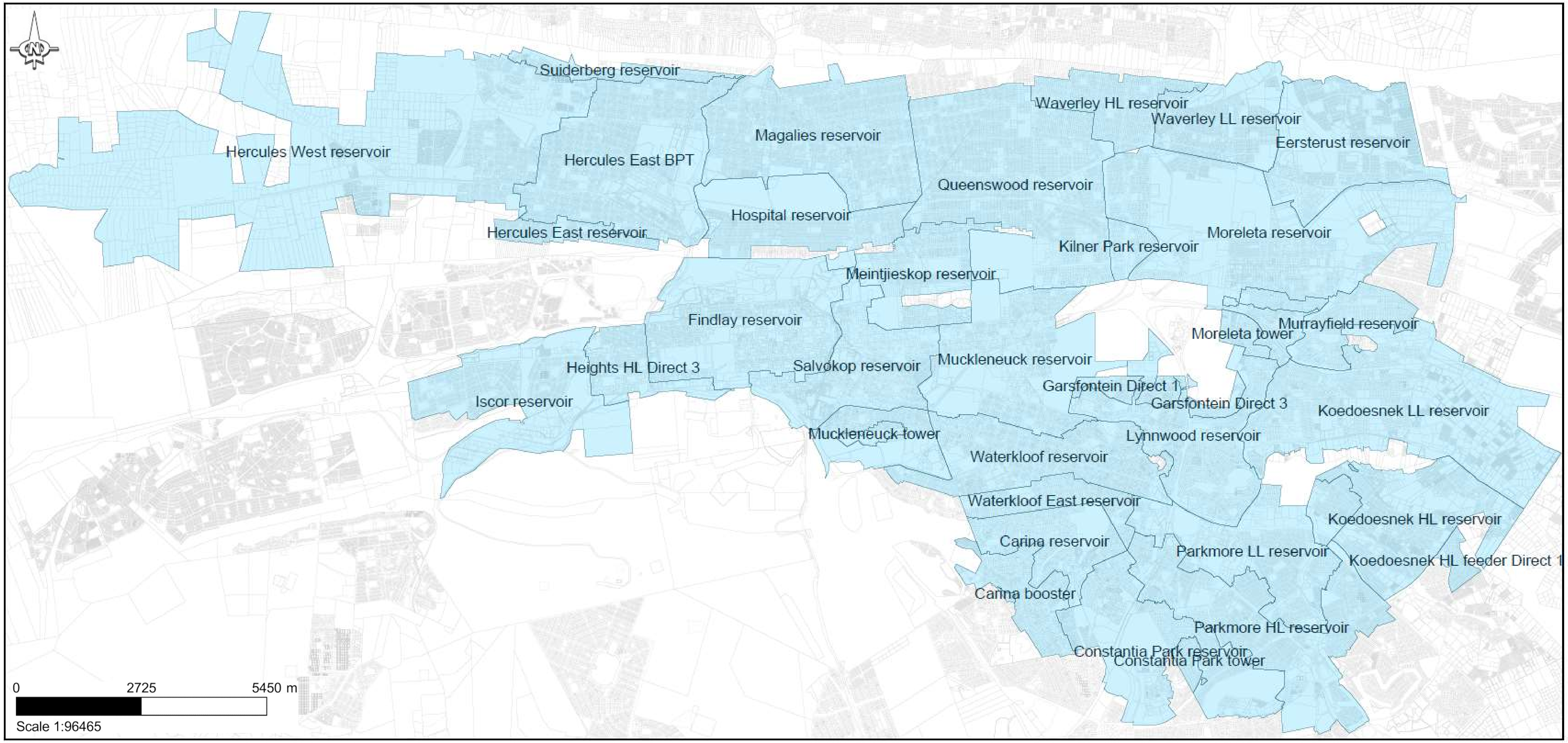
Remaining Usefull Life	
Material	RUL
AC	40
HDPE	80
MPVC	50
STEEL	60
UPVC	50

System_Original | System_Pivot | Failures Pivot | Failures_DB | MATERIAL | RUL | ZONE | AADD | PRESSURE_RATIO | DIAMETER | System_Calcs | Results | Verification | Final_Verification

APPENDIX I: INTEGRATED BUSINESS INFORMATION SYSTEM DATA EXAMPLE

STANDCODE	STEETCODE0	STREETCODE1	STREETCODE2	STREETNAME0	STREETNAME1	STREETNAME2	ACTIVITYCO	ACTIVITYDE	ACTIVITYUN	JOBCODE	TASKDATE	TASKREFERE
	028984	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-01	291337/1
	060064	079160	025792				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-01	291337/2
	005660	091788	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-01	291364/1
	071544	004820	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-01	291364/2
	016468	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-01	291364/3
	031112	000000	000000				509	LEAKS: REPAIR 125/150/200MM PIPE	NUMBER	AM9954	2000-01-01	291442/2
	076024	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-02	291328/10
	007256	088820	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-02	291338/3
	071544	078292	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-02	291342/1
	036348	078096	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-02	291367/1
	037552	000000	000000				509	LEAKS: REPAIR 125/150/200MM PIPE	NUMBER	AM9954	2000-01-02	291340/1
	076136	000000	000000				509	LEAKS: REPAIR 125/150/200MM PIPE	NUMBER	AM9954	2000-01-02	291340/3
	000000	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291330/1
	000000	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291330/2
	000000	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291329/7
	032568	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291339/4
	099999	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291339/5
	042956	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291368/2
	031112	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291409/1
	016468	072496	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291409/2
	034360	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291409/4
	011596	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-03	291415/3
	016468	036348	000000				509	LEAKS: REPAIR 125/150/200MM PIPE	NUMBER	AM9954	2000-01-03	291368/1
	094784	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-04	291438/1
	055976	024504	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9954	2000-01-04	291385/1
	002188	027696	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9955	2000-01-04	291391/4
	089310	031224	060064				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9955	2000-01-04	291391/5
	305939	086384	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9955	2000-01-04	291418/4
	031000	035984	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9955	2000-01-04	291427/2
	048248	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	AM9955	2000-01-04	291427/4
	308304	000000	000000				508	LEAKS: REPAIR 50/75/100MM PIPE	NUMBER	[OHNET	2000-01-04	390125/1

APPENDIX J: PRESSURE ZONES IN THE STUDY AREA



APPENDIX K: EXISTING WATER RETICULATION PIPES IN THE STUDY AREA