Hydraulic Performance Assessment of an Algorithmic Generated Simplified Foul Sewer Network

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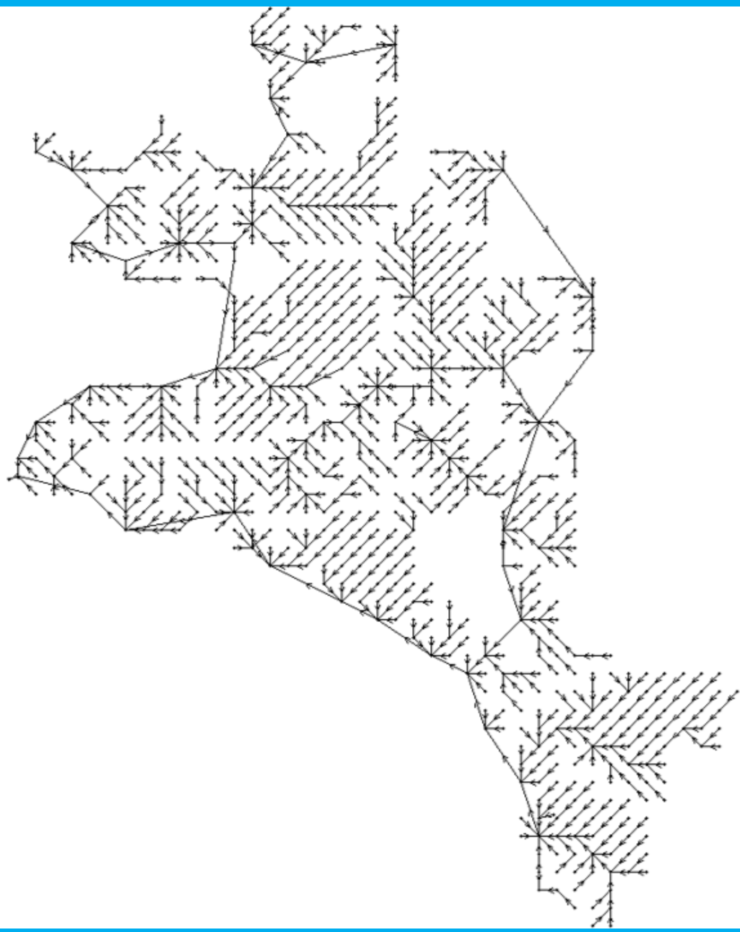

TU Delft

Hydraulic Performance Assessment of an Algorithmic Generated Simplified Foul Sewer Network

by

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Abstract

Foul sewer networks face many challenges related to new pressures and ageing infrastructure. There is a need to be able to evaluate how networks will be able to adapt to varying population densities, urban development and ecologic changes. Some suggest the use of exploratory models to test large numbers of network configurations, intended as alternative responses to the driving pressures. However, in order to carry out exploratory modelling, a trade-off between computational time and accuracy must be achieved. An approach to generate and size foul sewer networks which allow computational savings was developed in a collaboration between Eawag, ETH Zurich and TU Delft.This approach was used in this study to evaluate the computational time savings and accuracy of a generated sewer network in hydraulic performance assessment. Two case studies of $7km^2$ (Port Phillip) and $57km^2$ (Melbourne), and different land uses were used to evaluate the ability of the generated network to represent a real network.

It was found that computational time was reduced for both case studies, by a maximum factor of 10. Hydraulic performance was compared under high, typical and low flow conditions. It was found that high flow parameters are better represented in small case studies, where network capacity reduction and topology differences are less evident between real and generated networks. For low flow conditions, percentages of network length at risk of sedimentation were well represented for both case studies, with the larger case study showing slightly better performance. Under typical flow conditions, it was shown that topology simplification in the generated network leads to significant changes in times of concentration between networks, and hydrograph discrepancies, especially for the larger case study. However, general trends, including distribution of pipe cumulative flow percentage for pipes in sedimentation, or pipe diameter for pipe surcharge, are well represented by the generated network. Therefore, it is found that the generated network should be used to evaluate hydraulic performance trends in generated networks, as well as global values (eg. flood volume) if the network differences (eg. capacity, path length) are taken into account.

Finally, future work needs are highlighted which could strengthen the findings of this study, including use of real network flow and hydraulic performance data. Moreover, the computational time savings for a full assessment (rather than just hydraulic evaluations) should be quantified and compared.

Acknowledgments

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Contents

1
Introduction

1.1. Urban Water Management

Sewer networks of various types and sizes have been built around the world with the main objective of enhancing human hygiene and preventing floods. As a consequence of the economies of scale and perceived higher reliability of a central wastewater treatment plant,these networks have, in most cases, taken shape as centralised organised infrastructures [25]. Urban water management (UWM) issues (eg. supply, quality) have been typically dealt with purely technological adaptations without addressing demand management, public engagement or socio-economic structures as part of the solution [28]. As a result, the characteristics of the systems which have been developed until recent years do not reflect the current regard of water as a scarce resource which should be managed in a sustainable manner.

Recently, a variety of arguments have been made for challenging the assumption that a centralised piped sewer network should continue to be considered the paradigm of the 'normal' network. Literature research uncovers a trend towards advocating for change in UWM planning. Globally, some of the drivers for change in urban water management, and specifically sewer networks, cited include: (1) Rapid urban transformations, increasing the water demand density and heterogeneity calling for changes in the water supply, treatment and collection systems [20, 27]; (2)population growth, increasing overall demand [20, 23, 27]; (3)environmental concerns related to the impact of human development on the ecosystem both in terms of exhausting existing natural resources and the polluting impact of human activities [20, 23, 27, 28];(4) insufficient budgets, as most systems approach the end of their lifetime (typically below 100years), high maintenance and replacement costs create dis-economies of scale at the network level [20, 25];(5) climate change, which creates uncertainty in predicted water resources availability (for water supply) and in rainfall intensity and duration (for water collection) [27, 28]. Many suggest that higher network flexibility, sustainability and adaptability is needed to deal with challenges including climate change, demographic changes, and water scarcity which will have an uncertain effect on the existing systems. Due to these challenges, the environmental and water supply security aspects of networks have become influential design criteria [2, 12, 14, 27].

Thus, to cope with the existing challenges, alternative technical solutions aimed at recovering resources, reducing water consumption and creating flexible systems adaptable to boundary conditions (eg. demographic changes), have emerged. These have been discussed and to some extent integrated in policy, and implemented in practice.

Examples of implementations of these alternatives include Eawag in Zurich, Switzerland, where urine is collected by urine-diverting toilets and treated on-site for fertilizer production. In Hamburg, Germany blackwater (from toilets) and greywater (from sinks, showers, etc.) are collected at the building level and treated in decentralised treatment plants. Grey water, once treated, is reused on site, while heat is recovered from blackwater. Similarly in Beijing, China, blackwater and greywater are collected and treated on site for non-potable reuse, while rain water is collected for toilet flushing [18]. These schemes are enabled by technologies such as active carbon filters and nitrifying processes to convert urine into nutrients, heat pumps and exchangers enabling heat recovery from blackwater [38], and membrane bioreactors, gravel filters and heavy-metal specific adsorption materials to treat wastewater for re-usal at decentralised plants[4]. Additional technologies include rainwater harvesting tanks, low flush toilets or recirculating showers, which can be applied at any scale from single site solutions to city level schemes, effectively generating endless possibilities to chose from for designers and planners.

However, aside from pilot studies and small scale applications (such as those described in the previous paragraphs), practical implementations of alternative foul sewer systems, especially on greater scales, are scarce. In fact, despite the abundance of possible technical alternatives to the current system, current investment decisions by governments continue to show a preference for traditional technical solutions over emerging technologies[18]. This phenomenon is not uncommon to large socio-economic systems. Research a range of fields (from the agricultural to the automotive or telecommunications sector) has shown that social dependencies tend to lead to inertia and stability in decision making. Recent studies in UWM clearly show that although investment in research and pilot studies of new technologies are substantial, the implementation of such systems remains limited. This behavior, whereby legacy solutions continue to be preferentially implemented, is referred to as technological lock-in or entrapment and is mainly described as a socio-technical phenomenon driven by cognitive frames embedded in water management institutions [7].

While the social and behavioural barriers to implementation of new systems in UWM are undoubtedly recognized, the existence of 'conceptual weaknesses' in the implementation of alternative systems must also be addressed. Brown et al. (2011) highlights performance uncertainty as one major 'weakness' of alternative systems, as do other studies in the literature.Performance uncertainty creates a loop of inertia, whereby new technologies are not applied due to the lack of case studies and performance data, while performance data cannot be observed due to lack suitable of implementations[7].

1.2. Hydraulic Performance

In the context of planning, performance of sewer networks can be defined as the ability of sewer systems to meet certain objectives. The objectives of sewer asset management are multifaceted and have been formulated differently in a range of studies and by many organisations and authorities. The UNESCO and Global Water Partnership (GWP) also defined the goals of urban water management (less specific to sewers) as providing access to sanitation and protection against harmful effects of water, either physical (e.g., flooding), or chemico-biological (contamination of water resources by chemicals or micro-organisms) [5]. Marlow, Beale and Burn (2010) define the objective of sewer asset management specifically as "[...] Maximizing the value derived from an asset stock over a whole life cycle,within the context of delivering appropriate levels of service to customers, communities and the environment, and at an acceptable level of risk"[22]. From both the definitions, a focus on (1) service provision and (2) human and environmental health is highlighted. These objectives are however extremely broad and not directly quantifiable, therefore studies assessing the performance of sewer systems tend to rely on indicators to approximate the values of service provision and safety.

Based on these objectives, studies and guidelines providing frameworks to aid decision making, propose the use of performance indicators to inform decisions on a variety of aspects of sewer networks. Performance indicators are used in almost every field, their formulation and evaluation has been the object of many studies. Regarding indicators specific to UWM, a comprehensive guide on indicators for sewer networks is given by the International Water Association (IWA). The association outlines a strategy for selecting appropriate indicators based on the goals of the analysis, and calculating them to inform decision making and aid benchmarking in sewer asset management [10]. The IWA defines performance indicators (PIs) divided in categories aimed at encompassing the multi-faceted aspect of UWM, these include Environmental indicators (wEn), Personnel indicators (wPe), Physical indicators (wPh), Operational indicators (wOp),Quality of service indicators (wQs), and Economic and financial indicators (wFi) [24]. Each category consists of many sub indicators which should be chosen based on the objective of the analysis. These indicators cover a range of aspects, from social to economic, for the assessed network. Many of the indicators, however, focus on technical aspects of the network, including pumping consumption, flooding, sedimentation, or surcharge to be obtained from monitoring or modelling data.

Hydraulic performance is thus an important factor to accurately evaluate sewer network performance, as such hydraulic performance assessment has been the object of various studies.Other performance indicators have been formulated and used to guide strategic planning by comparing and classifying elements within the same network. Tagherouit el al. (2011) proposed an indicator of pipe surcharge impact to allow prioritisation in rehabilitating sewer pipes [34], El-Housni et al. (2019) used performance indicators to identify pipes vulnerable to the impacts of climate change [15], Cardoso et al. (2004) proposed performance functions related to basic network properties including water level and velocity to aid comparison between networks in different areas and for target monitoring [9]. According to Cardoso's approach, comparison of 3 sewer networks in terms of flow velocity and water level is carried out preserving the spatial variability of the network by using percentiles and weighted averages. The approach is strictly aimed at aiding technical decision making, concerned with issue detection, solutions and design for sewer networks. The Performance assessment system, referred to as PAS, is based on grading functions for water levels and velocity based on design parameters (eg. optimal filling ratio of 80% for the pipes, flow velocity between 0.6-3.0 m/s). Grades from 0-4 are then assigned to different values of velocity and water levels, these values are assigned to all pipe sections, and can be aggregated using pipe length as a weight factor for water level, and pipe volume as a weight factor for velocity. Vulnerability can also be considered where this is known for all sections. This methodology allows for the establishment of a weighted average for performance, and for the analysis of percentiles. The percentiles are drawn from the performance values from the entire network and represent the percentage of pipes multiplied by their weight, with or below a certain performance value.

The presented studies, however, show a trend whereby performance assessments focus on benchmarking few options based on existing network comparison or model comparison, which appear to implicitly lead to the choice of legacy decisions (rehabilitation, expansion etc.) and enhance lock-in effects.

1.3. Exploratory Modelling of Sewer Networks

Given the lack of sufficient existing networks using new technologies, modelling seems the most prominent choice to overcome the uncertainty barrier. This is more complex than for typical sewer systems due to the uncertainty in new technology performance, and the lack of historic knowledge of the large scale impacts of new system implementations, leading to the need to consider a much wider range of alternatives than has been done for typical sewer systems adaptation strategies. Moreover, uncertainty in the boundary conditions of sewer network modelling (population, climate change, resource availability), further enhances the complexity of the problem, creating what is referred to as a *wicked problem*.

Some have attempted at solving this problem by considering few manually generated alternatives to optimize storm water management sewers cost and environmental impact [39],or evaluate integrated water management systems with respect to cost and environmental impact [27]. However, the consideration of few possible solutions implies knowledge of boundary conditions.

To deal with uncertainty, in many fields, computational trials can be used to explore the impact of parametric and non-parametric uncertainties, testing system behaviour under different sets of assumptions. Exploratory modelling has to some extent been used in UWM to explore various water network layouts [14], to explore what-if scenarios to reflect the impacts of planning strategies, and to explore transitions for a sewer network [3].

From the review of existing modelling methods (exploratory or not), it can be concluded that most of the existing approaches are characterised by either a level of complexity which makes their utilization unfeasible and doesn't necessarily add more value to the planning process [14, 27, 32], or are not specific enough to be used in planning [32, 39]. Some of the computationally viable models reviewed, in fact, do not generate or size sewer infrastructure [27, 39], instead relying on hydraulic balance models and neglecting hydraulic performance despite its importance in planning and management. Baron's methodology allows exploration of system alternatives to take place, however the computational time was found to be high for even a small case study taking 4 hours for a 4km long sewer network [3]. Assuming a linear relation between network size and computational time, for a city-scale simulation of around 200km it can be expected that computational time will increase linearly or exponentially with case study size which would mean taking between weeks to months for larger case studies. In exploratory modelling, literature suggests that a number of alternatives in the order of 10^3 [11, 26], is typically considered. If the time taken for a single simulation is in the order of hours, a reduction of computational time in the order of 10 would mean a reduction from thousands of hours (months) to hundreds of hours (days). Hence, there is a need to reduce complexity and computational times while still ensuring the required level of accuracy is met. An effective model should allow exploratory modelling of extensive alternatives for sewer systems at various temporal and spatial scales such that performance data can be obtained to inform the decision making process [10].

Reducing computational times and complexity, at the expense of accuracy, is typically done through network simplification. Simplification can include either manual or automated approaches. Tested manual simplification methods in the literature include "storage-node simplification" where part of the network is substituted by a storage node which reflect the geometric characteristics and storage volume of the sewer system, or the "storage-pipe" simplification aimed at merging pipes depending on their slope and distance to the downstream node. Fischer et al. (2009) carried out a comparative case study analysis for such simplification techniques showing satisfactory results for water level, flow and outflow representation [16]. Rouault et al (2008) simplified network layout by grouping all pipes of a certain region into a single synthetic pipe representing the cumulative characteristics of the removed pipes[31]. Fully conceptual models have also been used for sewer representation by grouping sewer infrastructure data in single hydraulic and transport modules defined by sets of equations. This approach has been tested in the literature for a range of scopes, including predicting flooding and sedimentation [21]F, with successful results shown in the literature. However, the simplifications are typically only carried out in small portions of the network, and would require great analytic efforts to be amplified to larger scales.

To address this, a semi-automated simplification approach was proposed by Kroll et al. (2010), starting from a full model and eliminating side branches of the network[19] .However, similarly to all the reviewed methods, Kroll et al (2010)'s approach starts from in depth knowledge of the existing system, and the existence of accurate hydraulic models to drive the simplification which may not be available for many cases [6] or would require data gathering which might not be suitable for exploratory models.

In order to use modelling in evaluating large numbers of potential alternatives, the formulation of a model which enhances computational efficiency, and allows hydraulic performance evaluation, under low input data requirements is needed. To address this gap, the UrbanBEATS (Urban Biophysical Environments And Technologies Simulator)model was proposed. An urban form abstraction method was developed based on both conceptual and procedural approaches for planning purposes specifically.Compared to the previously described exploratory models,UrbanBEATS provides an accessible modelling framework, which is intended for planning support by allowing exploration of interactions between wastewater infrastructures [13]. A wastewater infrastructure generation module (separate from UrbanBEATS) has been developed (Duque et. al 2020) which can be used to delineate the network topology based on the water demand from UrbanBEATS. The final output is a simplified outline of the network which should be representative of a real network.

The final step to evaluate hydraulics in the generated foul network topology is the network dimensioning. This has been explored in a number of studies and optimized design algorithms have been proposed, however optimization tends to lead to increased computational times or highly abstract solutions. A compromise between abstract and complex solutions has been recently presented by the Pipe-by-pipe algorithm, which does not aim at optimizing pipe sizing with regards to a specific parameter but rather gives a feasible design, obtained from a step-by-step design of each network pipe from the extreme nodes towards the outfall[13]. While the set-up of the Pipe by Pipe algorithm is promising, it has not yet been validated in its ability to give an appropriate representation of a real sewer network.

The performance evaluation of exploratory models with regard to their ability to produce a computationally efficient, yet informative, representation of sewer systems is a topic which has not been treated extensively in literature. Some of the exploratory models defined in the previous paragraphs of this chapter ([27, 39]) rely on water balance models which do not allow hydraulic performance to be assessed. Other rely on virtual cases whose findings cannot be extrapolated to specific areas for decision making processes [32], or do not simplify the network layout to achieve computational feasibility [2]. In fact, to the author's knowledge, no large-scale algorithmic simplified sewer generation algorithm study provides a real network validation with regards to network topology and hydraulic performance. Yet, hydraulic performance not only drives sewer design and planning, but is also used to quantify pollutant loads or sedimentation which leads to the estimation of ecologic impacts and cost of sewer schemes [6], hence playing a key role in the evaluation of potential sewer systems alternatives. Hydraulic assessments have been carried out to estimate impacts of new technologies using full hydraulic models, but not for simplified networks. Hydraulic assessments for simplified networks have been carried out for simplified networks in cases where extensive input data is available, but not for algorithmic generated network under limited data availability.

Therefore, the assessment of an algorithmic generated simplified sewer network's ability to represent a real network, based on hydraulic performance indicators used in planning, is identified as the research gap area to be investigated. The final aim is to enable the use of exploratory modelling, in an attempt to overcome uncertainty barriers and avoid technological lock-in effects.

1.4. Objective

The objective of this study is to firstly determine hydraulic performance indicators of interest for planning and decision making. Secondly, to define a methodology for generated simplified network validation, considering a situation of limited data availability (which is common to many sewer systems [6]), and thirdly quantifying the hydraulic performance indicators based on the methodology and outputs defined.

In evaluating the outcome of the study, both accuracy and computational time of the proposed simplification will be taken into account, with particular focus on comparison with other simplification and sizing strategies (where data is available).

2
Research Purpose

Based on the research gap formulated in the previous section of this report, the overarching research question to be answered is reported below.

> Can a simplified, algorithmically generated, foul sewer network be used for the estimation of hydraulic performance behaviours and planning parameters of a real network, sized according to the same principles?

The overarching question can be divided into two main areas of analysis, (1) definition of hydraulic performance parameters and behaviours to be assessed, (2) comparison of the performance parameters and behaviours to assess the simplification's ability to represent the real network, and (3) the effect of various case study scales and simplification resolution on ability to represent the hydraulic performance parameters and behaviours. The three related sub-questions are reported below:

> 1. Which hydraulic performance behaviours and planning parameters should be used to assess the ability of an algorithmic generated simplified foul sewer to represent a real network designed based on the same principles?

Once the hydraulic performance indicators and behaviours to be assessed are defined, an evaluation and comparison method should be formulated.

> 2.How can the hydraulic performance parameters and behaviours evaluated for a simplified, algorithmic generated network, and a real network, be compared to assess the ability of the simplified network to represent the real network?

Finally, the indicators defined and evaluated can be used to make comparison between simplification scales and resolutions with respect to computational time and accuracy.

> 3.What is the impact of different network scales (eg. neighbourhood, city, etc.) and simplification resolution on the computational time savings and ability of the simplified network to represent the hydraulic performance parameters and behaviours of the real network?

3 Methodology

3.1. Overview

The overview of the methodology for the analysis followed in this research is given in figure 3.1.

Based on the research questions, the methodology aims at firstly defining the indicators chosen for the hydraulic performance assessment and an overview of the process followed to select and, where necessary, adapt them (1). This includes a brief description of the indicators of interest found in the literature, and their adaptation based on the availability of data and purpose of this research. The the indicators' units, scales and, where necessary, comparison method are also defined in the methodology. Secondly, the methodology outlines a framework to generate comparable hydraulic models for real and simplified algorithmic generated network for networks with limited data availability (2). This includes network inflows generation, network topology definition, hydraulic design, and hydraulic model parameters. Finally, the case study and their comparison method are described (3).

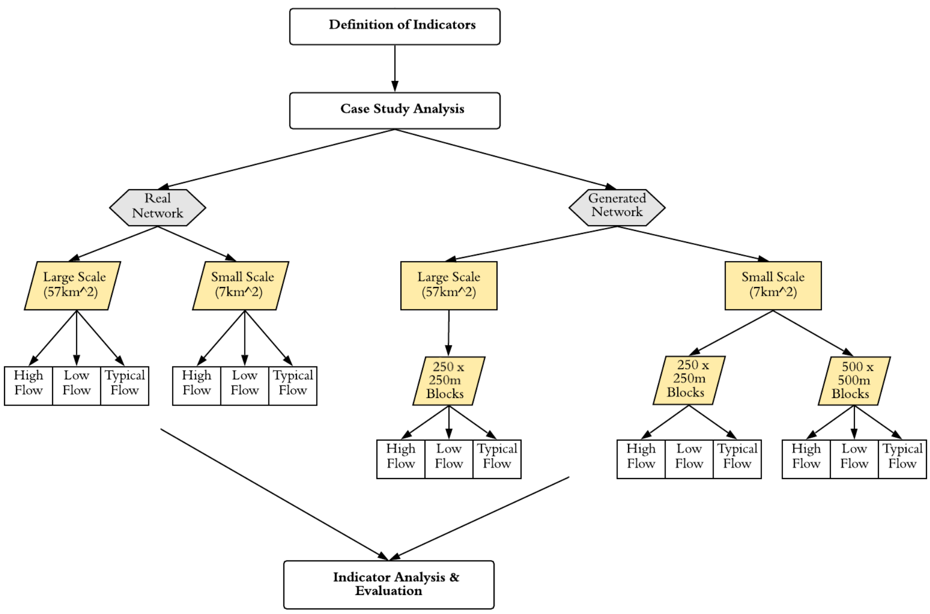

Figure 3.1: Methodology Overview for case study data generation to be used for comparison

3.1.1. Materials and Methods

In order to formulate the methodology the data available and the projected data requirements are taken into account. The methodology aims at filling the gap between the data available and the data requirements for an hydraulic performance assessment.

Data Available	Source
Drawing of Real Network Topology	State Government of Victoria repository (2020)
Case Study Population	Australian Bureau of Statistics (2020)
Case Study Topography	State Government of Victoria (2020)
Case Study Land Use	State Government of Victoria repository (2020)

Table 3.1: Data available for the hydraulic performance assessment

The data requirements, on the other hand, were assessed based on hydraulic performance assessments reviewed in the literature [9, 16, 19, 31], and include the following:

Data Required
Corrected Real Network Topology
Real network sizing and elevations
Generated network topology
Generated network sizing and elevations
Generated and Real Network flows
Land use wastewater patterns

Table 3.2: Basic data requirements for hydraulic performance assessments

The methods and materials needed to obtain the required data from the available inputs and carry out the hydraulic performance assessment is summarised below.

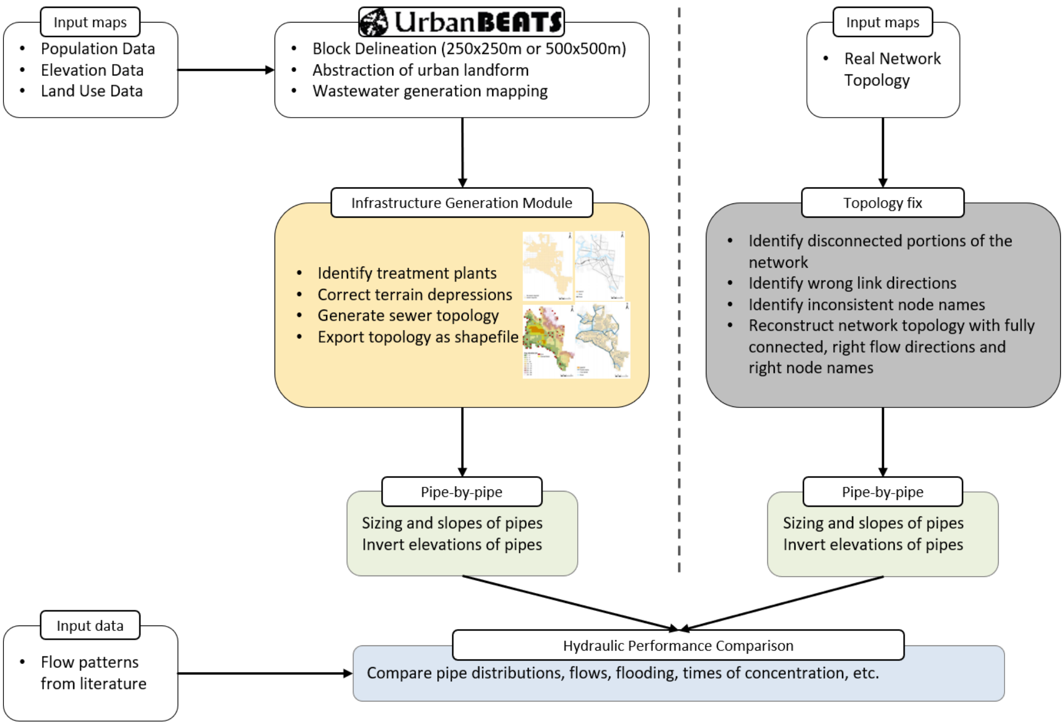

Figure 3.2: Methodology for the generation of the hydraulic model for the generated (in yellow) and real (in grey) networks.

3.2. Performance Indicators

Based on the literature review, hydraulic performance performance indicators were selected to eventually compare the real and generated networks. The selection of indicators followed the steps outlined in the literature [9, 24], which specify that firstly, the goals of the analysis should be defined. The goal of the analysis is defined by the research question as evaluating whether a simplified, algorithmic generated foul sewer network can be used for the estimation of hydraulic performance behaviors and planning parameters of a real network. Thus, the evaluation of hydraulic performance behaviors and planning parameters to compare two networks is found to be the goal of the performance assessment. The hydraulic performance behaviours indicators to assess were defined based on a combination of literature review and discussion with experts, while planning parameters indicators were chosen based on planning documentation.

Hydraulic performance parameters to use for the comparison of the real and generated networks were established based on the goals of sewer asset management, and the goal of the simplification algorithm itself. The aim of the network simplification and generation algorithm is to aid planning and implementation of future transition pathways of foul sewer networks. Therefore, planning goals defined as 'maximizing the value derived from an asset stock over a whole life cycle, within the context of delivering appropriate levels of service to customers, communities and the environment, and at an acceptable level of risk" [22], are considered as the goals which could, in the future, be used to compare alternative pathways and planning strategies for network adaptations. This study exclusively focuses on hydraulic performance indicators which are related to planning, rather than focusing on planning indicators directly. This is because planning indicators tend to be linked to country regulations or sector specific interests, while assessment of hydraulic performance represents a global yet informative aspect of sewer asset assessment.

The IWA guidelines on sewer asset management were chosen as a starting point to evaluate which planning indicators are impacted by hydraulic performance and consequently identify which aspects of hydraulic performance should be represented to enable network comparison. The IWA guidelines were chosen specifically due to their global and comprehensive character, although future studies may want to consider more specific indicators. Using the IWA guidelines, key hydraulic performance indicators were selected, and if necessary, adapted. The IWA hydraulic performance indicators used for network assessment are summarised in table 3.3.

Code	Indicator	Definition
wOp20	Standardized energy consumption (kWh/m3/m)	Energy consumed for pumping wastewater / (wastewater volume pumped x pump head in meters)
wOp37	Flooding from sanitary sewers	(Number of flooding incidents related to sanitary sewers during the assessment period x 365 / assessment period) / total sewer length at the reference date x 100
wPh5	Surcharging in sewers in dry weather (%)	Length of sewer where surcharging has occurred in dry weather during the assessment period / total sewer length at the reference date x 100
wPh8	Pump power utilized in SE (%)	for all pumps installed in SE(pump nominal power x pump working hours during the assessment period) / (total pump nominal power installed in the sewer system x assessment period x 24) x 100
wPh10	Sewer system pump headroom (%)	Number of pumping stations in the sewer system where pumps operated more than 75% of the time during the assessment period / number of sewer system pumping stations at the reference date x 100
wEn3/4	Intermittent overflow discharge volume/frequency	(Number of overflow discharges that occurred during the assessment period x 365 / assessment period) / number of overflow devices at the reference date
wEn12	Sediments from sewers (ton/km sewer/year)	(Drained weight of sediments removed from sewers during the assessment period x 365 / assessment period) / total sewer length at the reference date

Table 3.3: Performance Indicator Selection form the IWA guidelines [24]

The indicators shown above are used by the IWA for benchmarking of networks where observation and monitoring data is available, as specified in the Introduction, however, such data is not typically available for free. Therefore, the hydraulic performance indicators were broke down into 'natural attributes of the network and cross-compared with modelling outputs to select quantifiable indicators. The results of the analysis are shown in Figure 3.3, it is found that sediment from sewers cannot be calculated in terms of weight due to lack of water quality data, and that the lack of real pump operation data (and the use of the Ideal pump assumption) impacts the calculation of the other indicators.

Indicators	Code	Required Natural Attributes					
		Water Depth	Velocity	Flow	Water quality	Volume at WWTP	Pump operation data
Intermitternt overflow discharge volume/frequency	wEn3 & wEn4	-	-	✓	-	-	✗
Treatment utilisation	wEn*	-	-	✓	-	✓	✗
Surcharging in sewers in dry weather	wPh5*	✓	-	-	-	-	✗
Pump power utilised in SE	wPh8	✓	-	✓	-	-	✗
Standardised energy consumption	wOp20	✓	-	✓	-	-	✗
Flooding from sanitary sewers	wOp37	✓	-	-	-	-	✗
Sediments from sewers	wEn12	✓	✓	-	✗	-	✗
Model output		Water Depth	Water velocity	Flow		Volumes	

Figure 3.3: Natural attributes required for hydraulic performance indicator assessment compared to model outputs available for hydraulic performance indicator quantification. The ✓or ✗symbols show available or unavailable properties needed to calculate each indicator, the (-) symbol represents properties not needed for the specific indicator. Pump operation data is considered to be an unavailable attribute although the ideal pump assumption makes it possible to calculate the indicators which include this attribute.

Based on the obtainable and required information, hydraulic performance indicators are chosen and adapted. Energy consumption indicators are not selected for the analysis due to the requirement for more detailed pump operation data (wPh10, wPh8). The standardised energy consumption indicator (wOp20) is not selected due to the fact that the compared networks, following the inflow calculation methodology, should have the same carrying capacity and site characteristics, therefore no standardisation is required and pumping energy can be compared directly.

For other indicators. wOp37, wPh10, wPh5, wEn3/4, wEn12, the IWA guide makes reference to an 'assessment period' for the calculation of the indicators. Due to the lack of time patterns to carry out long term assessments, and in the absence of monitoring data, the indicators were adapted. A stress test was carried out for the network, this is done by simulating high, low, and typical flow conditions to evaluate different performance indicators in the real and generated network. Energy consumption, similar to wOp20, is quantified for typical flow conditions. The indicators selected include include wPh5, wOp37, WEn3 under high flow conditions, and wEn12 for low flow conditions. Their adaptations are described below.

3.2.1. High Flow Indicators

Approaches reviewed in the literature aim at linking surcharge with replacement and rehabilitation needs of the network, or use surcharge for benchmarking and network comparison. Based on IWA guidelines (indicator wPh5), each pipe in the network is classified as either in surcharge(1) or not in surcharge (0). Conduit surcharge is considered to take place when both ends of the pipe are full (above the maximum 80% filling ratio), surcharge at each pipe in the network is then calculated as:

$$Conduit\ Surcharge(\%) = \frac{\sum_i^n (l_i \times surcharge)}{\sum_i^n l_i} \times 100 \tag{3.1}$$

Where l_i is the conduit length, n is the number of pipes.
An additional indicator was calculated to account for the time for which each conduit is in surcharge, this is calculated as:

$$Surcharge\ Time(\%) = [\sum_i^n \frac{(surcharge \times surcharge\ time_i)}{simulation\ time}] \times \frac{100}{n} \tag{3.2}$$

This gives an indication of how long the system is in surcharge, the volume is also included as a weighting factor for each pipe and its associated surcharge time.
However, this quantification of surcharge doesn't include other conditions in the network where capacity is limited and surcharge takes place, for example if the upstream end of a pipe is full and the Hydraulic Grade Line (HGL) slope is greater than the conduit slope. To account for these conditions, a further indicator is calculated (similar to wOp37), quantifying node surcharge (and thus flooding) in the networks. Node surcharge is assumed to take place when the water level exceed the maximum available depth at the manhole (manhole depth) without any ponding allowed, and is calculated as:

$$Node\ Surcharge(\%) = \frac{n^\circ\ of\ nodes\ in\ surcharge}{n^\circ\ of\ network\ nodes} \times 100 \tag{3.3}$$

The conduit and node surcharge indicator are adjusted by scaling the length of pipe in surcharge with the total length of the network and the number of nodes in surcharge with the number of network nodes respectively (eq. 3.3, 3.1). This adjustment is made to account for the differences in average pipe lengths and number of network nodes in the compared networks, the comparison is made possible by the fact that both networks have the same carrying capacity. However, to make an absolute comparison between the networks, flood volume (wEn3) can be compared as defined below:

$$Flood\ Volume(\%) = \frac{Flood\ Volume\ (m^3)}{Total\ Network\ Inflow\ Volume\ (m^3)} \times 100 \tag{3.4}$$

3.2.2. Low Flow Indicators

The amount of sedimentation accumulated in sewer pipes is considered as an important environmental parameter in the IWA performance assessment guidelines [24], and has been assessed in studies mentioned in the literature. However, the guideline proposes the use of sediment weight over pipe length as an indicator (in the guide referred to as WEn12). Due to the lack of water quality data, sedimentation was instead estimated based on the approach proposed by Penn & Maurer (2018). The study, as opposed to most studies which rely on standard values of shear stress, uses shear critical shear stress, velocity and depth to estimate whether or not a pipe will accumulate sediment. The most critical value is chosen to evaluate whether a pipe will accumulate sediment or not. The study goes further in simulating stochastically generated diurnal patterns with various flow reductions for a 7 day period, which was not done in this study [29]. Low flow conditions (as defined in the previous section of this chapter) are used to estimate sedimentation instead of a real time pattern. Moreover, the study evaluates probability of a pipe accumulating sediment by using regression to evaluate the impacts of slope, diameter and flow reduction on sediment accumulation, this was also not done in this study. The approach is simplified by assuming that sedimentation will take place (with 100% probability) if:

- simulated depth (y) exceeds critical depth (y_{crit}),
- simulated velocity (v) doesn't exceed critical velocity (v_{crit}),
- if shear stress (τ) does not exceed critical shear stress (τ_{crit}).

In summary, if $min\{(y_{crit} - y), (v - v_{crit}), (\tau - \tau_{crit})\} < 0$, the pipe is assumed to be at risk of sedimentation. The decision to consider shear stress, velocity and depth parameters reflects the traditional use in design codes of minimum velocities, shear stresses and pipe capacity as predictors of sedimentation risk [36]. At low flow, the critical parameters ($y_{crit}, v_{crit}, \tau_{crit}$) are calculated using numerical methods, while (y, v, and theta) are calculated from the simulated flow and depth data. The critical parameter θ_{crit} is calculated by solving for Froude's number (Fr) = 1 for critical flow conditions.

$$Fr^2 = \frac{(Q^2T)}{(gA^3)} = 1 \tag{3.5}$$

Where Q is the flow, g is gravitational acceleration (9.81 ms^{-2}), T is the top width:

$$T = D\ sin(\theta_{crit}/2) \tag{3.6}$$

and A is the cross sectional area:

$$A = (D^2/8) \times (\theta_{crit} - sin(\theta_{crit})) \tag{3.7}$$

and shear stress is calculated as:

$$\tau_{crit} = \rho \times g \times S \times D \times \frac{(\theta_{crit} - sin(\theta_{crit}))}{4\theta_{crit}} \tag{3.8}$$

Where S is conduit slope.

Once the critical parameters are calculated, the pipes are classified as either at risk or not at risk of sedimentation (sedimentation risk = 1 or 0). The pipe length l, under sedimentation is calculated, for a network with n pipes, as a percentage of total pipe length in the network:

$$Total\ sedimentation\ risk(\%) = \frac{\sum_i^n (l_i \times sedimentation\ risk_i)}{\sum_i^n (l_i)} \times 100 \tag{3.9}$$

3.2.3. Typical Flow conditions

Typical flow conditions are used to calculate energy consumption for the network, as well as average time of concentration for the networks. The time of concentration is calculated as the time taken for water from each extreme node of the network to reach the outfall. This is done by considering for each extreme node (n), for all the conduits $\{i, ..., c\}$ in the path (p_n) to the outfall. Velocities(v) in each pipe of length d, are calculated, and the final time of concentration (Tc) for the node is calculated as follows:

$$Tc_n = \sum_i^c \frac{d_i}{v_i} \tag{3.10}$$

Typical flow conditions are also used as a verification step to ensure that no flooding occurs for the design and typical network operation conditions. In addition, with typical flow conditions one can also analyse outflow hydrographs at comparable nodes in the real and generated networks. The hydrographs show outflow in [m^3] over time. The comparable nodes are selected using the node degree and node strength properties. Node degree is the number of edges connected to each node (in-going or out-going), node strength is the sum of the weights (in this case cumulative flows) of the conduits connected to each node. These two parameters, along with their geographic location, can be compared both in combination, by multiplying the weight by the degree, or singularly. In general, while node strength and node degrees can be used to narrow down the selection of nodes for comparison, best judgement should still be applied rather than a fully automated approach.

Summary

The calculated indicators' scale, unit, and simulation scenarios are summarised below for clarity.

Table 3.4: Indicators Scale, Unit and Simulation Scenario

Indicator	Scale	Unit	Simulation Scenario
Flood Volume	Network Level (Global)	Volume (% of incoming flow)	High Flow
Surcharging of pipes	By length of pipe	Percentage length of pipe experiencing surcharge (m/m)	High Flow
Surcharge time	By pipe	Percentage time the system is in surcharge (s/s)	High Flow
Node Surcharge	Network Level (Global)	Incidents per number of nodes(%)	High Flow
Total Sedimentation Risk	By length of pipe	Length of pipe accumulating sediment (m/m)	Low Flow
Pump power utilised	Network Level (Global)	kW	Typical flow
Time of concentration	For each extreme node	Time from extremity to outfall (s)	Typical Flow
Outflow Hydrograph	At comparable nodes	Volume per time step (m^3hr^{-1})	Typical flow

3.3. Hydraulic Performance

3.3.1. Network Inflows

Simplified Sewer Network

To investigate the accuracy and computational savings of algorithmic generated sewer networks in their representation of real sewer networks, algorithmic generation and sizing methods were selected. The UrbanBEATS model was chosen to generate data from basic input data. The model uses land-use, elevation and population data to create a gridded database of geospatial information which includes wastewater volumes and breakdown (grey water, black water etc). The 'grid' considered by UrbanBEATS can have varying sizes (from 250x250m to 1000x1000m) which allows exploration in computational efficiency - accuracy trade-offs.The detailed methodology used by UrbanBEATS to abstract urban environments is summarised by Duque et al. (2020) and explained in detail by Bach et al.(2018), and is based on planning guidelines and documents to identify specific land-use characteristics.

The wastewater generation process for residential areas was carried out using the direct input method which takes into account occupancy (calculated from urban form abstraction) and an user defined per capita daily water consumption, where wastewater generated is calculated as:

$$wastewater = occupancy \times per\ capita\ water\ consumption \tag{3.11}$$

For other land uses (education, commercial, industry), are based on the floor areas abstracted by the model and estimated values of water consumption per square meter (L/d/sqm) based on literature values. For these uses, the wastewater is calculated as:

$$wastewater = land\ use\ (sqm) \times\ water\ consumption\ per\ sqm \tag{3.12}$$

Default values used in the model are listed below.An example of wastewater demand abstraction carried out in UrbanBEATS is shown in figure 3.4 below.

Land Use	Water Consumption
Residential	155 L/d/person
Commercial	40 L/d/sqm
Industrial Demand	40 L/d/sqm
Office Demand	40 L/d/sqm

Table 3.5: Water demand per land use (from UrbanBEATS)

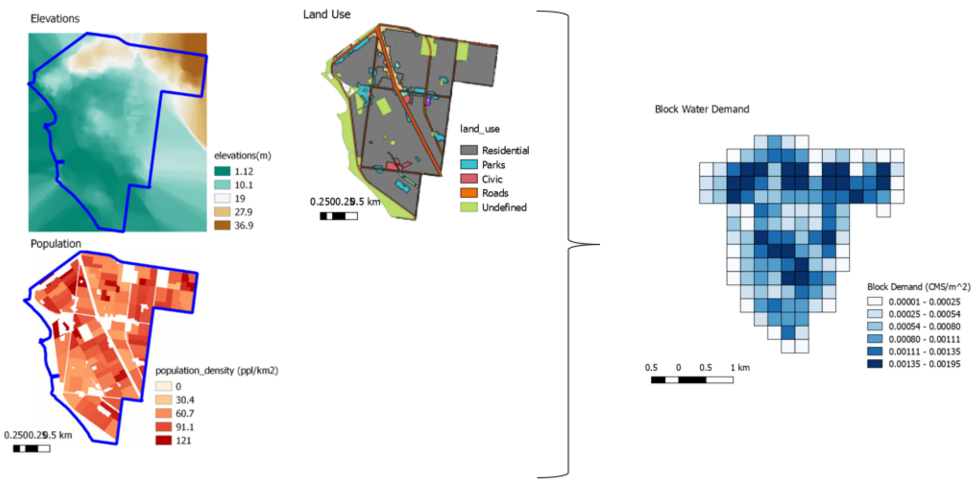

Figure 3.4: Data aggregated in UrbanBEATS for wastewater estimation per block.

Real Network

For the real network, in the absence of existing flow data, inflows can be inferred by assuming the flow distribution and magnitude calculated in the UrbanBEATS blocks. The wastewater generated in the UrbanBEATS blocks (in $L/d/m^2$) is assigned to the contributing area for each pipe in the real network. In order to do so, the contributing area for each pipe in the network has to be defined. This is done by delineating Voronoi regions for the set of manholes in the network. For a set of n manholes $p_1 ... p_n$ the Voronoi region, R_x (here the area of contributing flow) for a manhole p_x, contains all the points in the metric space S (given by the case study boundary), whose distance (d) to p_x is inferior to the distance to any other manhole in the network. Equation 3.3 below formally represent this concept, and is at the base of the scipy.voronoi (implemented also in GIS) tool used to delineate the Voronoi polygons for the network.

$$R_x = \{s \in S \mid d(s, p_x) \leq d(s, p_j) \; for \; all \; j \neq x\} \tag{3.13}$$

This simplification assumes that distance is the only contributing parameter to determining area of contributing inflow, the assumption was made based on typical sewer design processes and supported by examples in the literature [17]. An example of Voronoi polygons delineated using scipy.voronoi in QGIS is shown in figure 3.5. The wastewater generated in the UrbanBEATS blocks (in $L/d/m^2$) is assigned to Voroni poligons by geographic location. The voronoi polygon wastewater can be then calculated from the assigned wastewater multiplied by the polygon area.

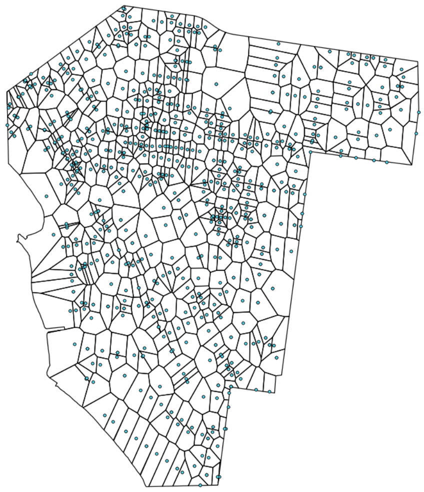

Figure 3.5: Voronoi Polygons delineated for the 7km^2 case study

3.3.2. Network Topology

Simplified Sewer Network

Once the wastewater per area has been established, for the algorithmic generated network, a topology has to be defined. the information is used as input, along with information on connectivity of each block (adjacent blocks list), and the elevation at the centroid of each block, for the Infrastructure delineation module. The delineation module also requires the definition of a 'sink' block for the network (an outfall or treatment plant). Flow paths are then delineated using the D8 algorithm choosing the steepest slope flow path. In flat or local depression areas a 'pit removal method' is used to alter the elevations artificially ('dig') until a lower block is reached, the details of this method are described by Duque et. al (2020).

Real Network

For the real network, topology is usually found in a shapefile format, however the information is not always of suitable quality to be used for network modelling. Often these files contain, for each link,

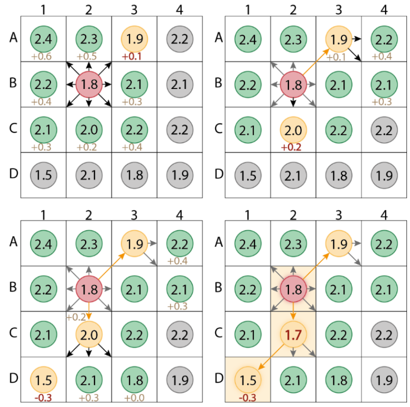

Figure 3.6: Carving algorithm used for terrain depression correction for topology delineation (from [13])

information on upstream and downstream nodes, pipe material and year of construction. However, readily available files are not intended to be used for detailed network modelling, therefore typical errors in network files include disconnected portions of the network (shown in figure ??), wrong link directions creating unexpected sinks in the network (shown in figure 3.7, 3.8), and incoherent node names. Taking into account these typical problems, a methodology to correct real networks topology was formulated. Firstly, new node names are assigned based on coordinate location to avoid naming mistakes, the upstream and downstream nodes for each link (or pipe) are then reassigned based on the new node (or manhole) names. The disconnected portions of the network are then identified using the python networkx package. Once the disconnected parts of the network are found, based on the sewer shape file information, links are extended using the snapping tool of QGIS, or manually added. The node coordinates, names, and link upstream and downstream nodes are then redefined. To correct link directions (upstream downstream nodes) in parts of the network which created unexpected sinks based on the network topology (in areas where there are known to be no outfalls or treatment plants), an algorithm was formulated. The algorithm identifies the most external ('source') nodes of the network, and the 'sink' (outfall/treatment) nodes of the network. The network is then formulated (in Python) as an undirected graph, from which paths from extreme nodes to outfalls are found. The paths, based on the assumption of network connectivity, represent the right link directions in the network, and are used to automatically correct the given network topology file a simplified example of this method is given below .

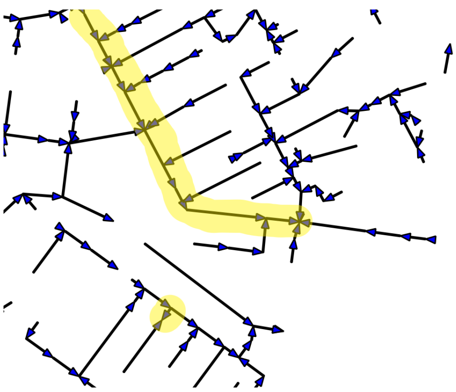

Figure 3.7: Wrong flow direction (highlighted in yellow) creating unexpected sink in the network

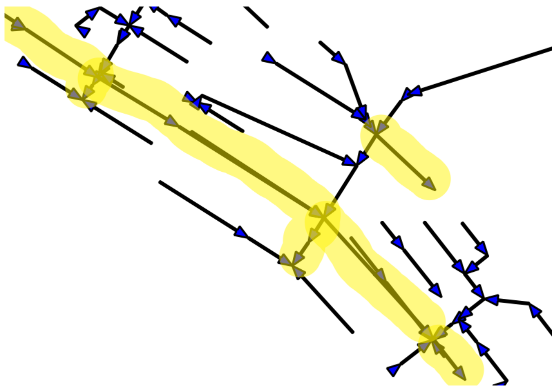

Figure 3.8: Wrong flow direction showing pipes (highlighted in yellow) not flowing to the outfall and creating unexpected sinks

Figure 3.9: Disconnected components of the 57km^2 network. Disconnected components subgraphs, defined as smaller graphs which are not joined to the main (larges) graph, are represented by coloured thick lines while the main network is shown with brown thin lines.

Real network topology fixing method

1. Identify disconnected sub-graphs using nx.connected_components algorithm. A connected component is a subgraph in which two nodes are connected to each other through a path (of 1 or more edges), and which is connected to no vertices in the supergraph (the largest component).

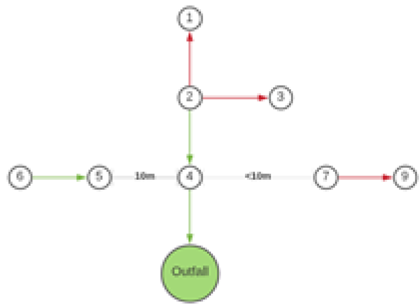

Figure 3.10: Step 1. Identify disconnected sub graphs

2. Connect the connected components within the threshold distance (foound to be 10m by trial and error) using QGIS GRASS v.snap algorithm, which snaps links to the closest vertex within the threshold distance. Redefine network links and node names based on new link coordinates using the QGIS GRASS v.allocate_network tool which identifies network structures by defining links (upstream and downstream nodes) and nodes.

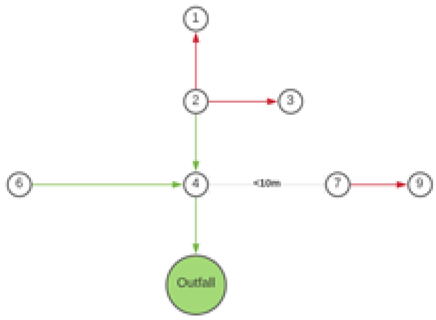

Figure 3.11: Step 2. Connect sub graphs within threshold distance through v.snap

3. Conncect the conected components outside the threshold distance manually using the function nx.add_edge

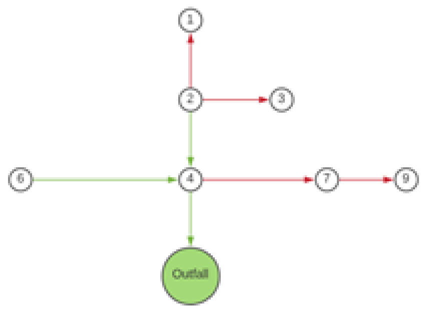

Figure 3.12: Step 3. Add edges to connect sub graphs outside of threshold distance

4. Identify source nodes in graph by (1) creating arrays with upstream and downstream nodes, (2) joining the two arrays, (3) identifying unique nodes in the joined array. This is possible because, even if flow directions are wrong, source nodes will be referenced only once.

Listing 3.1: Unique IDs code

```
Create upstream ids array
Create downstream ids array
Join upstream and downstream ids
FOR id in joined array:
     Count number of times each id is present in joined array
Create empty unique is set
IF id is found only once in joined array THEN
    Add id to unique id set

OUTPUT -> Unique ids are: [6, 1, 3, 9, O]
```

5. Create an undirected graph for the given node list and identify all simple paths from source to outfall (O). The nx.find_simple_path algorithm was used to find the path from source to outfall using a depth-first algorithm which discovers, for each node, all the backwards and forwards edges, and returns the first found path from source to target. This will return the right flow path, given that, for a path with no loops, only one path will be recognized from source to target.
A new graph, is created with the new corrected flow paths.

Listing 3.2: Create graph code

```
Create empty directed graph G
FOR each element in unique ids:
    IF element is not an outfall:
       Find path from element to outfall  #using nx.simple_path
       FOR node (n) in path length:
          IF n is not the last node in path THEN
            Add an edge in graph G from (n) to (n+1)
Print graph edges
OUTPUT -> [(6, 4), (4, O), (1, 2), (2, 4), (3, 2), (9, 7), (7, 4)]
```

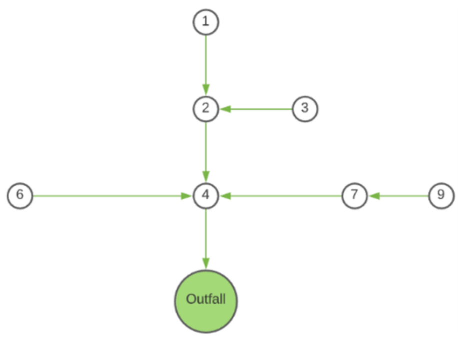

Figure 3.13: Step 5. Create directed graph with identified correct edge directions

3.3.3. Hydraulic Design

After topology delineation, flows for the real network are accumulated along the network paths using the path finding algorithm and the flows attributed to each pipe as reported in listing 3.3. The hydraulic design of the network is carried out using the Pipe by pipe (PBP) algorithm proposed by Duque et al. (2020). The design flow is calculated from the established manholes inflows as:

$$Q_{max} = Q_{ww} \times fww \times f_{max} \tag{3.14}$$

Whew Q_{ww} is the inflow calculated from the previous sections, f_{max} is the water use peak factor, and f_{ww} is the wastewater return factor as the percentage of water which returns to the sewer after being used. For the case study,$f_{max} = 1.85$ and $f_{ww} = 0.85$ were assumed [8, 13]. The same design factor were used for all land uses to design for redundancy, although commercial and industrial land uses typically have more regular consumption patterns throughout working hours.

For each pipe, and for each diameter a range of feasible slopes are defined for a maximum filling ratio of 0.75 [8]. Once the boundary conditions have been calculated, the algorithm identifies the external pipes of the network (with no incoming pipes) and, starting from the smallest diameter and slope, carries out iterations until the first feasible diameter, which satisfies hydraulic constraints and excavation limits, is found. After, the downstream pipes are dimensioned, making sure that the starting diameter is greater or equal to the diameter of the upstream pipe. At intersections, the algorithm checks that all upstream pipes have been connected and designed to ensure network connectivity and hydraulic design feasibility. An overview of the methodology is given in figure 3.14.

Listing 3.3: Flow accumulation code based on the definition of sources and paths defined in the previous step by step topology fix process

```
#Cumulative flows
Create empty sum array
FOR element in  unique ids set:
     list all simple paths from element to outfall #using nx.all_simple_paths
     FOR each path in paths list:
          FOR each node in the path:
               list all the nodes which contribute to node inflow
                set the sum object to 0
               list all the edges contributing to node inflow (to which design flow
               has been assigned)
               FOR edge in contributing edges:
                    sum flows for all edges contributing to the inflow
                    go through all the edges in the path
               FOR all edges in all paths:
                    add an edge attribute 'cumulative_flow' =
                    sum of flows from all contributing edges + edge design flow
```

The dimensioning algorithm is used to dimension the generated simplified network and the real network to make the networks comparable for the hydraulic performance evaluation in the absence of an existing hydraulic model or calibration data. The decision to dimension the real network using the PBP algorithm, instead of using the diameters of the real network, is based on (1) lack of information regarding network pipe invert elevations, (2) lack of information on the design principles used to originally size the real network and (3) the satisfactory network diameter representation shown by Duque et al (2020). Due to the typical age of sewer networks, as well as lack of available documentation regarding the processes followed in sewer design (especially for older systems), it is not feasible to predict the design criteria used to size older networks, therefore the PBP algorithm is used to size the network based on current design parameters and flows calculated as explained in the previous sections of this chapter.

3.3.4. SWMM Model

The hydraulic model for both the simplified algorithmic generated network and real network were formulated in SWMM 5. The specific software was used because of its open source nature and its ability to run in Python with PySWMM. The main inputs into the SWMM model include links, nodes, node inflows, flow directions, pump locations.In some cases, pumps in proximity of one another were merged

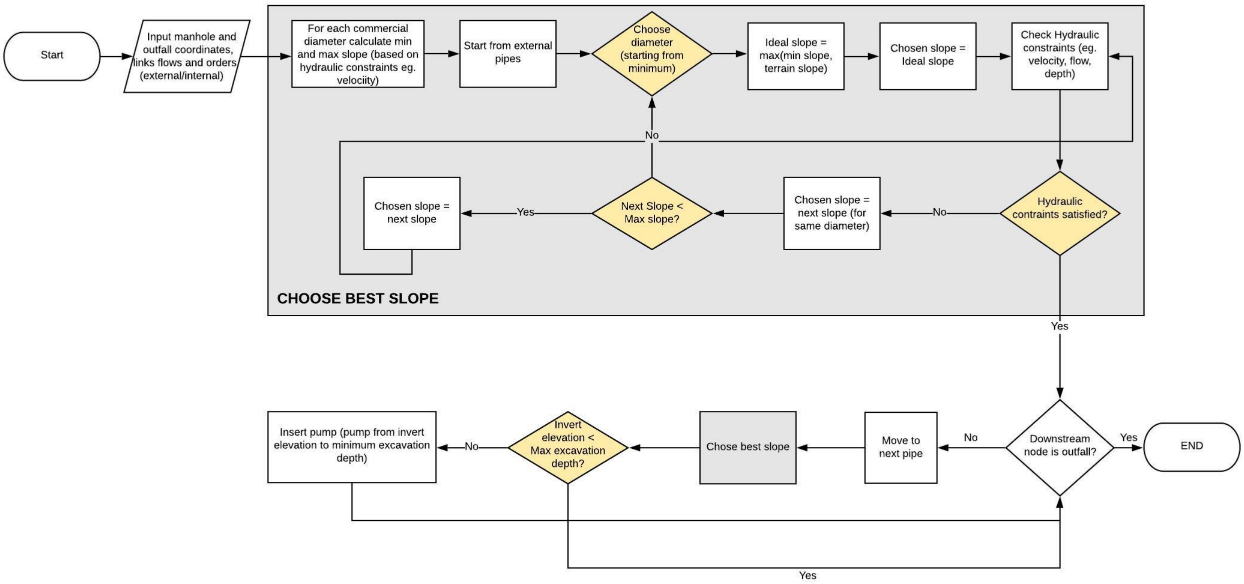

Figure 3.14: Pipe by pipe algorithm overview

into a single equivalent pump. Given the lack of specific pumping data, pump design was simplified by not considering real pump specifications. The pumps in the SWMM model are characterised as 'Ideal' pumps, defined as pumps which have 100% efficient transfer capacity (the flow rate is equal to the inflow rate at its inlet node), this is an approach typically used in preliminary modelling and design [30].

The dry weather flow was defined for each manhole based on the previously mentioned inflow generation method (without the design peak factor). To establish time variation patterns for the baseline wastewater flow, the nodes in the network were assigned a land use based on the predominant land use in their influence area (either the Blocks or the Voronoi polygons). Patterns for commercial, industrial and educational land use are assumed to be equal, while residential land use is assigned a separated pattern.

Due to the lack of a long-term time series of water flows in the foul network, and to reduce computational time, high and low flow conditions were simulated based on patterns found in the literature. While the patterns might not reflect strictly realistic conditions in the network, the objective is to carry out a 'stress-test' for the algorithmic generated simplified network to evaluate performance outside of the design flow conditions.

Time patterns were thus defined to simulate design, high flow and low flow conditions in the network. Design flow conditions were simulated as a verification step to ensure no flooding occurred in the network under typical conditions 3.15, typical demand patterns for the network were obtained from analysis of inflows at the Melbourne wastewater treatment plant [37]. The peak factor for the typical demand pattern is found to be 1.2, which is slightly lower than the design factor of 1.57, used for redundancy. As the typical flow pattern aggregates various land uses, the same pattern was used for all the land uses on site.

High flow conditions can be used to investigate flooding and surcharging conditions in the network. This is done by applying an extreme demand pattern to the existing inflows, these are obtained from literature which aggregates data over 1000 demand nodes and is presented below 3.16[33]. The residential design pattern was used to represent extreme flow conditions, while for commercial and industrial flow conditions the design flow conditions were used, thus applying a constant peak factor of 1.57 between 6:00 to 19:00, this was done because commercial and industrial water demands are assumed to be more stable than residential water demands, thus the design flow was deemed a sufficient testing factor.

The low flow conditions are used to assess low velocities and shear stresses which can be used to investigate sedimentation. This is done by multiplying the calculated demand by a constant factor of 0.6, identified as the base flow factor in the sewer network from analysis of wastewater treatment plant data [37]. Summary of the simulation set up is given in Appendix A.

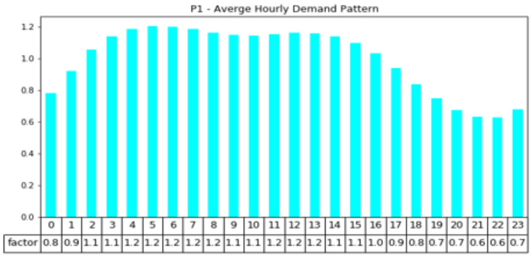

Figure 3.15: Typical demand pattern Melbourne [37]

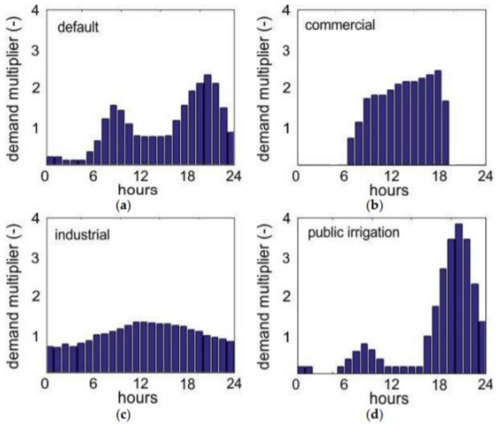

Figure 3.16: Extreme water demand pattern Melbourne [33]

Two different simulation methods were used to test the network under high low and typical flow conditions. The Dynamic Wave simulation method which solves the full one-dimensional form of the Saint-Venant equations, solving continuity and momentum equations for conduits and a volume continuity equation at nodes, was used for high and typical flow conditions. Dynamic wave accounts for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow [30]. In Dynamic wave simulations, pressurization of conduits is represented when flow in the conduit exceeds normal flow, once the maximum available depth (in the conduit) is exceeded, flooding occurs. This method allows to obtain a most theoretically accurate solutions, however at the expense of computational time, small routing time steps are required to achieve numerical stability.

In low flow simulations, where flow is not expected to exceed normal flow, and the output data requirements enhance the importance of computational time, Kinematic wave was used as a flow routing method. This method achieves numerical stability under larger flow routing steps, hence requiring less computational time. Kinematic wave simplifies the routing process by solving the continuity equation along with a simplified form of the momentum equation in each (not accounting for pressure or entrance losses). The water slope is assumed to be equal to the conduit slope, and the maximum flow capacity corresponds to the normal flow [30].

3.4. Case Studies

For the comparative study, two case studies were chosen to test the generated simplified network under two different scales. To attempt at understanding the effects of up-scaling, two case studies of various sizes, but in the same location were chosen. The case studies are both located in Melbourne Australia, the and have different land use mixes, population densities and site topographies. The city-scale case study has an area of 57km^2, while the smaller case study has an area of 7km^2 and its boundary lies within the larger, 57km^2, case study 3.17. Both case studies represent separated foul sewer systems. Input data for both sites was obtained from open data repositories from the State Government of Victoria [35] and the Australian government [1] and adapted to be used as input for UrbanBEATS. For both sites, no data regarding flows in the foul sewer systems or pumping power was available, as such these properties were inferred based on the methodology previously outlined.

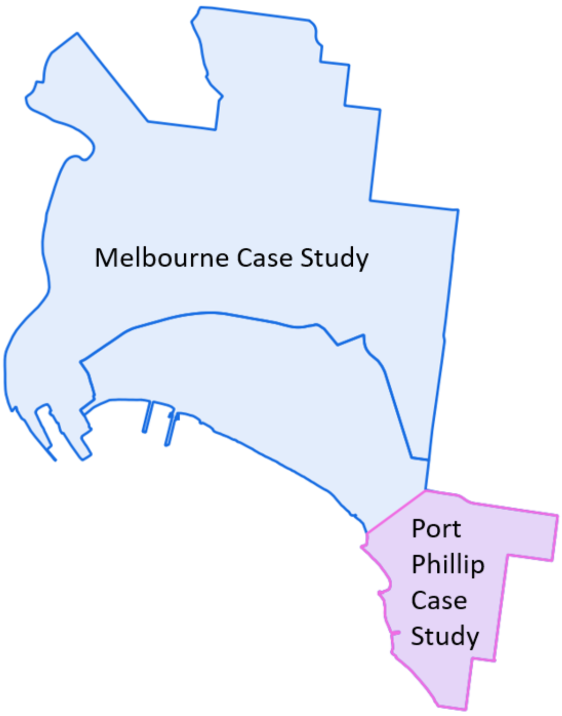

Figure 3.17: Case studies locations, showing the Port Phillip case study is located within the Melbourne case study. This was chosen both due to boundary conditions given that this study is in collaboration with Eawag and because of the ability to investigate up-scaling effects by selecting case studies in the same location.

3.4.1. Port Phillip 7km^2 Case Study

The smallest case study considered is located in Port Phillip, in South Melbourne, and has an area of 7km^2). The sewer system in this area is separated, which allows the foul sewer system alone to be assessed. Some key information about the site is represented in figure 3.18, as can be seen in the Land Use Map, the site land use is predominantly residential (50%) with some Parks and Gardens (18%) and Roads (28%). The site is mostly flat, especially near the coast, higher elevations and steeper terrain slopes can be observed in the north eastern area of the site where hilly areas are observed with a maximum elevation of 36m. Port Phillip is one of the most densely populated areas in Greater Melbourne, the site's estimated population is 40,000 inhabitants, the population density distribution can be observed in the bottom right of figure 3.18.

With respect to the existing wastewater infrastructure, the majority of the foul sewer system has been built in the late 1800's to early 1900's, throughout the years the system seems to have been expanded by building smaller (225mm diameter) pipes to connect all areas. Waste water from the site is directed to the Werribee wastewater treatment plant in Western Melbourne.

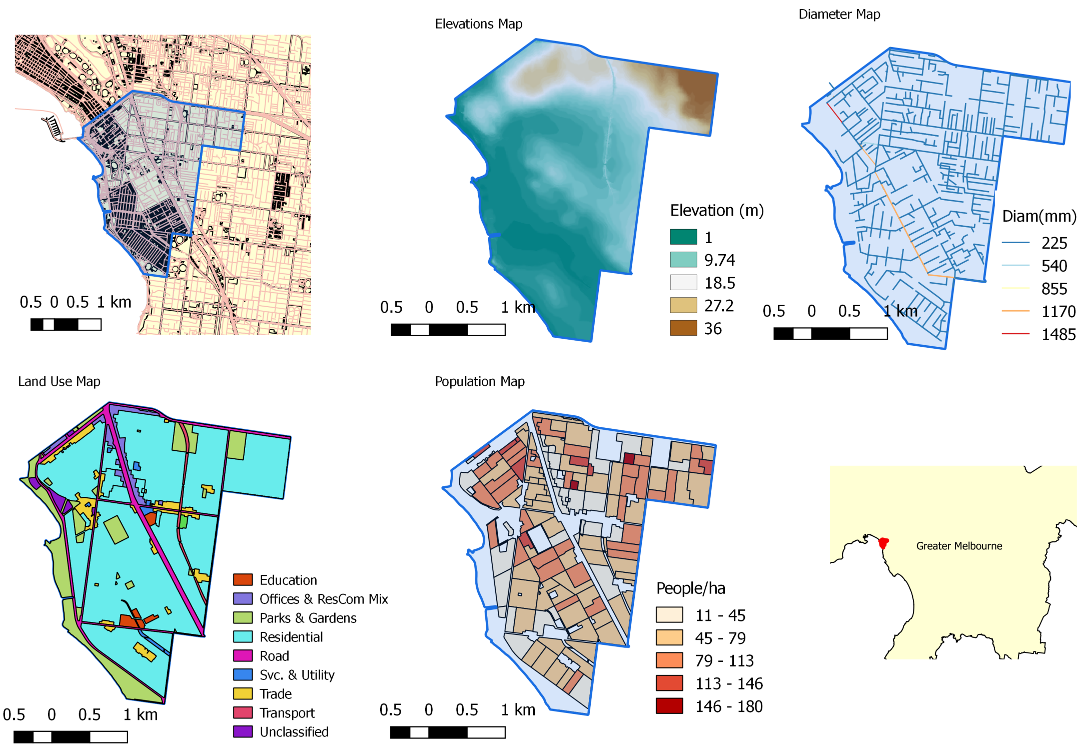

Figure 3.18: 7km^2 Case Study information

3.4.2. Melbourne 57km^2 Case Study

The largest case study considered includes two councils, the City of Melbourne and Port Phillip, both located in central Melbourne with a total area of 57km^2. The site has an estimated total population of 148,633 people. The land use is diversified, residential land use is the dominant classification (28%), followed by parks and gardens (16%) and offices (8%), a relatively large (11%) portion of the site is unclassified (or undeveloped), and there are some heavy and light industry areas concentrated in the central part of the site. Housing density includes both medium (10 dwellings/ha) and high density (30 dwellings/ha) area. The site is characterised, again, by flat areas in proximity of the coast and hilly areas in the eastern inland direction. A summary of the case study information is represented in figure 3.19.

As for the 7km^2 case study, the majority of the main trunk of the system was built in the late 1800's to the early 1900's, some extension to achieve larger coverage were built in the mid to late 1900's. The waste water from the site is treated at the Werribee wastewater treatment plant in Western Melbourne.

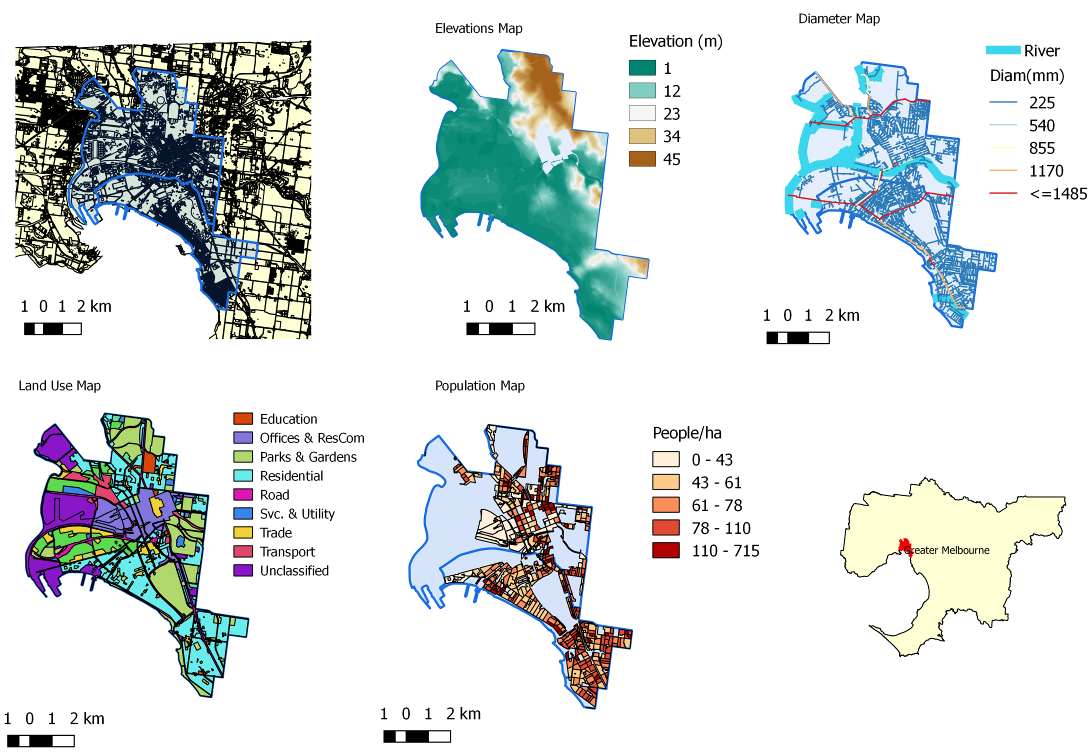

Figure 3.19: 57km^2 Case Study information

3.4.3. Testing and Evaluating Results

The selection of two case studies of various dimensions, with different land use distributions and topographies, allows both the algorithmic generated simplified network and the methodology to be assessed. The tests to be carried out with the $7km^2$ and $57km^2$ case studies are summarised below in Table 3.6.

Additionally to testing two case study sizes, two algorithmic generated network resolutions are also tested. For the $7km^2$ case study, the 250x250m and the 500x500m UrbanBEATS 'block' sizes are tested to evaluate the trade-offs between computational time and ability to represent the real network.

For all the case study sizes and resolutions listed, the hydraulic performance indicators are evaluated. Hydraulic performance comparison should take place both in terms of absolute values (flooding, surcharge, sedimentation, time of concentration, hydrographs), and in terms of trends.

Area	Properties	Test
7 km^2	• Mostly Residential • Mostly gravity driven • Flat Area	• Test algorithm performance at the neighbourhood level • Test methodology at low complexity
57 km^2	• Mixed Land Use • Pump regulated flow • Mostly Flat area	• Test algorithm performance at city level • Test methodology at higher complexity

Table 3.6: Case studies characteristics and testing purposes

To evaluate trends in high flow conditions, pipe surcharge is compared by pipe diameter to evaluate whether surcharge takes place at network extremities (typically with smaller diameters) or at network trunks (typically with greater diameters). Node flooding is also evaluated spatially by considering the coordinates of the flooded nodes.

To evaluate trends in low flow conditions, pipe sedimentation in the networks is compared by percentage of cumulative flow in the pipe. This approach is followed to infer hierarchy of the pipes developing sedimentation. Evaluating sedimentation by hierarchy allows to identify whether sedimentation takes place at extremities or at other more central points in the network.

The distributions of diameters for link surcharge, of cumulative flow percentage for sedimentation and for time of concentrations for the networks are compared using the Kullback-Liebler (KL) divergence distance (Kullback 1959). The KL divergence distance compares the probability of occurrence of each observation (eg. each diameter) between two probability distributions. The KullbackLeibler divergence from discrete probability distributions Q(x) to P(x) in the same probability space (χ) is quantified as:

$$D_{KL}\ (P||Q) = \sum_{x \in \chi} P(x) * log\left(\frac{P(x)}{Q(x)}\right) \tag{3.15}$$

The lower the value of KL divergence, the better represented the real network is by the simplified network, if both distributions are equal, KL divergence =0.The comparison is made for blocks resolutions and case study sizes and evaluated with computational time.

The hydrograph data, on the other hand, is compared using the Nash Sutcliffe Efficiency (NSE), which is commonly used to measure relative residual variance compared to the measured data variance, and is calculated as one minus the ratio of the error variance of the modeled time-series divided by the variance of the observed time-series:

$$NSE = 1 - \frac{\sum_{t=1}^{T}(Q_m^t - Q_o^t)^2}{\sum_{t=1}^{T}(Q_o^t - Q_o^m)^2} \tag{3.16}$$

Where Q_o^m is the mean of observed flow, Q_o^t is the observed flow at time t, and Q_m^t is modelled flow at time t. For similar flow distributions NSE is close to 1.

4

Results

4.1. Generated Flows

The flows generated for the 7km^2 and $57km^2$ case studies are shown below (in $m^3/m^2perday$) in figures 4.1, 4.3and 4.4. The flows estimated below determine a design capacity of 0.06m^3/s for the 7km^2 case and of 5m^3/s for the 57km^2 case study.

4.1.1. Port Phillip 7km^2 Case Study

For the 7km^2 case study, the inflows are generated as shown in figures 4.1, 4.3, 4.2. It can be seen that the highest water demands are concentrated in the northern and central areas of the case study where trade, offices and high density residential areas are concentrated. The distribution of water demands in the Real network voronoi polygons resembles that of the generated network with 250x250m blocks (over the 500x500m blocks) due to the fact that the inflows are directly abstracted from this block resolution.

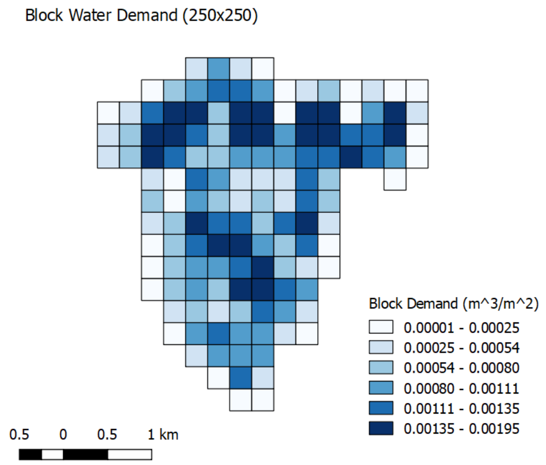

Figure 4.1: Generated Network Block Water Demand (7km^2(250x250m)

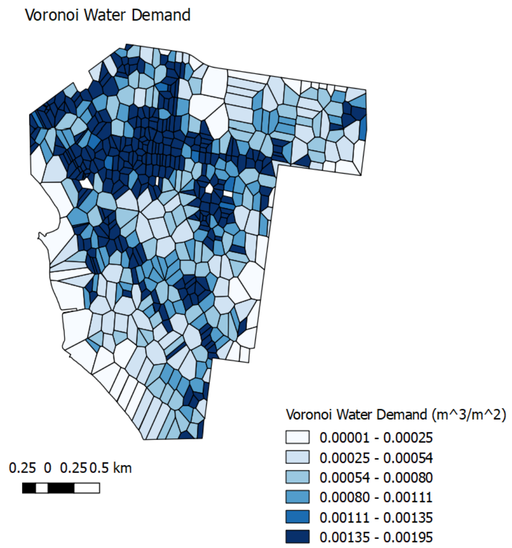

Figure 4.2: Real Network Voronoi Water Demand (7km^2)

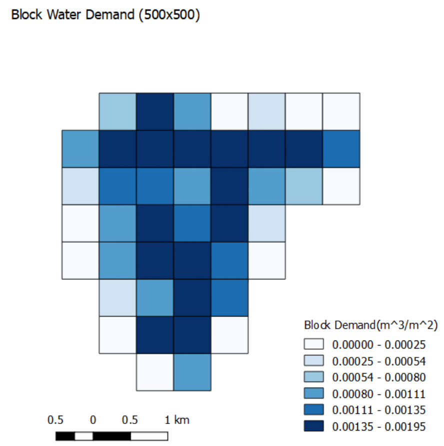

Figure 4.3: Node Degrees (500x500m) 7km^2

4.1.2. Melbourne 57km^2 Case Study

The wastewater inflows for the 57km^2 case study are shown in figure 4.4. The majority of high wastewater generation areas are located in the northern and western side of the case study. The higher inflows are related to areas where office and high density residential land uses are identified. In the north-east side of the case study high flows are attributed to light industry land uses. Again, the inflows distribution in the real network closely resembles that of the generated network with 250x250m blocks due to the fact that wastewater inflow is directly derived from the generated network.

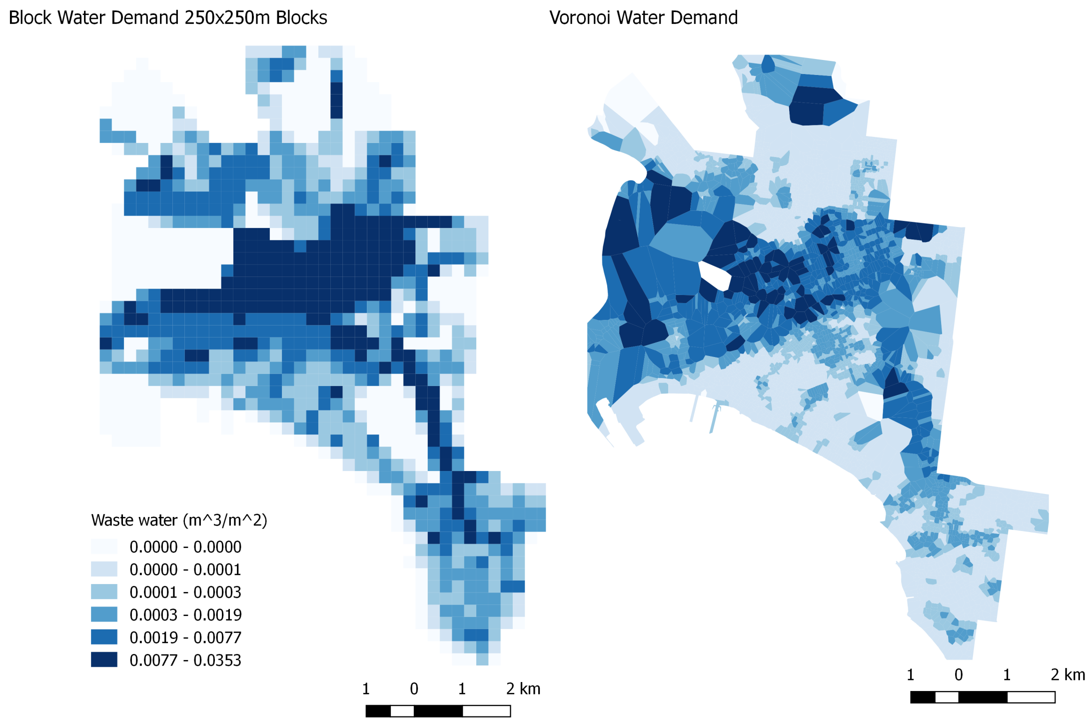

Figure 4.4: Water Demand 57km^2 Case Study

4.2. Network Topology and Hydraulic Design

Once both the real and the generated network flows have been quantified, the generated network topology is generated using the Infrastructure Module, the real network topology is corrected as outlined in the methodology. The resulting topologies are shown in figures 4.6, 4.7, 4.5, 4.9, 4.8.

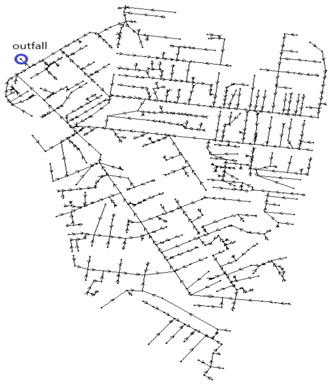

Figure 4.5: 7km^2 Case Study Real Network Topology Corrected (outfall circled in blue)

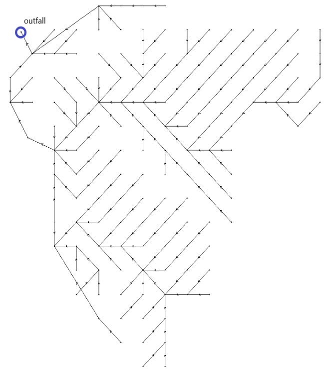

Figure 4.6: 7km^2 Case Study Generated Network (250x250m) Topology Corrected (outfall circled in blue)

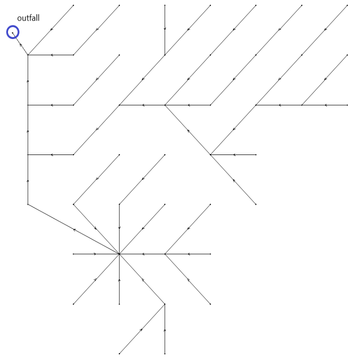

Figure 4.7: 7km^2 Case Study Generated Network (500x500m) Topology Corrected (outfall circled in blue)

Figure 4.8: 57km^2 Case Study Real Network Topology Corrected (outfall circled in blue)

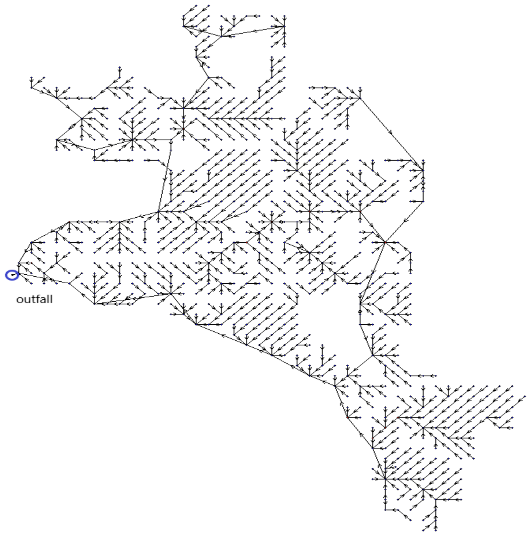

Figure 4.9: 57km^2 Case Study Generated Network (250x250m) Topology Corrected (outfall circled in blue)

4.2.1. Network Node Degrees

The generated and real networks node degrees for the 57km^2 and 7km^2 case studies are reported in figures 4.10, 4.11, and 4.12 below. In this case, only node degrees were compared to provide insight on network topology, which is defined regardless of network inflows. It can be observed that nodes in the real network tend to have, in all cases, higher node degrees than the generated networks, which appear to have less intersections. The difference between node degrees in the real and generated networks is more evident in the 57km^2 case study.

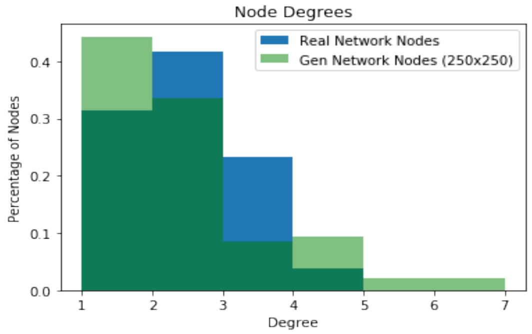

Figure 4.10: Node Degrees of real and generated (250x250m) networks for 7km^2 case study. The majority of nodes in the generated network have 1-2 node degrees (0 or 1 incoming pipes) while in the real network, most pipes have 2-3 node degrees (1 or 2 incoming pipes)

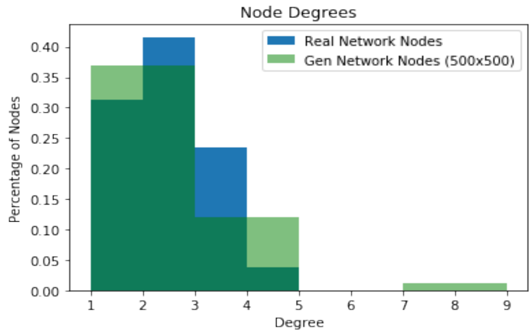

Figure 4.11: Node Degrees of real and generated (500x500m) networks for 7km^2 case study. The majority of nodes in the generated network have 1-2 node degrees (0 or 1 incoming pipes) while in the real network, most pipes have 2-3 node degrees (1 or 2 incoming pipes). However, the difference in node degrees is not as evident as for the 250x250m resolution generated network.

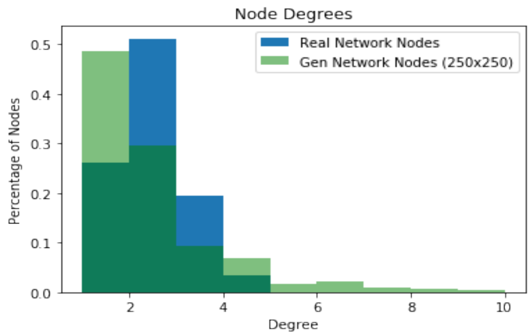

Figure 4.12: Node Degrees of real and generated (250x250m) networks for 57km^2 case study. The majority of nodes in the generated network have 1-2 node degrees (0 or 1 incoming pipes) while in the real network, most pipes have 2-3 node degrees (1 or 2 incoming pipes). The difference between node degrees is much more evident in this case study size than for both the 7km^2 simplification resolutions.

4.2.2. Port Phillip 7km^2 Case Study

Once the topology is defined, the PBP algorithm is ran to size the networks. Their physical properties can be compared and summarised to evaluate physical differences between networks and discuss their potential impacts. The pipe diameter distributions are represented in figure 4.13. It can be observed that the real, generated network with 250x250m and 500x500m blocks are designed with mostly 0.225m pipes. For both real and generated networks, the only pipes with higher diameters are those in immediate proximity to the outfall. The first considerations regarding the analysed networks concern the global aspects of the networks are summarised in Table 4.1 and 4.2.

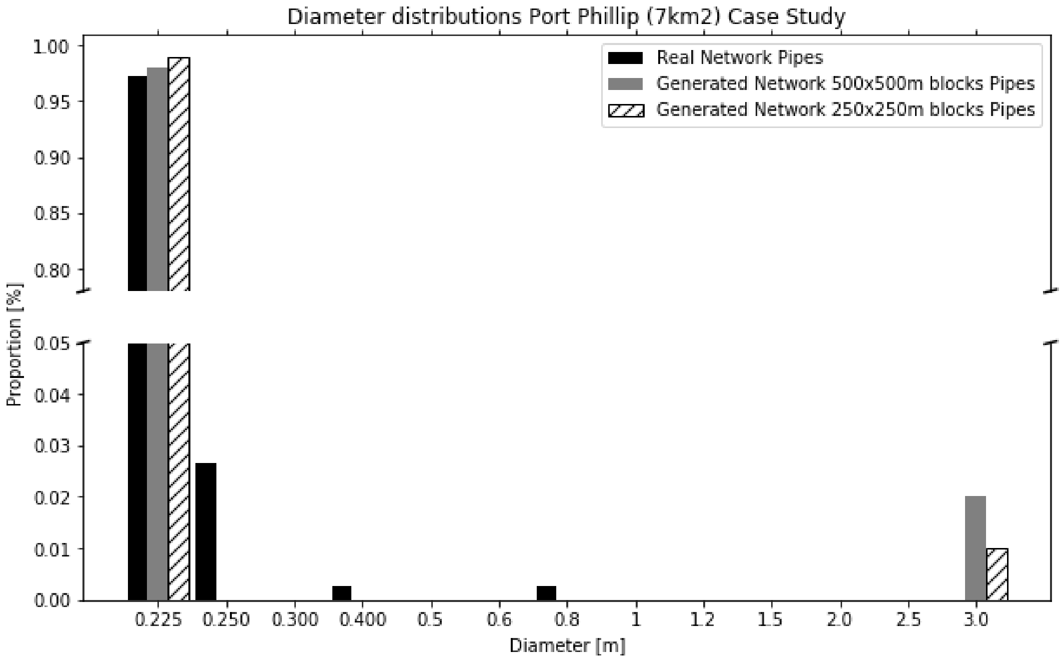

Figure 4.13: 7km^2 Case Study Diameters distributions for real (by proportion of total pipes in the network), generated 250x250m and 500x500m networks. Both the 250x250m and 500x500m generated networks re designed with 0.225m pipes for the whole network, and a single 3.0m pipe at the outfall.

Table 4.1 reports the basic sizing properties of the real and generated networks. It can be observed that the length, number of nodes and number of pipes are significantly larger for the generated network. The real network is 28% longer than the generated network (250x250), and 60% longer than the generated network (500x500). The pipe length is however, on average, shorter for the real network, resulting in a larger number of both pipes and nodes (manholes) compared to the generated networks. The generated network (250x250m) reduces the number of nodes by 81% (compared to the real network), the generated network (500x500m) reduces the number of nodes by 95%. The pumping power is lower in the real network than in the generated network.

Properties	Real Network	Generated Network 250x250	Generated Network 500x500
Total Network Length	61 km	44 km	25km
Number of Manholes	739	139	45
Number of Pipes	738	138	44
Pumping Power	205kW	359kW	50kW
Average Pipe Length	86m	317m	566m

Table 4.1: Networks Physical Properties ($7km^2$)

4.2.3. Melbourne 57km^2 Case Study

Table 4.2 reports the basic sizing properties of the real and generated networks. The same general observation made in the 7km^2 case study are applicable to the 57km^2 case study. The pipe diameters distributions are represented in figure 4.14, showing a majority of 0.225m pipes and some larger diameter pipes in high flow portions of the network. Basic physical properties are summarised in table 4.2. The real network is 17% longer than the generated network (250x250), the number of nodes is also reduced by 80% in the generated network. The pumping power is, again, lower in the real network than in the generated network.

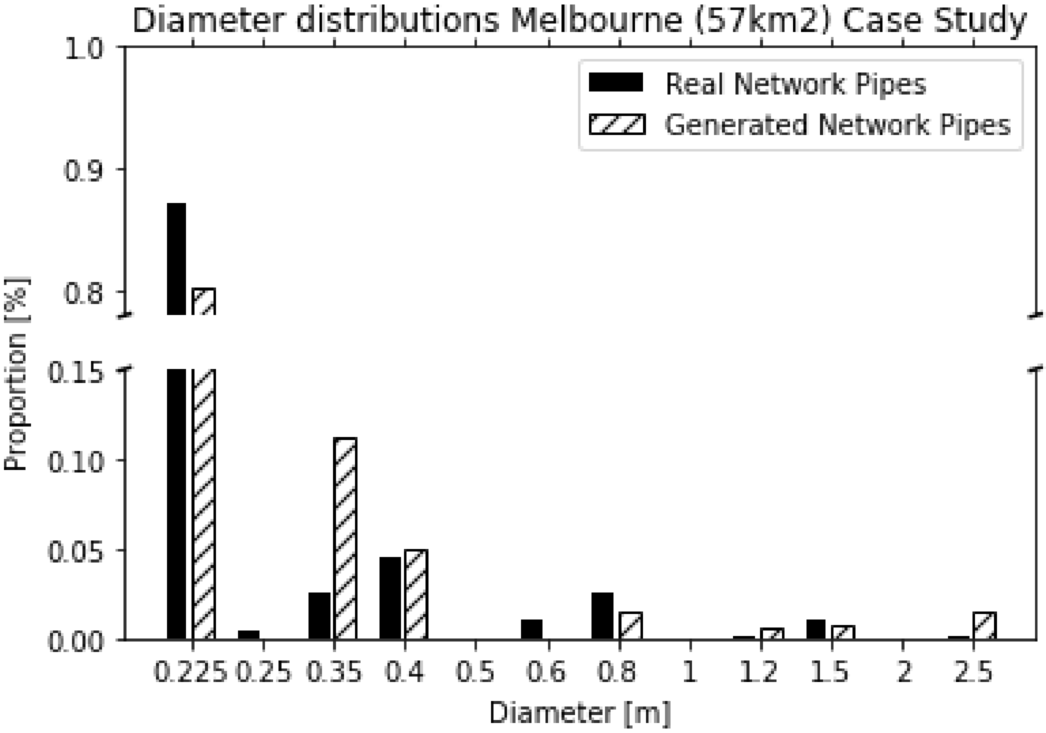

Figure 4.14: 57km^2 Case Study Diameters distributions for real (by proportion of total pipes in the network), generated 250x250m networks. The majority of pipes are 0.225m in diameter. Larger pipes are designed in the high flow portions of the network.

Properties	Real Network	Generated Network 250x250
Total Network Length	282 km	234 km
Number of Manholes	4155	764
Number of Pipes	4153	763
Pumping Power	2449MW	3063MW
Average Pipe Length	67m	308m

Table 4.2: Networks Physical Properties (57 km^2)

4.3. Typical Flow Conditions

The parameters assessed under typical flow conditions, energy consumption, time of concentration and outflow hydrographs are reported below for the 7km^2 and 57km^2 networks.

4.3.1. Port Phillip 7km^2 Case Study

Real Network

For the real network, from the Voronoi Polygons delineation, following the methodology outlined in the previous section of this report, a carrying capacity of 0,063 m^3/s is global design flow for the real

network simulation.
Under design flow conditions, 2 pumps are designed to pump for 15 m with energy consumption of 205kW (at design flow rate).

Generated Network (250x250m blocks)

For the generated network with 250mx250m blocks, the same carrying capacity (of 0,063 m^3/s) was calculated as the design flow.
5 pumps are designed to pump for 9.5 m with energy consumption of 359 kW (at the design flow rate),

Generated Network (500x500m blocks)

For the generated network designed with 500x500m blocks, the same carrying capacity (0.063 m^3/s) is considered, 3 pumps are designed to pump for 5m with energy consumption of 50kW (at the design flow rate).

Times of Concentration

The times of concentration calculated for the three analysed networks are reported in figure 4.15, the mean time of concentration is higher for the real network (3500s) than for both the generated 250x250m network (2000s) and the generated 500x500m network (2500s). The KL divergence between the real distribution and the generated ones (250x250 and 500x500m) is higher for the comparison with the generated 250x250m network than for the comparison with the 500x500m network.

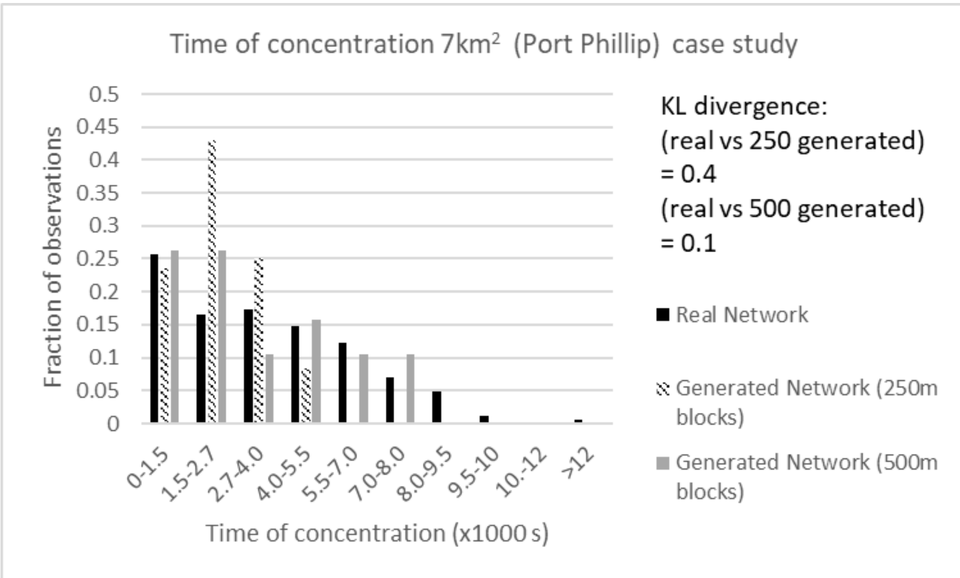

Figure 4.15: Times of concentration distribution 7km^2 Case Study. The real network is shown to have higher on average concentration times than both the 250x250m and 500x500m generated networks. This is confirmed by observed longer on average paths for the real network compared to the generated ones.

Hydrograph Comparison

The hydrograph comparison for the 7km^2 network took place at the nodes identified as comparable for real and generated networks identified as indicted in the methodology. Figures 4.16and 4.17 reflect the longer time of concentration in the real network by showing a slightly delayed hydrograph for the real network.

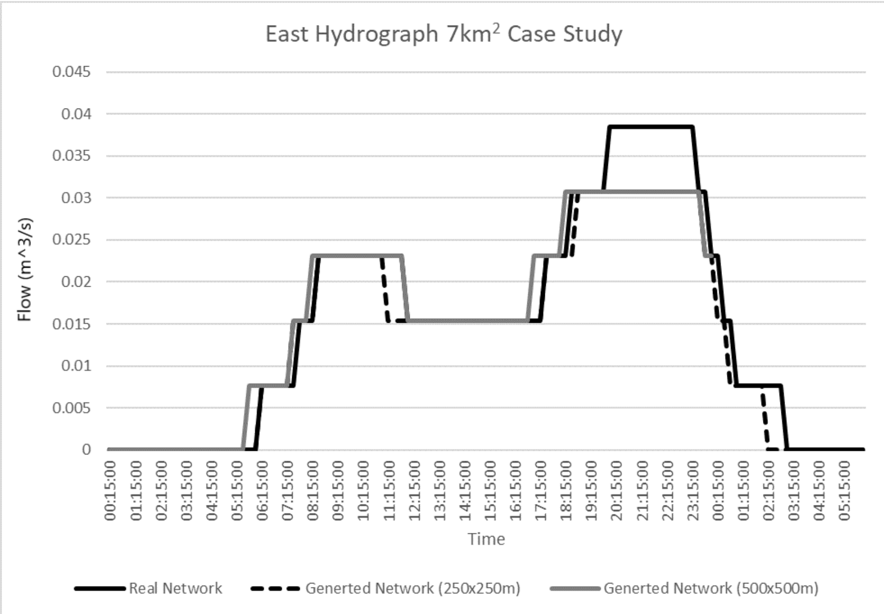

Figure 4.16: Hydrograph of the east portion of the network of the 7km^2 case study, NSE=0.85 (for 250x250m blocks) and NSE=0.79 (for 500x500m blocks). The real network shows a slightly delayed pattern compared to the generated networks.

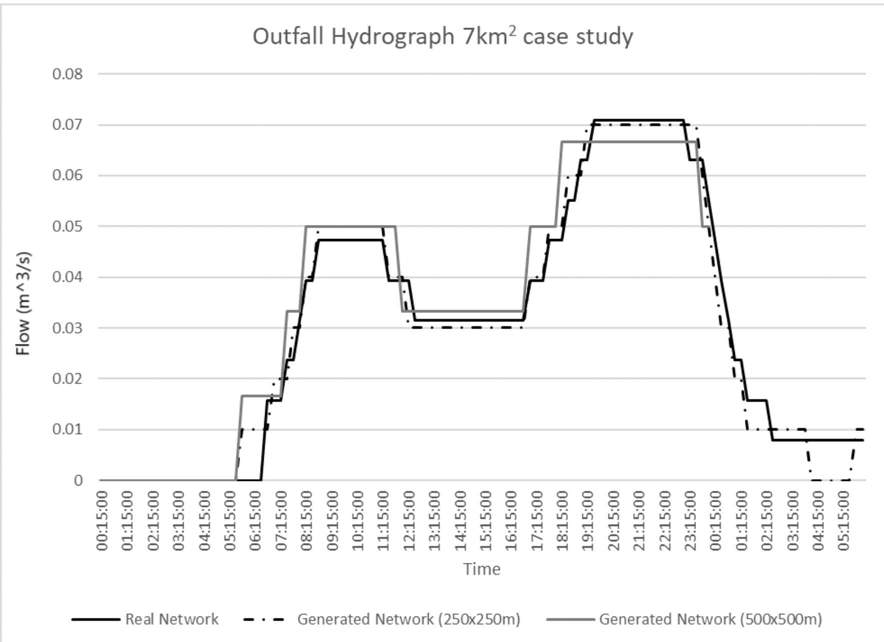

Figure 4.17: Hydrograph of the outfall of the network of the 7km^2 case study, NSE=0.92 (for 250x250m blocks) and NSE=0.84 (for 500x500m blocks). The flow from the real network shows a slightly delayed pattern compared to both the generated networks.

4.3.2. Melbourne 57km^2 Case Study

Real Network

From the wastewater generated following the methodology outlined in the previous section of the report, the same carrying capacity (of 5m^3/s) was used as design flow for the real and generated network (with 250x250 blocks).

Real Network

For the real network, 98 pumps are designed to pump for 677 m with energy consumption of 2449 MW (at the design flow rate).

Generated Network

For the generated network, 59 pumps are designed to pump for 355 m with energy consumption of 3063 MW (at the design flow rate).

Times of Concentration

The time of concentration has been calculated for both networks and is reported in figure 4.18. The mean time of concentration is higher for the generated network (14000s) than for the real network (6000s). The KL divergence for the rel and generated network for the 57km^2 case study is 0.9.

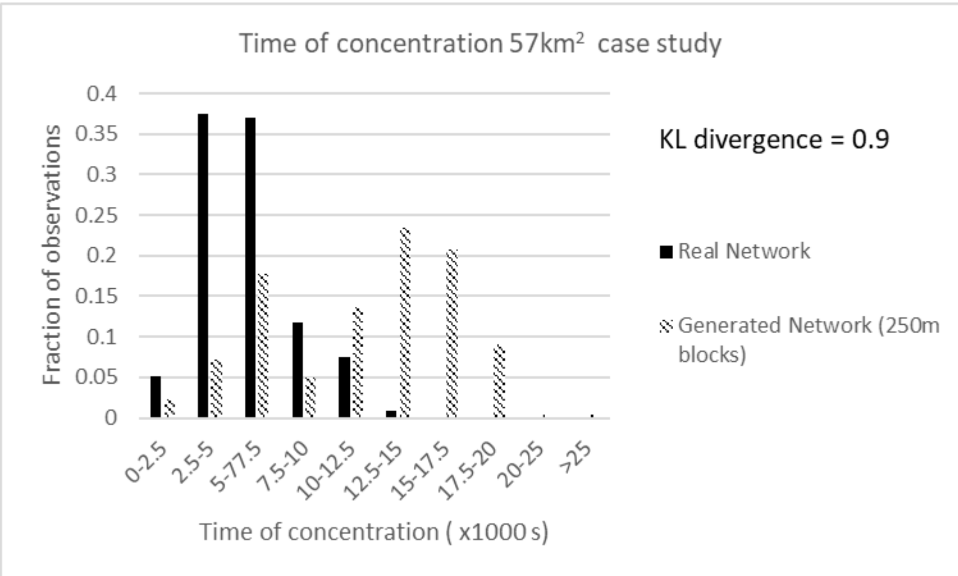

Figure 4.18: Times of concentration distribution 57km^2 Case Study. The real network is shown to have shorter on average concentration times than the generated network. This is coherent with the shorter on average path lengths for the real network, given by the higher node degrees and more tree-like structure of the real network topology.

Hydrograph Comparison

The hydrographs below show delay in the generated network in conveying the flow to the outfall and from the east side of the network.

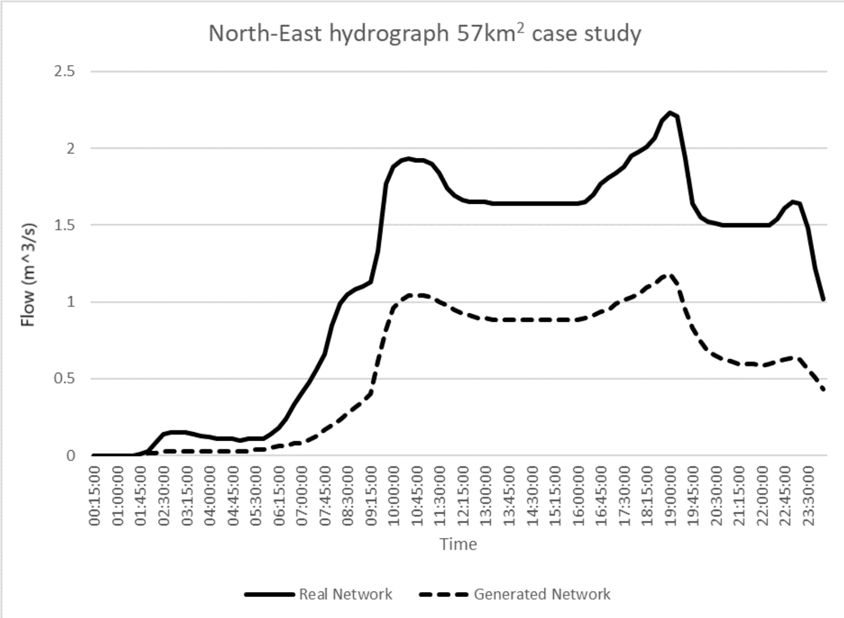

Figure 4.19: Hydrograph of the north-east portion of the network of the 57km^2 case study, NSE=0.2. Flow differences are given by the fact that flows from the east portion of the generated network are partially directed to the south portion of the network. However a delay in flow pattern can be observed in the hydrograph.

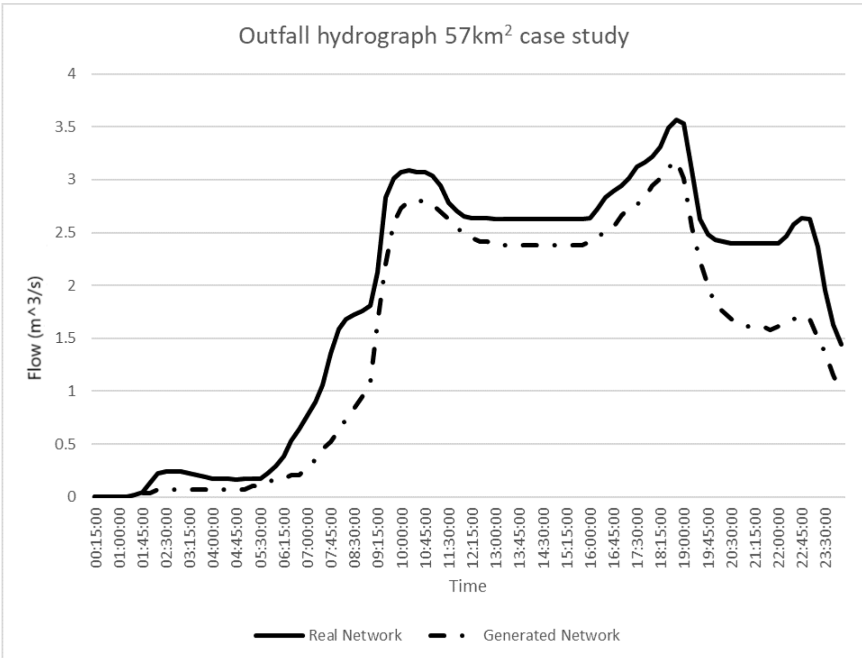

Figure 4.20: Hydrograph of the outfall of the network of the 57km^2 case study, NSE=0.33. Differences in flow are due to the fact that the flow in the last link before the outfall is shown, hence one 'block' of the generated network is omitted. However, a delayed pattern can be observed.

4.4. High Flow Conditions

4.4.1. Port Phillip 7km^2 Case Study

Real Network

For the real network, under high flow conditions the surcharging and flooding are considered (as outlined in the previous section of this report). The high flow conditions at peak demand hour (23:00) are illustrated in figure 4.21. Under high flow conditions, simulated with dynamic wave method, 2% of the incoming flow is lost to flooding . For this network, 0.8% of network length appears to be in surcharge. The conduits are in surcharge 0.02% of the time.

The rate of flooding incidents over network nodes is 0.55% (figure 4.24).

Generated Network 250x250m blocks

For the generated network with (250x250m) blocks,Under high flow conditions the surcharging and flooding are considered (as outlined in the previous section of this report). The high flow conditions at peak demand hour (23:00) are illustrated in figure 4.22. Under high flow conditions, simulated with dynamic wave method, a flooding of 2.3% of the incoming flow occurs (at 23:00) while 7.2% of the network length is in surcharge. The conduits are in surcharge 0.07% of the time

The rate of flooding incidents over network nodes is 2% (figure 4.25).

Generated Network 500x500m blocks

For the generated network with (500x500m) blocks, under high flow conditions the surcharging and flooding are considered (as outlined in the previous section of this report). The high flow conditions at peak demand hour (23:00) are illustrated in figure 4.23. Under high flow conditions, simulated with dynamic wave method, a flooding of 4% of the incoming flow occurs. For this network, 8% of network length appears to be in surcharge, with an average surcharge time of 0.01 hours (occurring at 23:00). The conduits are in surcharge 0.1% of the time.
The rate of flooding incidents over network nodes is 4.4% (figure 4.26).

Pipes in Surcharge Diameters Distribution

The distribution of pipe surcharge by diameter was plotted for the 7km^2 case study, however, given the small size of the site, only 225mm pipes are designed, therefore the distribution is not meaningful and the KL divergence appears to be equal to 0.

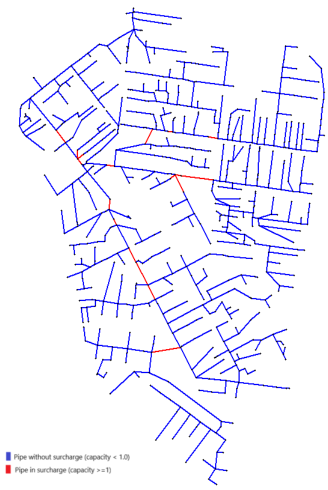

Figure 4.21: Real Network Surcharge

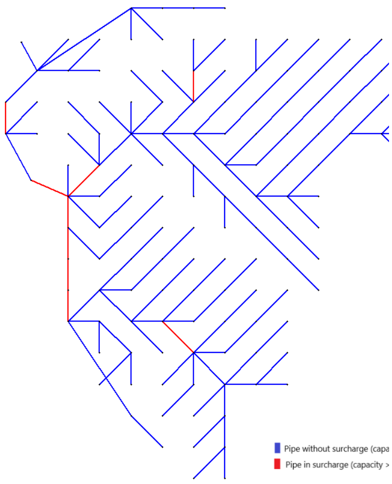

Figure 4.22: Generated (250x250m) Network Pipe Surcharge

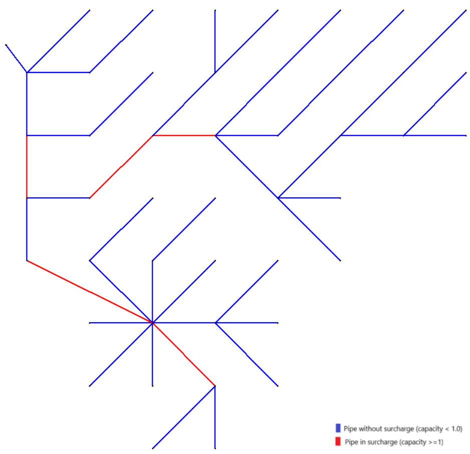

Figure 4.23: Generated (500x500m) Network Pipe Surcharge

Figure 4.24: Real Network Node Flooding

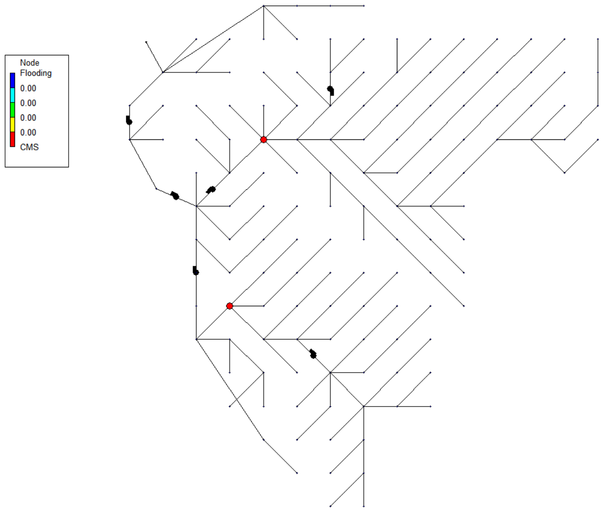

Figure 4.25: Generated (250x250m) Network Node Flooding

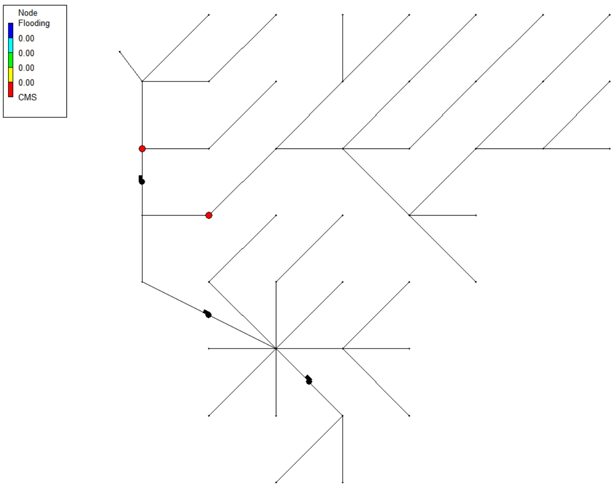

Figure 4.26: Generated (500x500m) Network Node Flooding

Node Flood Comparison

The flooded node comparison is shown in Figure 4.27, showing that flooding is not spatially comparable for the real and generated networks, but occurs in proximity of the outfall or high flow regions of the case study.

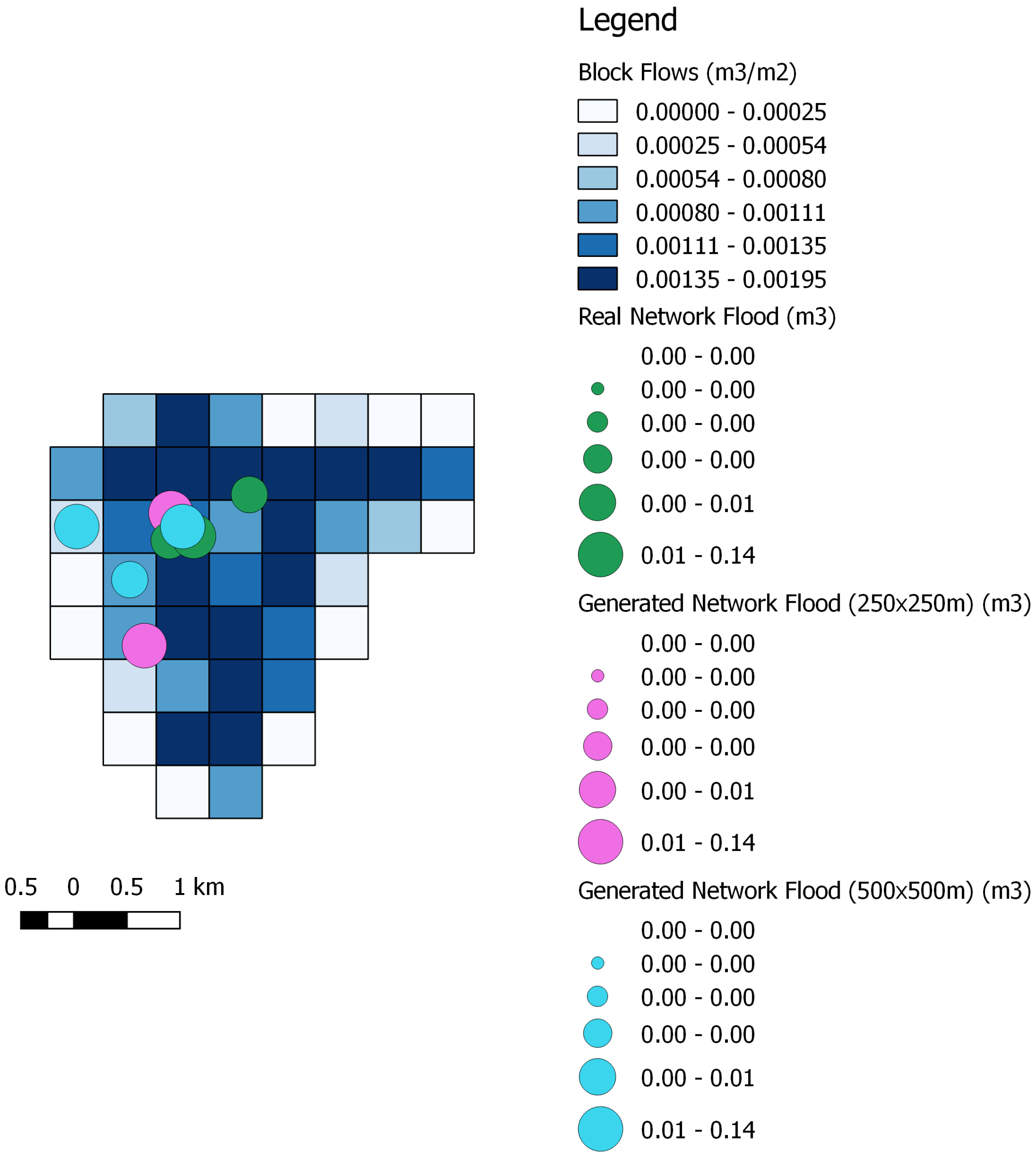

Figure 4.27: Flooded nodes in the 7km^2 case study, the flood volumes are represented by different node sizes

4.4.2. Melbourne 57km^2 Case Study

Real Network

For the real network, the high flow conditions at peak demand hour are illustrated in figure 4.28 below. Under high flow conditions, 26% of the incoming flow is lost to flooding, 8.9% of the network length is in surcharge. The network is in surcharge 34% of the time on average. The rate of flooded nodes over number of network nodes is 2% (figure 4.30).

Generated Network 250x250m blocks

For the generated network, the high flow conditions at peak demand hour are illustrated in figure 4.29 below. Under high flow conditions, 44% of the incoming flow is lost to flooding. In the generated network, 14.4% of the network length is in surcharge. The network is in surcharge 60% of the time on average. The rate of flooded nodes over number of network nodes is 3.4% (figure 4.31).

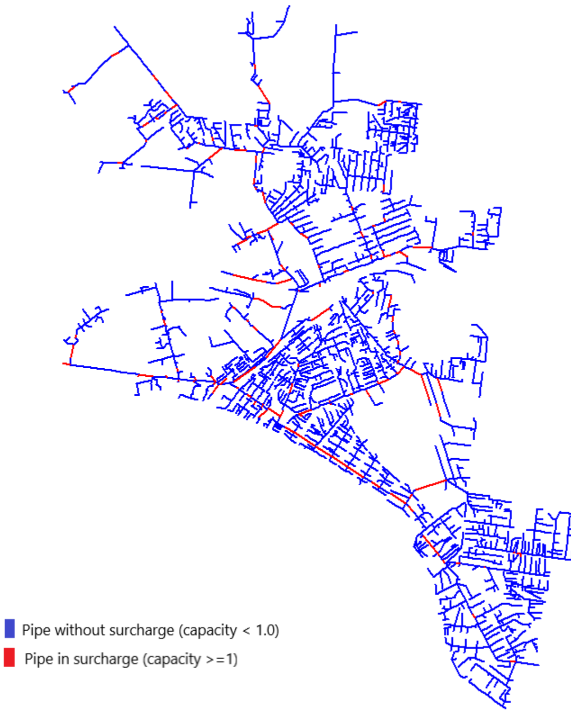

Figure 4.28: Real Network 57km^2 High Flow

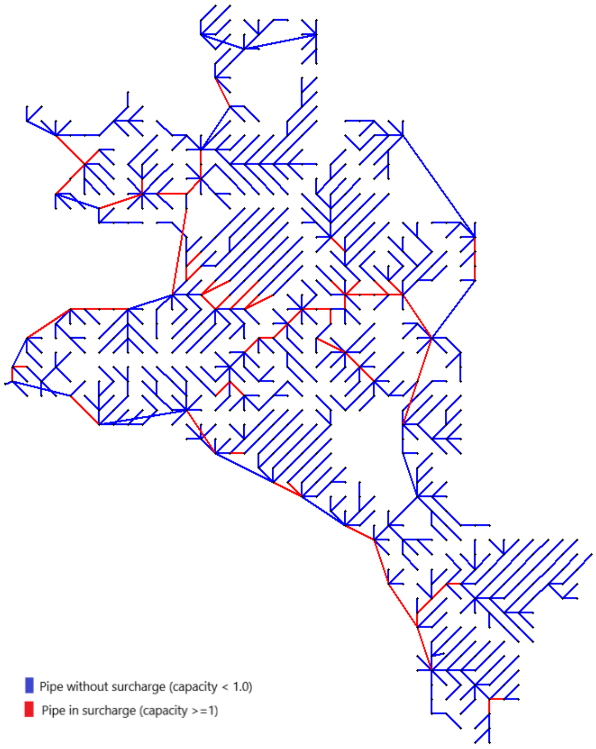

Figure 4.29: Generated Network 57km^2 High Flow

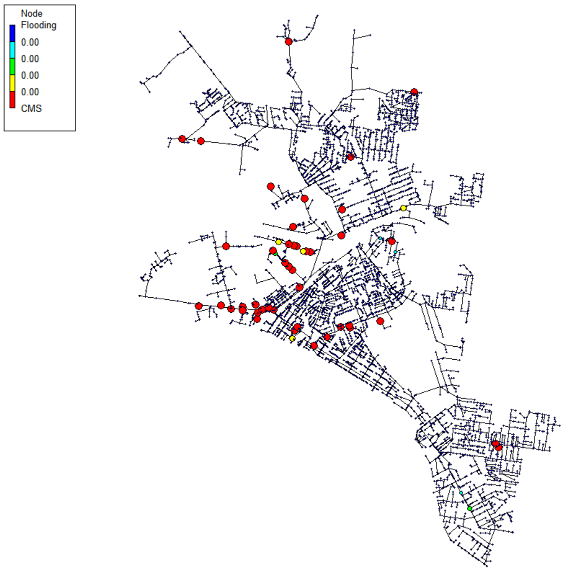

Figure 4.30: Real Network Node Flooding 57km^2 Case Study

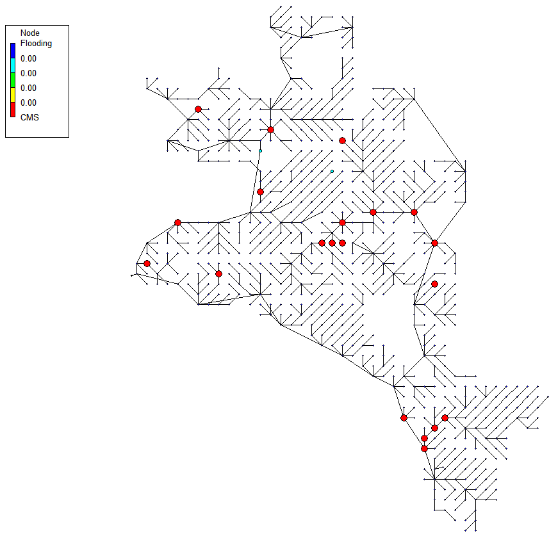

Figure 4.31: Generated (250x250m) Network Node Flooding 57km^2 Case Study

Node Flood Comparison

The node flood locations and volumes (for the whole simulation period) are shown in Figure 4.32 below. It can be seen that the locations of flooded nodes are loosely related to proximity to high flow areas and to the treatment plant for the case study, however no specific trends can be outlined.

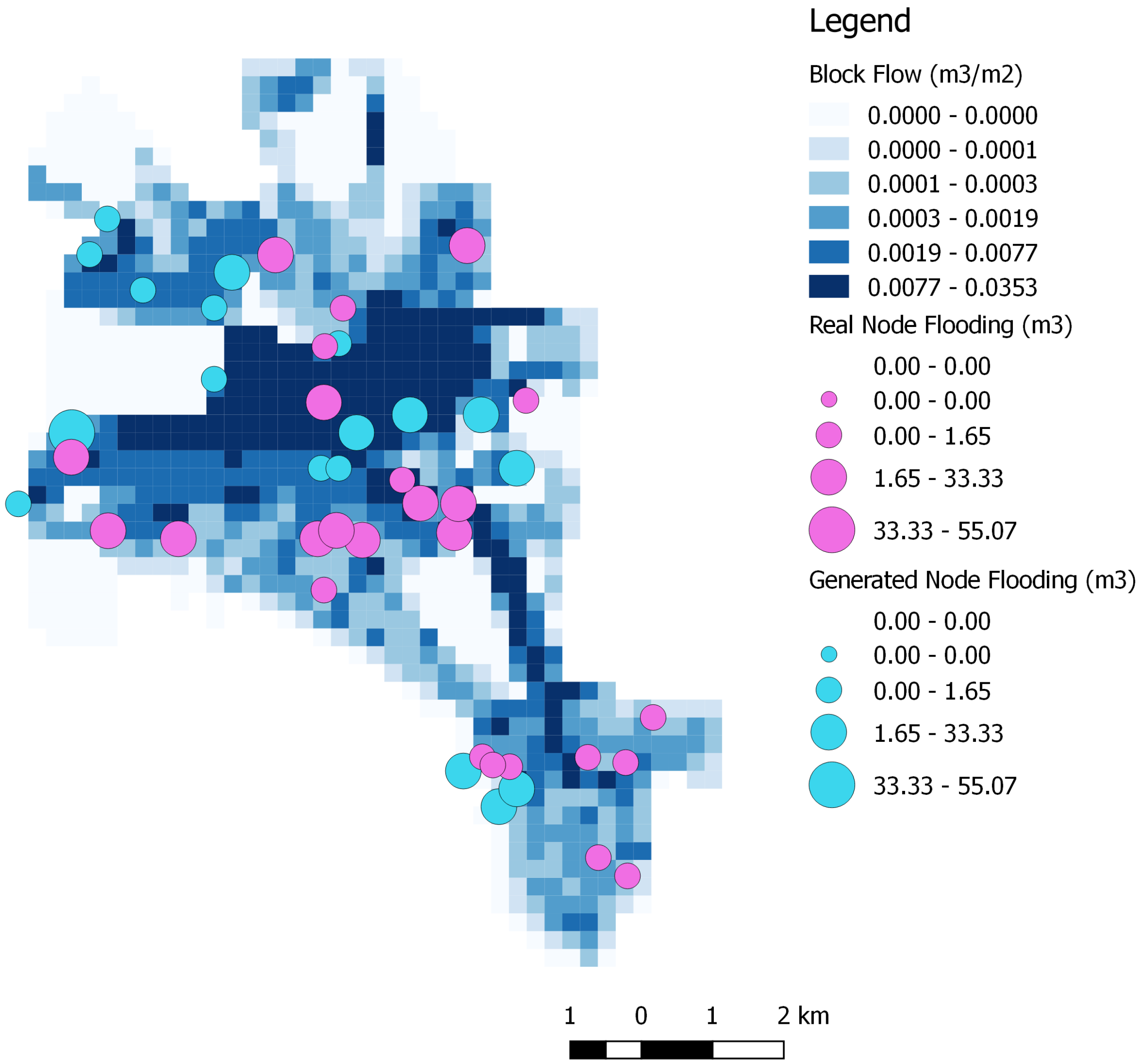

Figure 4.32: Flooded nodes in the 57km^2 case study, the flood volumes are represented by different node sizes

Pipes in Surcharge Diameters Distribution

Pipe surcharge by diameter distribution is shown in figure 4.33. The KL divergence between the real and generated network is 0.174. From figure 4.33, it can be seen that highest proportion of the pipes in surcharge has 0.225m diameter. From figures 4.28 and 4.29 it can however be seen that the pipes in surcharge are concentrated long the min the trunk of the network, showing that surcharge is occurring in portions of the main trunk designed at the limit of their capacity.

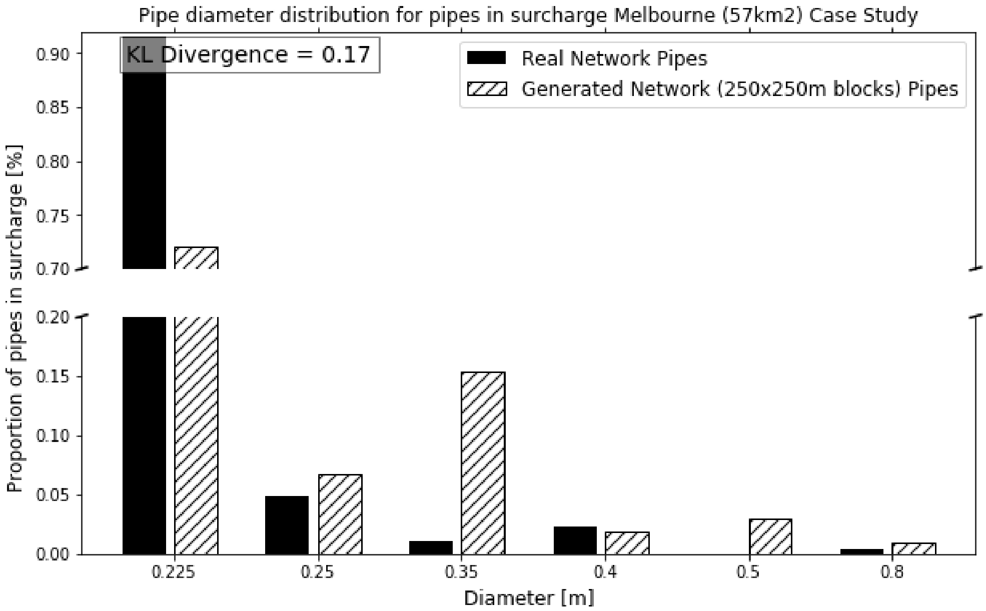

Figure 4.33: Pipe diameter distributions for real and generated networks for the 57km^2 case study

4.5. Low Flow Conditions

The low flow conditions simulated and analysed as specified in the methodology are reported below.

4.5.1. Port Phillip 7km^2 Case Study

In the real network, sedimentation occurs in 35% of the network length, while in the generated network (250 blocks) it occurs in 45% of the network length, for the 500 blocks it occurs for 100% of the pipes. The pipes in sedimentation is shown in figures 4.34 and 4.35 below, the pipes prone to sedimentation are shown in red.

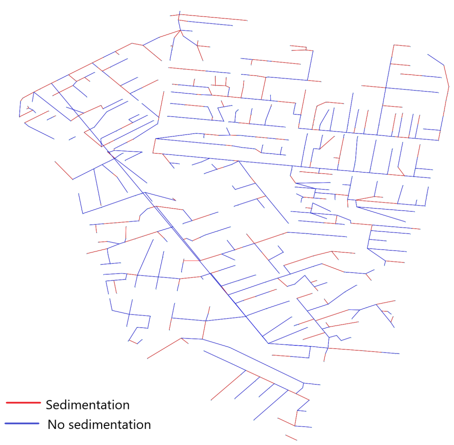

Figure 4.34: Real Network Sedimentation (7km^2)

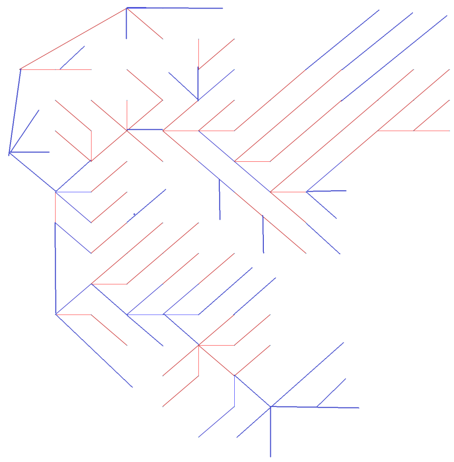

Figure 4.35: Generated Network Sedimentation (7km^2 250x250blk)

Pipes with Sedimentation Cumulative Flow Percentage Distribution

The distribution of pipes at risk of sedimentation by portion of cumulative flow was calculated for the real and generated (250mx250m) network, the (500x500m) network was excluded from the analysis as all pipes appeared to be at risk of sedimentation regardless of their location in the network and cumulative flow. Figure 4.36 shows the distribution of cumulative flows within the pipes which are classified as at risk of developing sedimentation. The KL divergence between the real and generated distribution is of 0.45.

Velocities

The velocities and lengths distributions are shown in figures 4.37, 4.38 . From the two figures, it can be observed that the velocity distributions are relatively similar for both case studies, with the generated network showing higher velocities than the real network. However, the lengths distributions are significantly different, showing that for the generated case study the lowest length being 250m, therefore the lengths with lower velocities are much higher for the generated network.

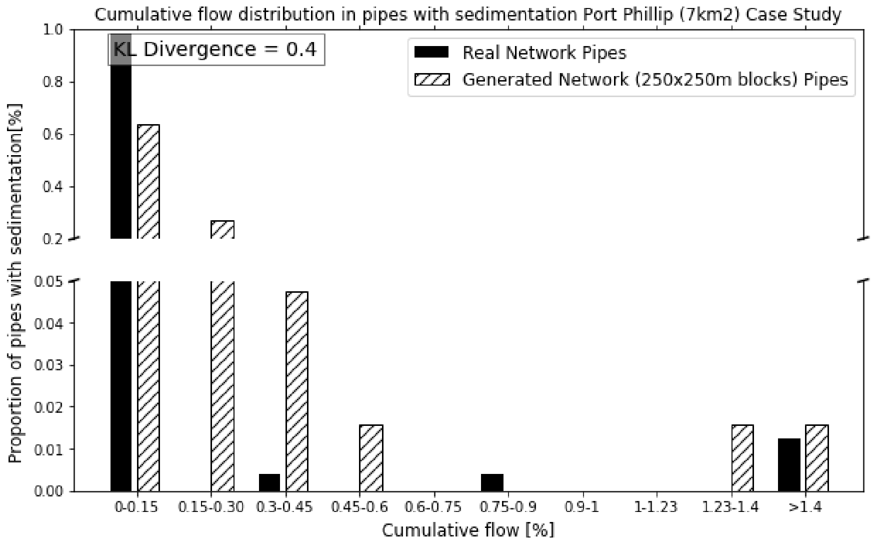

Figure 4.36: Cumulative flow percentage distribution in pipes classified as at risk of sedimentation for the 7km^2 case study real and generated networks (250x250m)

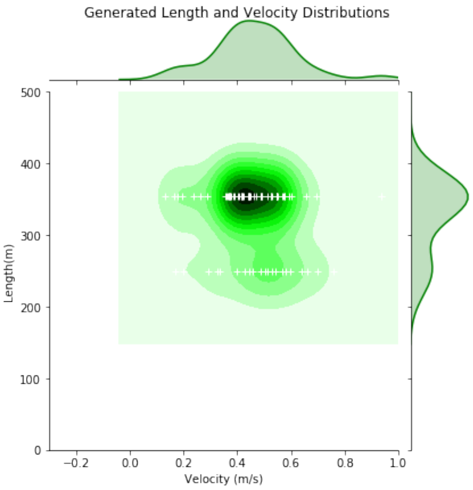

Figure 4.37: Velocity and Length Distribution Generated Network (250x250 7km^2)

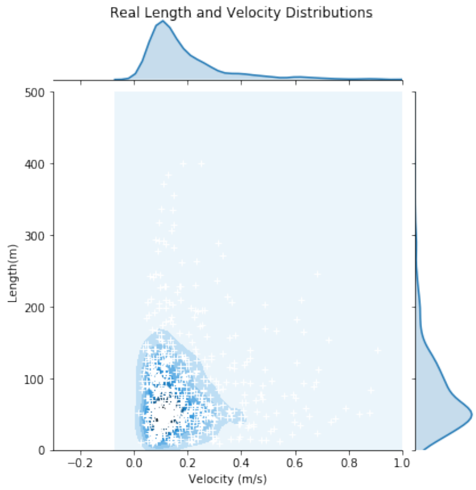

Figure 4.38: Velocity and Length Distribution Real Network 7km^2)

4.5.2. Melbourne 57km^2 Case Study

For the 57km^2 case study, in the real network, it is estimated that 22% of the pipes will develop sedimentation. In the generated network, 35% of the pipes could develop sedimentation (according to the set standards). The pipes in sedimentation is shown in figures 4.39 and 4.40 below, the pipes prone to sedimentation are shown in red.

Figure 4.39: Real Network Sedimentation (57km^2)

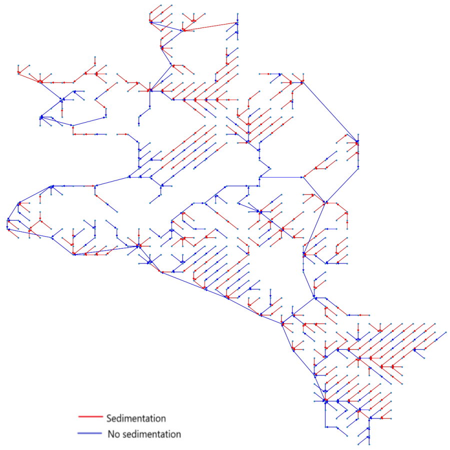

Figure 4.40: Generated Network Sedimentation (57km^2 250x250blk)

Pipes with Sedimentation Cumulative Flow Percentage Distribution

The distribution of pipes at risk of sedimentation by portion of cumulative flow was calculated for the real and generated (250mx250m) network. Figure 4.41 shows the fraction of pipes in sedimentation by percentage of cumulative flow. It can be observed that of the pipes at risk of developing sedimentation, the majority carry a very low percentage of the cumulative flow in the network (0-0.15%). This indicates that the majority of the pipes at risk of sedimentation are located at the network extremities, which tend to have low cumulative flows. This is confirmed by the data shown in figures 4.40 and 4.39. The KL divergence between the real and generated distribution is of 0.02.

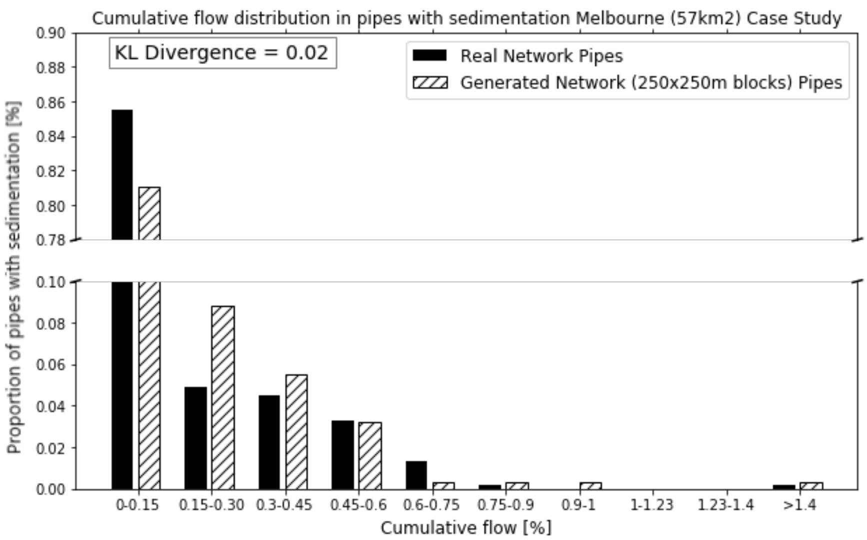

Figure 4.41: Cumulative flow percentage distribution in pipes classified as at risk of sedimentation for the 57km^2 case study real and generated networks (250x250m)

Velocities

The velocities and length distributions are shown in figures 4.42, 4.43 for the generated and real networks respectively. From the plots, it can be observed that the velocities distributions are comparable for the generated and real networks, however the generated network shows much longer pipes at low velocities compared to the real network. This is due to the resolution of the generated network which is much more coarse than for the real network, with a minimum pipe length of 250m.

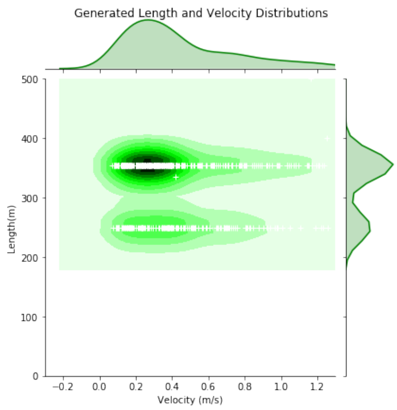

Figure 4.42: Velocity and Length Distribution Generated Network (250x250 57km^2)

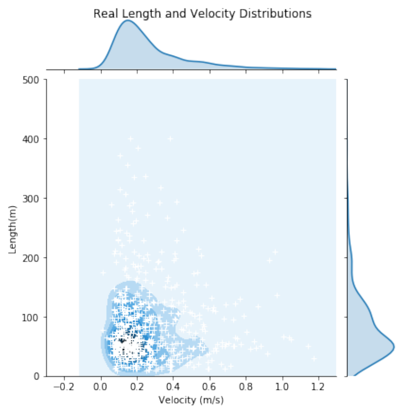

Figure 4.43: Velocity and Length Distribution Real Network 57km^2)

4.6. Computational Times

The computational times for the simulations ran in this study are reported below for the Melbourne (57 km^2) and Port Phillip (7km^2) case study in figures 4.4 and 4.3 respectively. Computational times are observed to be significantly lower for both the case studies and under both simulating methods (Kinematic and Dynamic wave). For the 57km^2 case study, computational tie is reduced by a maximum factor of 10. For the 7km^2 case study, for the 250x250m simplification, a maximum reduction in computational time by a factor of 2.5 is observed. For the 500x500m case study, computational time is reduced by a maximum factor of 10.

	Real Network	Generated Network (250x250m)	Generated Network (500x500m)
Dynamic Wave	0.1s	0.04s	0.01s
Kinematic Wave	0.07s	0.03s	0.025s

Table 4.3: Computational times for the 7km^2 case study simulations

	Real Network	Generated Network (250x250m)
Dynamic Wave	1.47	0.14s
Kinematic Wave	0.26s	0.06s

Table 4.4: Computational times for the 57km^2 case study simulations

A comparison between computational time and errors is shown in figure 4.44 below. It can be observed that a trade-off exists between error and computational time savings for both the 57km^2 and 7km^2 case study. For example, the high flow simulations for the 57km^2 shows the highest computational time saving (90%) and the highest error (26%), while the high flow simulation of the 7km^2 case study shows the lowest error (0.2%) and the lowest computational time saving (50%). In general, for the 57km^2 higher errors, but also higher computational time savings are observed.

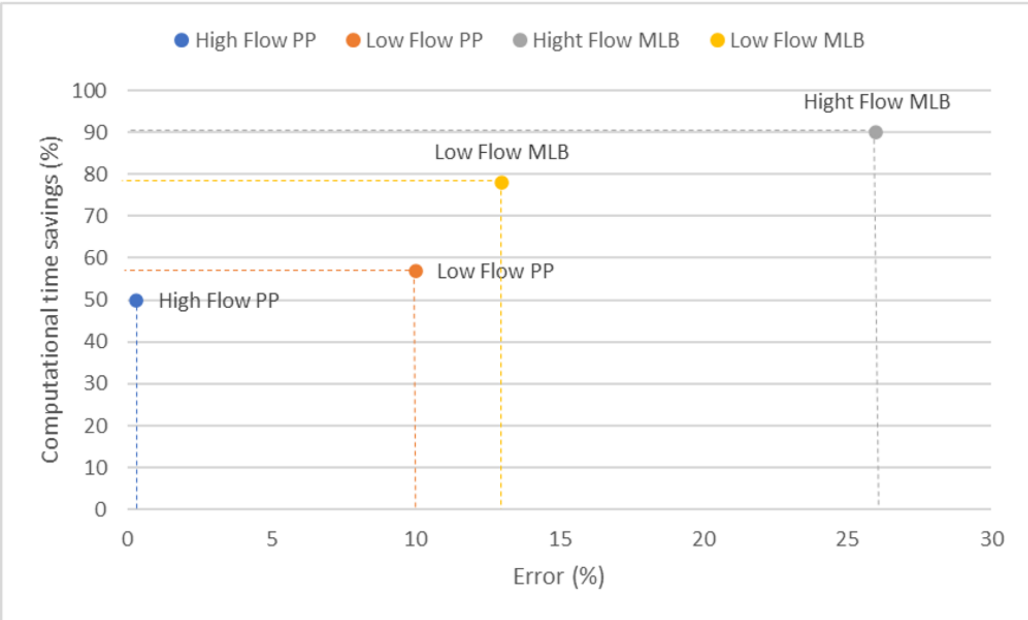

Figure 4.44: Error and Computational time comparison for the Melbourne 57km^2 case study (MLB) and Port Phillip 7km^2 case study (PP). Both low flow (kinematic wave) and high flow (dynamic wave) conditions are considered. For high flow conditions error is calculated as the percentage difference in flood volume between the real and generated case study. For low flow conditions error is calculated as percentage difference in estimated pipes at risk of sedimentation in the generated and real network.

5
Discussion

The outcomes of these analysis are discussed below.

For the typical flow conditions, it was found that the generated networks with 250x250m blocks predict higher pumping power than the real networks, although the pumping heights tend to be lower. Moreover, the number of pumps in the real networks is consistently higher than in the generated network for both the 7km^2 and 57km^2 case studies. This, along with the consideration of average pipe lengths, which are shorter for the real network, leads to the possible conclusions that (1) the pumping power in the real network is reduced due to the shorter length of the rising pipes and (2) the real network has a higher number of smaller, less energy consuming, pumps due to its finer resolution. Interestingly, the 500x500m resolution network for the 7km^2 case study was found to have a lower energy consumption than both the real and generated 250x250m network, however, this was found to be due to the neglect of land depressions and slopes which are overlooked at more coarse resolutions. Similarly, lower pumping heights for the generated 250x250m networks are likely to be due to the neglect of local changes in topography which are only recognized by finer resolutions.

The time of concentrations were calculated for the 7km^2 case study networks and the 57km^2 case study networks, showing significant differences in quantified travel times. For the 7km^2 case study, time of concentration was higher for the simplified network, while for the 57km^2 case study, the opposite phenomenon was observed. This is however consistent with the quantification of average path lengths, which show longer paths for the real network for the 7km^2 case study, while for the 57km^2 case study the generated network appears to have longer average path lengths. This is also consistent with the node degrees presented in the results section which show that for the generated network in the 57km^2 case study, a higher proportion of nodes in the generated network have high degrees (5 or higher) than for the real network, showing a less branched topology which minimizes total network length but not path length from extremity to outfall. This effect is not observed in the smaller 7km^2 case study, and KL divergence shows a closer representation of real network concentration times in the 7km^2 case study (KL divergence = 0.1 and 0.4 for the 500x500m and 250x250m respectively) compared to the 57km^2 case study (KL divergence = 0.9). The best representation of time concentration is given by the 500x500m blocks generated network (KL divergence=0.1). The times of concentration differences can also be observed in the plotted hydrographs for the full networks and for smaller portions of the network, for the 7km^2 case study, the flow distributions are sufficiently close with NSE$>$0.5, while for the 57km^2 case study the flow distributions are less well represented NSE$<$0.5. The east section of 7km^2 the network and north-east section of the 57km^2 were analysed as mostly gravity driven portions of the network to assess possible impacts of the pump design method on times of concentration, however, no evidence of this was observed as the behaviour of the network sections follows that of the entire network (analysed at the outfall).

At high flow conditions, both for the 7km^2 and for the 57km^2 case study the generated networks (with 250x250m blocks) predict higher surcharge than the real network. In the 7km^2 case study, the real network percentage surcharge is 0.8% of the real network length, for the generated network (250x250m) surcharge is observed in 7.2% of the system length, and in 8% of the network length for the generated network (500x500m). For the 7km^2 case study, figures 4.21 and 4.22 show that surcharge is concentrated in the main trunk of the network. For the 57km^2, 8.9% of the network length is in surcharge, while for the generated (250x250m) network, 14.4% of the pipes length is in surcharge. Figure 4.28 and 4.23 show that in the 57km^2 case study surcharge is more distributed. The plot of pipe surcharge by pipe diameter for the 57km^2 case study shows that smaller pipes tend to be in surcharge more frequently than larger pipes in the main trunks for both the real and generated networks. This observation, along with the fact that no flooding occurs under typical design conditions, leads to the possible conclusion that some of the smaller pipes might be designed at the limit of their capacity for the design flow hence causing flooding under more extreme time variation patterns. Generally, for the 57km^2 case study the surcharge in the real and generated network seems to be localised in comparable areas of the network where estimated flows are higher (the north-east portion of the network and south to north west along the main pipes). Given that the diameters distributions for the 7km^2 and 57km^2 case study are similar, it can be deduced that the higher surcharge, depends on the generated network's reduced storage capacity compared to the real network.

Similarly, the flood node frequency is higher for the generated networks than for the real network for both the 7km^2 and 57km^2 case study. As seen in figures 4.30, 4.24, 4.25, 4.40, node flooding largely follows the patterns of conduit surcharge, as expected, therefore the locations of node flooding for the generated network are, like conduit surcharge, similar to those of the flooded nodes in the real network.

The flooding volume, follows similar trends to pipe surcharge and similar behaviour between the 57km^2 and 7km^2 case study. For the 7km^2 case study, flooding is 2% of the incoming flow for the real network, 2.3% of the incoming flow for the generated (250x250m) network, and 4% of the incoming flow for the generated (500x500m) network. For the 57km^2 case study, the differences are more drastic, 26% of the incoming flow is lost to flooding for the real network, while 48% of the incoming flow is lost to flooding in the generated (250x250m) network. The increased flooding in the generated network can be explained by the reduced capacity of the generated network compared to the real network, due to shorter overall length and similar diameter distributions. Reduced capacity due to shorter network length also explains the higher flooding in the generated (500x500m) network, which is 25km long, compared to the generated (250x250m) network which is 44km long. However, reduced network length alone does not explain the drastic increased flooding in the 57km^2 case study. The increase in flooding in this case can, on the other hand, be explained as a combination of reduced network capacity and significantly longer times of concentration in the generated network compared to the real network due to longer on average path lengths for the generated networks.

Sedimentation has also been evaluated by percentage of network length at risk of sedimentation. The study shows that the percentage of pipes in sedimentation are higher for the generated networks than for the real network for both the 57km^2 and 7km^2 case study. For the 7km^2 case study, 35% of the real network is estimated to be at risk of sedimentation, while the estimated value of sedimentation risk in the generated networks is 45% and 100% for the (250x250m) and (500x500m) resolutions networks respectively. A comparable trend is observed in the 57km^2 case study, where 22% of the real network length is calculated to be at risk of sedimentation and 35% of the generated (250x250m) network length is at risk of sedimentation. Figures 4.36 and 4.41 show that for both case studies, the majority of the pipes which are estimated to be at risk of sedimentation are found to be those carrying less than 0.15% of the network flow. The prevalence of sedimentation in pipes carrying small percentages of cumulative flows is consistent with the information showed in figures 4.34, 4.35, 4.39 and 4.40, which indicates most pipes in sedimentation occurs at network extremities.

The reason for the higher percentage of estimated pipe length at risk of sedimentation in the generated networks was investigated by analysing the distributions of velocity and pipe length for pipes in the real and generated networks for both case studies. Figures, 4.37, 4.23 and 4.42 show that on average, the velocities in the generated networks, both in the 7km^2 and 57km^2 case studies and for both the (250x250m) and (500x500m) blocks, are higher than for the real networks. Moreover diameters distributions in the real and generated networks are comparable, and the generated network has lower storage capacity (pipe volume) as accentuated by the high flow condition indicators. However, from figures 4.37, 4.23 and 4.42, it can be observed that when low velocities occur in the generated networks, the length of the pipes experiencing low velocities is much higher than the length of pipes experiencing low velocities in the real network (for both case studies). Therefore, it can be hypothesised that the difference in average pipe length between real and generated networks is the main factor affecting the sedimentation percentage indicator accuracy. This is due to the fact that for the analysis of depth, velocity and τ, average values of velocities and depths for each pipes were used. This therefore leads to longer pipes being classified as completely at risk of sedimentation where in reality only a portion of the pipe might not exceed the critical parameters. This is supported by the finding that for the 500x500m simplification, 100% of the network is assumed to be at risk of sedimentation.

The distributions of the percentage cumulative flows of pipes with sedimentation risk are compared between case studies by comparing KL divergence. The 7km case study shows a KL divergence between the real and generated (250x250m) networks distribution equal to 0.4, while the KL divergence between the real and generated (250x250m) networks distributions for the 57km^2 case study is 0.02. It can be concluded that sedimentation is better represented by the larger scale case study than the small case study. This is hypothesised to be due to the reduced influence of whole pipe length sedimentation classification on networks with greater overall length.

In summary, with respect to hydraulic indicators in case study comparison, it is found that times of concentration and flood volumes are better predicted in the 7km^2 case study than in the 57km^2 one. Sedimentation is better represented by the 57km^2 case study. Surcharge and sedimentation trends (by pipe diameter or design carrying capacity) are better represented than absolute values related to the same phenomena. The differences between calculated indicators are found to be due to differences in network capacity, pipe average length and path lengths (and therefore times of concentration) between the real and generated networks. Comparing the 250x250m simplification and the 500x500m simplification showed that the 500x500m simplification tends to neglect terrain variation hence predicting low energy consumption. With respect to high flow conditions, flooding and surcharge are worse represented by the 500x500m simplification than for the 250x250m simplifications. With respect to sediment, the increased pipe length leads to the overestimation of sedimentation. All the mentioned aspects were observed for the 500x500m simplification in the 7km^2 case study, and all of them excluding sedimentation, were expected to show more drastic discrepancies for the real network at higher scales (as occurred for the 250x250m representations).

With respect to computational time savings, for the 7km^2 case study, the 250x250 simplification reduces computational time by a factor of 2.5 for the dynamic wave simulations and 2.3 for the the kinematic wave simulations, while the 500x500m simplification reduces computational time by a factor of 2.8 and 10 for the same simulations. The time savings are more significant for the 57km^2 case study, where the 250x250m simulation reduces computational time by a factor of 10.5 for the kinematic wave simulations, and by 4.3 for the dynamic wave simulations. In the 57km^2 case study, for the 250x250m simplification, computational times are reduced by higher factors than for the 7km^2 case study despite the lower reduction in network lengths (length is reduced by 28% while in the 7km^2 and by 17% in 57km^2 case study) showing that other factors, including number of pipes and manholes have a higher impact on computational time. Kroll et al.(2010) state that computational time, using the Saint-Venant equations (as done in this study), is not only dependent on the number of pipes, but also on the number of critical structures, especially pumps. This conclusion, supports the findings of this study, which showed that computational time reduction is not proportional to link number reduction, and highlights the need for further research into how to further reduce the number of pumps predicted by the sizing algorithm to approximate the number and behaviour of real pumps in the system [19].

To evaluate the performance of the generated simplified model relatively to other available approaches, the studies found in the literature were used as a point comparison. To the author's knowledge, no studies assessing simplified foul sewers only (and not combined sewers) were found, therefore combined sewer simplifications were used as a comparison instead. Kroll et al. (2010) presented a study comparing simplified network generated through a semi-automated approach, with flow data from the real combined sewer network serving the same area as the generated network. The semi automated approach applied, eliminates side branches in a sewer network, reducing sewer length by 50% for an

$11km^2$ case study area. The simulation time was reduced by 30%, while flooding was overestimated by 10%. The simplification algorithm reduces computational times by 50% overestimating flooding by 0.3% in the $7km^2$ case study, showing possibly better performance in terms of hydraulic performance and computational times for a case study of comparable size. However, it should be taken into account that the hydraulic performance comparison took place with overflow volumes data simulated using a detailed model of the sewer network, including the real control structures and pipes dimensions and elevations, this was not the case for this research, therefore hydraulic performance accuracy is likely to be overestimated, while computational time savings are likely to be underestimated.

A study evaluating the hydraulic performance of an algorithmic generated network was presented by Bluemenstaat et al. (2012), the study, which aimed at algorithmic sewer generation under minimum data requirements, and also compared two case studies with network lengths of 14.4km and 124 km. The 14.4km case study produced a 3% flow balance deviation, while the 124km case study produced a 40% flow balance deviation from the real network observations[6]. The data for the case studies simulated in this research shows a flow balance deviation of 0.3% for the 250x250m simplification and of 2% for the 500x500m simplification for the $7km^2$ case study and of 22% for the 250x250m simplification for the $57km^2$ case study. While the data seems to provide evidence of improved accuracy with the simplification proposed in this case study with respect to flow balance, it is important to note that Bluemenstaat et al (2012) compares the generated network flows with those from the real network's monitoring data, while this study compares the generated network flows with simulated data from an algorithmic sized sewer network assumed to have no control structures (eg. weirs, storage tanks, etc.). Therefore, further research using real networks monitoring data is suggested to evaluate the 'real' accuracy of the algorithmic generated simplified network.

Manual simplification approaches have also been evaluated with respect to flow balance errors and computational time savings. Rouault et al (2008) simplified network layout by grouping all pipes of a certain region by replacing them with a couple of synthetic pipes, applying this methodology, computational time was reduced by 41% and overflow volume deviation was measured to be 4% [31]. Fischer et al (2009) manually removed side branches of the network considered to be of lower importance, reducing computational times by 36% and measuring a volumetric error of 1% [16]. This shows that computational times are more reduced by the simplification proposed in this research, and that flooding error, for the $7km^2$ case study are possibly lower. Moreover, both literature approaches heavily modify network topology (as shown in figure 5.1) to achieve computational time savings, hence not allowing for spatial variations to be assessed, and only allowing global phenomena (such as flooding) to be assessed. A summary of the findings regarding computational time savings and errors is shown in figure 5.2

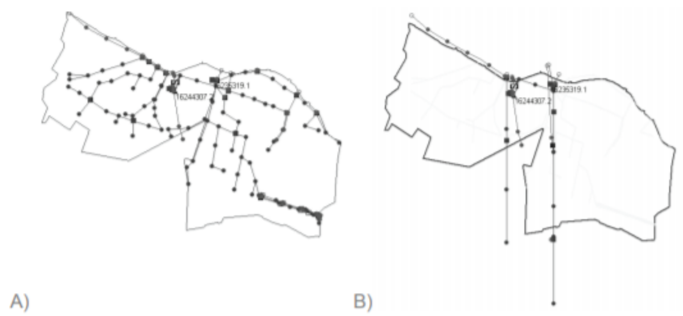

Figure 5.1: Network structure of the real (A) and simplified (B) networks from Fischer et al, 2009

With respect to low flow conditions, Mannina et al (2012), studied sedimentation comparing a detailed model to a simplified conceptual one. The model showed flow residuals within 30% . The simplified model, provided sedimentation height data at one point of the system for different points in time, while, with similar computational times, the detailed model provided sedimentation data at one point in time but for all pipes in the network. With respect to Mannina et al's (2012) study, the approach proposed in this research allows sedimentation risk to be estimated at different points and times for the simplified network, producing relatively accurate results[21]. However, sedimentation in this study was assessed

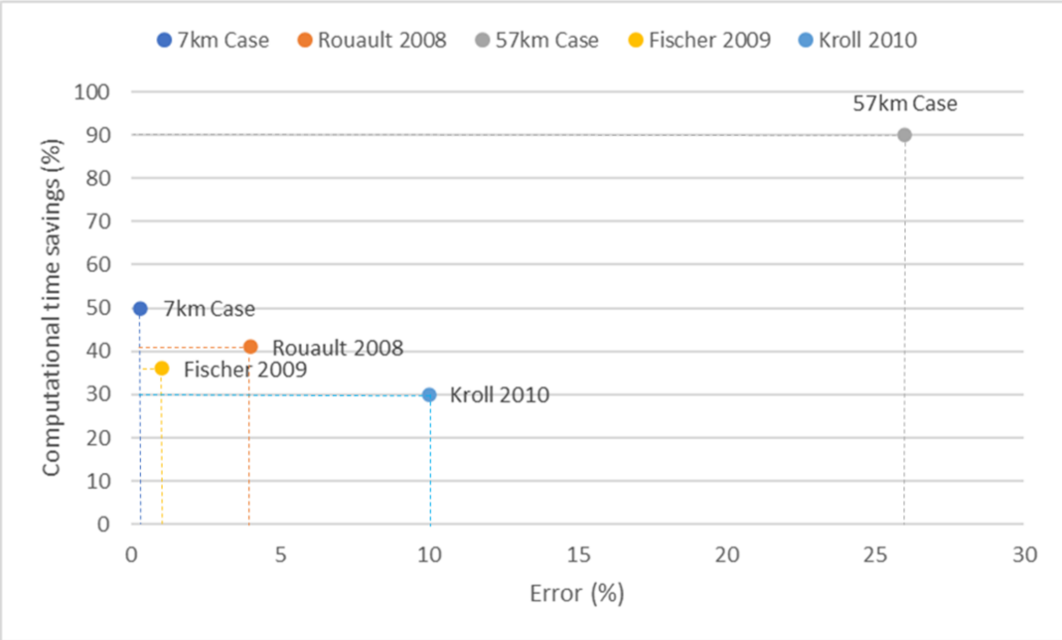

Figure 5.2: Comparison of simplification studies with respect to computational time savings and error. Error is intended as percentage difference between estimated or observed real and generated flows.

generically as a 'risk' and not as a specific quantity, therefore it is not certain whether a more precise analysis using the proposed simplification would still provide representative results.

Overall, from literature studies comparison, it can be concluded with certainty that the computational time savings obtained with the simplification proposed in this study are higher than those found in the literature using other simplification approaches. With respect to accuracy, it is found that for the 7km^2 case study both high and low flow conditions are well represented compared to other simplification studies and their relative flow balance errors. For the 57km^2 case study, limited comparable case studies were found. Bluemenstaat et al. (2012), observed higher overflow volume variations for a 124km case study than those observed for the 57km^2 case study investigated in this research. From comparison with other literature approaches, it is found that a major limitation of the current study is however not comparing the simplified network data with real network data, or at least a calibrated model of the real network. Moreover, a further limitation was observed to be the fact that no data was available on the operation of control structures, especially pumping stations, whose impact on hydraulic performance and computational times is found to be highly relevant.

6 Conclusions

The aim of this study is to assess whether a simplified algorithmic generated network is able to represent the hydraulic behaviours and planning performance indicators of a real network. Results regarding error and computational time savings were evaluated to answer the research questions.

From the discussion of the results, it is found that the proposed algorithmic generated simplified sewer network can, for small case studies (7km^2) predict high and typical flow conditions and behaviours with sufficient accuracy compared to that of other commonly used simplification techniques. For the large case study (57km^2), the representation of high and typical flows conditions and behaviours is worse than for the small case study, although surcharge and flood frequency is closely predicted spatially, the absolute values of flooding and times of concentration are predicted with high error values. However, similar trends, whereby performance accuracy worsens with case study size, is also reported with other simplification models, which show similar flow balance errors [6, 16, 19, 31], leading to the conclusion that the representation evaluated in this study can be considered acceptable.

The estimation of sedimentation, on the contrary, is found to be more accurate for larger case studies (57km^2) than for the smaller ones, which are more sensitive to the assumptions made (full length at 100% risk of sedimentation). Sedimentation is found to be statistically accurate both in terms of absolute values and spatial locations (evaluated through hierarchy), however no comparison could be made to performance of other methods or required accuracies as no such data was available.

Computational time was found to be greatly improved by the proposed network generation method, with time savings exceeding those of all the reviewed simplification methods. As the main objective of the network generation is to reduce computational time, it is found that the proposed method is effective in favouring computational efficiency, especially at larger scales (57 km^2 case study). Given the necessary computational time reduction outlined in the Introduction of this report, it is found that the 57km^2 case study satisfies the computational times savings required (factor of 10). For the 7km^2 case study, the 500x500m provides the required computational time savings, although at a smaller scale computational time savings might not be required. It is important to note that computational times are reported for the hydraulic performance assessment only, which in general has low computational time. However, further assessments are required to fully assess the generated networks for the intended use of the simplification, therefore it is expected that computational times for a full assessment would be much higher, making reductions in computational time essential.

Finally, whether the generation method can be used as a representation of the real network largely depends on the intended use of data obtained from the model itself. The indicators quantified in this case study tend to have relative meaning rather than an absolute one, as they can be used to compare different networks, case study areas and simplification resolutions. Therefore, the presented analysis essentially proves that the generated simplification can be used for comparison of various networks producing results which are consistent with the assumptions made and approximate (to varying accuracy) the behaviour of the real network. Therefore, it is concluded that the estimated absolute values and indicators calculated should not be used for as stand-alone values for design purposes, but rather as relative values for comparison of network schemes or topologies. Since the intended use of the model established in the introduction is to enable exploratory modelling, which includes the evaluation of alternatives relative to each other, it can be stated that the hydraulic performance is sufficiently represented by the generated network with respect to this objective.

7

Recommendations and Future Work

7.1. Recommendations

The findings reported in this study aim to inform the use of the generated simplified network as a representation of the real network for the comparison of alternative paths. As such, some recommendations are made on the possible applications of the assessed networks:

- It is found that the computational time savings are higher for the 57km^2 case study, which is able to reduce the time by a factor of 10, deemed as the required time reduction factor. For this case study, the use of the simplification method is necessary and justified. For the 7km^2 case study, computational time savings are found to be lower, as are simulation times. For this case study, it is suggested that the low simulation times might not warrant the use of a simplification method to reduce computational time. Overall, it is recommended that the simplification should be used for city-wide case studies.
- Regarding typical flow conditions, it is found that times of concentration and node degrees network sufficiently represent differences in network topologies. The impacts of such differences can be used to predict the differences between real and generated network behaviours.
- For high flow conditions, times of concentration and network capacity heavily affect flood volumes, and can thus be used to predict behaviours of the real and generated networks.
- In estimating sedimentation, it is found that pipe length affects the way in which sedimentation risk is calculated. It can be expected that in all cases sedimentation will be overestimated by the generated networks, this can be taken into account when evaluating sedimentation estimates.
- In comparing the studies simplification and generation method to other case studies, the computational time savings and errors seem to be generally lower for the method chosen in this research. As such it is recommended that this simplification should be further applied to real case studies to further support the robustness of the findings.
- The spatial distribution of pipes in surcharge, flooding and pipes with sedimentation is highly dependant on network topology. As the generated topology does not necessarily aim at representing that of the real network, it is suggested that global (network wide) findings and trends (pipe diameter distributions, cumulative flow percentages distributions) should be used for future assessments and comparisons rather than location-based indicators (flooded node locations etc.).
- For this analysis, only simulated data is considered for the real network. In reality, it has been shown that in some case hydraulic simulation cannot always accurately predict the impacts of new technologies, even with the use of detailed models [23]. Unexpected problems including sedimentation, odour problems, and ecosystem changes are hard to predict through modelling alone. Therefore, the 'success' of a real network representation should be also based on real observed phenomena. Transitions for the real network should in general, be based not only on modelling but also on real life application. It is therefore suggested that the simplification assessed

in this study should be used as a first step in the decision making process rather than as a way to draw final conclusions.

7.2. Future Work

Throughout the indicator definition, calculation and analysis of the results, a few areas of improvement were found in the proposed approach and existing model capability which could be investigated in further studies:

- Pumping energy was observed to be a crucial factor in evaluating hydraulic performance. Lack of pumping data thus is found to hinder the reliability of the results. It is thus suggested that such data should be acquired to investigate how pumping impacts network performance. Future research in the network generation could be expanded to investigate how to appropriately preserve pumping and energy consumption characteristics, which is usually done in the other reviewed simplification studies [16, 19, 31];
- Other control and storage structures were not considered in both the real and generated networks. Presence of these structures in the real system might have heavy impacts on flows in the network, and should therefore be integrated in the generated network where such data is available;
- Sedimentation estimation was simplified by assuming average velocities, flows, and depths throughout conduits to evaluate critical parameters. This assumption is likely to overestimate sedimentation risk in the generated network. In future studies this could be changed by calculating depths and flows at various points in each conduit;
- The hydraulic performance assessment is based on the assumption that the real and generated network are designed according to the same principles. This assumption was made due to lack of data for the real network and evidence that the sizing algorithm produced accurate enough results. However, the absence of a real model and/or of real flow data undermines slightly the findings of the study and limits its scope to comparing real and generated network topologies more than whole designs;
- Due to time constraints, only 2 case study areas were evaluated. While the analysis allowed to find potential explanations for the observed phenomena, these could be strengthened by additional case study comparisons. In particular, different terrain profiles, land use distributions, and network sizes could be compared;
- Due to lack of data and time constraints, only three simulation scenarios were tested (high, typical and low) for a 1 day period each. Especially for sedimentation, most studies refer to longer simulations (at least 7days) to calculate sedimentation risk and flow balance errors in the reviewed studies typically use monthly or annual simulations. To give a more realistic estimate of flow balance errors, long term simulations should be run where data is available;
- Due to time constraints, only the 'block' abstraction was used to delineate network topology in the infrastructure module. This was found to have considerable impacts in times of concentration and network path length, especially with the 57km^2 case study. Other abstraction units have been developed (patches) or are in development (hexagons) in UrbanBEATS, their impact on possibly improving topology representation could be investigated.

A

Swmm Model Assumptions

A.1. Typical Flow Conditions

A.1.1. Dynamic Wave Simulation Assumptions

For the dynamic flow simulations, simulations set up was defined using SWWMM guidelines from Rossman (2015). The simulation options are summarised in figure A.2 below. The Darcy-Weisbach equation is used when pressurized flow occurs.

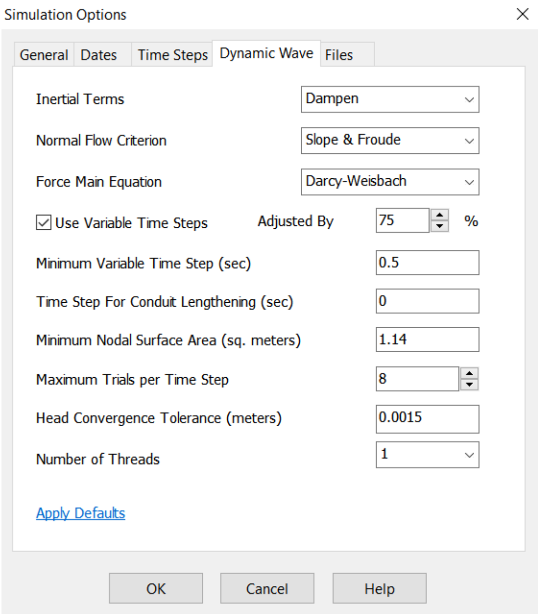

Figure A.1: Dynamic Wave simulation options set for high flow simulations based on Rossman (2015)

A.1.2. Time Patterns

The time patterns used for the typical flow conditions for all land uses and are summarised below.

weekly_var_avg	HOURLY	0.8 0.9 1.1 1.1 1.2 1.2
weekly_var_avg		1.2 1.2 1.2 1.2 1.2 1.2
weekly_var_avg		1.2 1.2 1.1 1.1 1.0 0.9
weekly_var_avg		0.8 0.7 0.7 0.6 0.6 0.7

A.2. High Flow Conditions

A.2.1. Dynamic Wave Simulation Assumptions

For the dynamic flow simulations, simulations set up was defined using SWWMM guidelines from Rossman (2015). The simulation options are summarised in figure A.2 below.

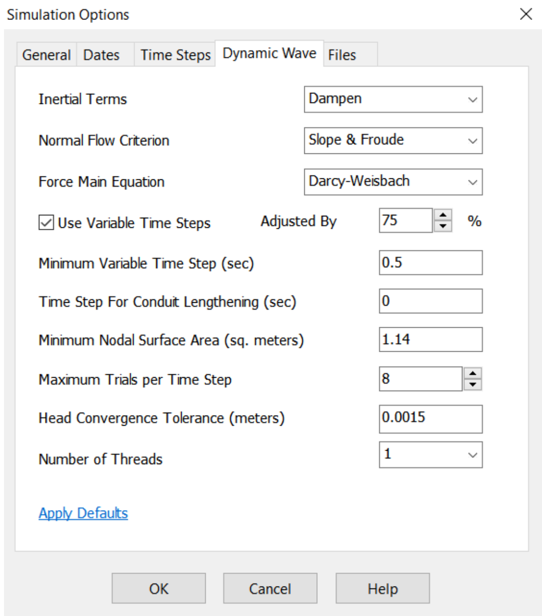

Figure A.2: Dynamic Wave simulation options set for high flow simulations based on Rossman (2015)

A.2.2. Time Patterns

The time patterns used for the high flow conditions include residential (weekly_var_res) and commercial/industry/office patterns (weekly_var_comm). These are assigned to nodes based on predominant land use and are summarised below.

weekly_var_res	HOURLY	0.2 0.2 0.1 0.1 0.1 0.4
weekly_var_res		0.6 1.3 1.5 1.4 1.3 0.8
weekly_var_res		0.8 0.8 0.8 0.8 1.2 1.5
weekly_var_res		2.0 2.1 2.2 2.1 2.5 0.8

weekly_var_comm	HOURLY	0.0 0.0 0.0 0.0 0.0 0.0
weekly_var_comm		0.0 1.0 1.0 1.0 1.0 1.0
weekly_var_comm		1.0 1.0 1.0 1.0 1.0 1.0
weekly_var_comm		1.0 1.0 0.0 0.0 0.0 0.0

A.3. Low Flow Conditions

A.3.1. Kinematic Wave Simulation Assumption

The options used for kinematic wave simulations are summarised in Figure A.3 below. The routing time step used is of 60s, found, through trial and error to provide numerical stability and reduce computational time to a sufficient extent. The infiltration model chosen is not relevant since the assessment considers a foul sewer.

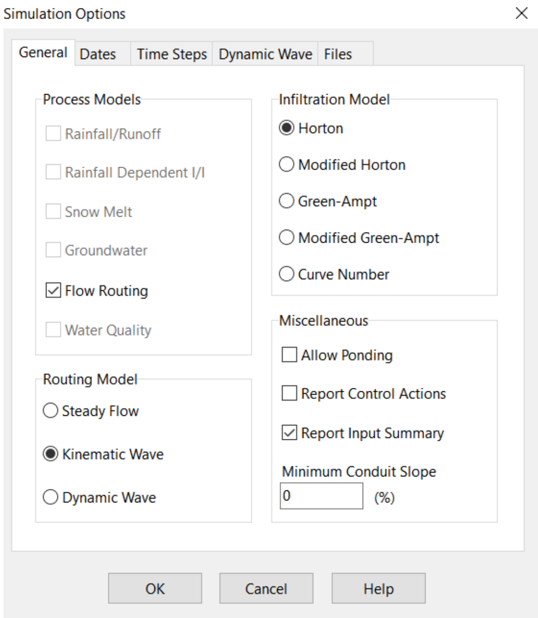

Figure A.3: Kinematic Wave Simulations options

A.3.2. Time Patterns

weekly_var HOURLY 0.6 0.6 0.6 0.6 0.6 0.6
weekly_var 0.6 0.6 0.6 0.6 0.6 0.6
weekly_var 0.6 0.6 0.6 0.6 0.6 0.6
weekly_var 0.6 0.6 0.6 0.6 0.6 0.6

Bibliography

[1] Australian Government Australian Bureau of Statistics. Census of population and housing - undercount (cat. no. 2940.0). URL https://www.abs.gov.au/AUSSTATS/abs@.nsf/DetailsPage/2914.02006.

[2] Amin E. Bakhshipour, Milad Bakhshizadeh, Ulrich Dittmer, Ali Haghighi, and Wolfgang Nowak. Hanging gardens algorithm to generate decentralized layouts for the optimization of urban drainage systems. Journal of Water Resources Planning and Management, 145(9):04019034, 2019. doi: 10.1061/(asce)wr.1943-5452.0001103.

[3] Silja Baron, Jannis Hoek, Inka Kauffman Alves, and Sabine Herz. Comprehensive scenario management of sustainable spatial planning and urban water services. Water Science Technology, 73.5, 2016. doi: doi:10.2166/wst.2015.578.

[4] Adriano Battilani, Michele Steiner, Martin Andersen, Soren Nohr Back, J. Lorenzen, Avi Schweitzer, Anders Dalsgaard, Anita Forslund, Secondo Gola, Wolfram Klopmann, and et al. Decentralised water and wastewater treatment technologies to produce functional water for irrigation. Agricultural Water Management, 98(3):385402, 2010. doi: 10.1016/j.agwat.2010.10.010.

[5] Akica Bhari. Integrated urban water management. Global Water Partnership Technical Committee (TEC), 2012. URL https://www.gwp.org/globalassets/global/toolbox/publications/background-papers/16-integrated-urban-water-management-2012.pdf.

[6] Frank Blumensaat, Martin Wolfram, and Peter Krebs. Sewer model development under minimum data requirements. SpringerLink, Jul 2011. URL https://link.springer.com/article/10.1007/s12665-011-1146-1.

[7] Rebekah Brown, Richard Ashley, and Megan Farrelly. Political and professional agency entrapment: An agenda for urban water research. Water Resources Management, 25(15):40374050, 2011. doi: 10.1007/s11269-011-9886-y.

[8] David Butler and John W. Davies. Urban drainage, 3rd ed. Spon Press, 2010.

[9] Maria Cardoso, Sérgio Coelho, Rafaela Matos, and Helena Alegre. Performance assessment of water supply and wastewater systems. Urban Water Journal, 1(1):5567, 2004. doi: 10.1080/15730620410001732053.

[10] Paul Davis, Steven Bankes, and Michael Enger. Enhancing strategic planning with massive scenario generation theory and experiments, 2007. URL https://www.rand.org/content/dam/rand/pubs/technical_reports/2007/RAND_TR392.pdf.

[11] Fjalar de Haan, Briony Rogers, Rebekah Brown, and Ana Deletic. Many roads to rome: The emergence of pathways from patterns ofchange through exploratory modelling of sustainability transitions. Environmental Modelling Software, May 2015.

[12] Ana Deletic, Kefeng Zhang, Behzad Jamali, Adam Charette-Castonguay, Martijn Kuller, Veljko Prodanovic, and Peter M. Bach. Modelling to support the planning of sustainable urban water systems. New Trends in Urban Drainage Modelling Green Energy and Technology, page 1019, Jan 2018. doi: 10.1007/978-3-319-99867-1_2.

[13] Natalia Duque, Peter Bach, Lisa Scholten, and Max Maurer. Simplifying sewer system representation for planning-support modelling. In preparation, 2020.

[14] Sven Eggimann, Bernhard Truffer, and Max Maurer. To connect or not to connect? modelling the optimal degree of centralisation for wastewater infrastructures. Water Research, 84:218231, 2015. doi: 10.1016/j.watres.2015.07.004.

[15] Hind El-Housni, Sophie Duchesne, and Alain Mailhot. Predicting individual hydraulic performance of sewer pipes in context of climate change. Journal of Water Resources Planning and Management, 145(11):04019051, 2019. doi: 10.1061/(asce)wr.1943-5452.0001127.

[16] Anita Fischer, Pascale Rouault, Stefan Kroll, Johan Van Assel, and Erika Pawlowsky-Reusing. Possibilities of sewer model simplifications. Urban Water Journal, 6(6):457470, 2009. doi: 10.1080/15730620903038453.

[17] Nicolai Guth and Philipp Klingel. Demand allocation in water distribution network modelling a gis-based approach using voronoi diagrams with constraints. Application of Geographic Information Systems, 2012. doi: 10.5772/50014.

[18] Sabine Hoffmann, Ulrike Feldmann, Peter M. Bach, Christian Binz, Megan Farrelly, Niki Frantzeskaki, Harald Hiessl, Jennifer Inauen, Tove A. Larsen, Judit Lienert, and et al. A research agenda for the future of urban water management: Exploring the potential of nongrid, small-grid, and hybrid solutions. Environmental Science Technology, 54(9):53125322, 2020. doi: 10.1021/acs.est.9b05222.

[19] Stefan Kroll, Tom Wambecq, Marjoleine Weemaes, Jan Frans Van Impe, and Patrick Willems. Semi-automated buildup and calibration of conceptual sewer models. Environmental Modelling Software, 93:344 – 355, 2017. ISSN 1364-8152. doi: https://doi.org/10.1016/j.envsoft.2017.02.030".

[20] Cedo Maksimovic and J.A. Tejada-Guibert. Frontiers in Urban Water Management: Deadlock or hope. IWA Publishing, 10 2005. ISBN 9781780402659. doi: 10.2166/9781780402659. URL https://doi.org/10.2166/9781780402659.

[21] Giorgio Mannina, A.n.a. Schellart, Simon Tait, and Gaspare Viviani. Uncertainty in sewer sediment deposit modelling: Detailed vs simplified modelling approaches. Physics and Chemistry of the Earth, Parts A/B/C, 42-44:1120, 2012. doi: 10.1016/j.pce.2011.04.003.

[22] David R. Marlow, David J. Beale, and Stewart Burn. A pathway to a more sustainable water sector: sustainability-based asset management. Water Science and Technology, 61(5):12451255, 2010. doi: 10.2166/wst.2010.043.

[23] David R. Marlow, Magnus Moglia, Stephen Cook, and David J. Beale. Towards sustainable urban water management: A critical reassessment. Water Research, 47(20):71507161, 2013. doi: 10.1016/j.watres.2013.07.046.

[24] Rafaela Matos. Performance indicators for wastewater services. IWA Publ., 2010.

[25] Max Maurer, Andreas Scheidegger, and Anja Herlyn. Quantifying costs and lengths of urban drainage systems with a simple static sewer infrastructure model. Urban Water Journal, 10(4): 268280, 2013. doi: 10.1080/1573062x.2012.731072.

[26] Enayat A. Moallemi and Shirin Malekpour. A participatory exploratory modelling approach for long-term planning in energy transitions. Energy Research Social Science, 35:205216, 2018. doi: 10.1016/j.erss.2017.10.022.

[27] Jeffrey P. Newman, Greame C. Dandy, and Holger R. Maier. Multiobjective optimization of cluster-scale urban water systems investigating alternative water sources and level of decentralization. Water Resources Research, 50(10):79157938, 2014. doi: 10.1002/2013wr015233.

[28] Leonie J. Pearson, Anthea Coggan, Wendy Proctor, and Timothy F. Smith. A sustainable decision support framework for urban water management. Water Resources Management, 24(2):363376, 2009. doi: 10.1007/s11269-009-9450-1.

[29] Roni Penn and Max Maurer. Effects of transitions to water efficient solutions on existing centralised sewer systems an integrated bio physical modelling approach. In preparation, 2020.

[30] Lewis A. Rossman. Storm water management model user's manual version 5.1, Sep 2015.

[31] Pascale Rouault, Anita Fischer, Kai Schroeder, Erika Pawlowsky-Reusing, and Johan Van Assel. 11th international conference on urban drainage, 2008.

[32] Robert Sitzenfrei, Michael Möderl, and Wolfgang Rauch. Assessing the impact of transitions from centralised to decentralised water solutions on existing infrastructures integrated city-scale analysis with vibe. Water Research, 47(20):72517263, 2013. doi: 10.1016/j.watres.2013.10.038.

[33] Robert Sitzenfrei, Jonatan Zischg, Markus Sitzmann, and Peter Bach. Impact of hybrid water supply on the centralised water system. Water, 9(11):855, 2017. doi: 10.3390/w9110855.

[34] Wided Ben Tagherouit, Saad Bennis, and Jamal Bengassem. A fuzzy expert system for prioritizing rehabilitation of sewer networks. Computer-Aided Civil and Infrastructure Engineering, 26(2): 146152, 2011. doi: 10.1111/j.1467-8667.2010.00673.x.

[35] Data Vic. Department of environment, land, water and planning regional boundaries - vicmap admin - victorian government data directory. URL https://discover.data.vic.gov.au/dataset/department-of-environment-land-water-and-planning-regional-boundaries.

[36] Natchapon Vongvisessomjai, Tawatchai Tingsanchali, and Mukand S. Babel. Non-deposition design criteria for sewers with part-full flow. Urban Water Journal, 7(1):6177, 2010. doi: 10.1080/15730620903242824.

[37] Melbourne Water. Wastewater inlet hourly flow eastern treatment plant (etp). URL https://data-melbournewater.opendata.arcgis.com/datasets/wastewater-inlet-hourly-flow-eastern-treatment-plant-etp-.

[38] M Winker, S Kunkel, A Davoudi, J Felmedem, H Kerber, J Trapp, and E Schramm. Heat and water recovery from wastewater in a passive house scaling up from building to district level. 52 (1):29, 2015. doi: 10.1016/s0015-1882(15)30041-0.

[39] Sergio Zubelzu, Leonor Rodríguez-Sinobas, Alvaro Sordo-Ward, Alan Pérez-Durán, and Rodolfo Cisneros-Almazán. Multi-objective approach for determining optimal sustainable urban drainage systems combination at city scale. the case of san luis potosí (méxico). Water, 12(3):835, 2020. doi: 10.3390/w12030835.