Modelling and Resilience-based Evaluation of Urban Drainage and Flood Management Systems for Future Cities

Submitted by

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Signature ..........................................................
Abstract

In future cities, urban drainage and flood management systems should be designed not only to reliable during normal operating conditions but also to be resilient to exceptional threats that lead to catastrophic failure impacts and consequences. Resilience can potentially be built into urban drainage systems by implementing a range of strategies, for example by embedding redundancy and flexibility in system design or rehabilitation to increase their ability to efficiently maintain acceptable customer flood protection service levels during and after occurrence of failure or through installation of equipment that enhances customer preparedness for extreme events or service disruptions.

However, operationalisation of resilience in urban flood management is still constrained by lack of suitable quantitative evaluation methods. Existing hydraulic reliability-based approaches tend to focus on quantifying functional failure caused by extreme rainfall or increases in dry weather flows that lead to hydraulic overloading of the system. Such approaches take a narrow view of functional resilience and fail to explore the full system failure scenario space due to exclusion of internal system failures such as equipment malfunction, sewer (link) collapse and blockage that also contribute significantly to urban flooding.

In this research, a new analytical approach based on Global Resilience Analysis (GRA) is investigated and applied to systematically evaluate the performance of an urban drainage system (UDS) when subjected to a wide range of both functional and structural failure scenarios resulting from extreme rainfall and pseudo random cumulative link failure respectively. Failure envelopes, which represent the resulting loss of system functionality (impacts) are determined by computing the upper and lower limits of the simulation results for total flood volume (failure magnitude) and average flood duration (failure duration) at each considered failure level. A new resilience index is developed and applied to link resulting loss of functionality magnitude and duration to system residual functionality (head room) at each considered failure level.
With this approach, resilience has been tested and characterized for a synthetic UDS and for an existing UDS in Kampala city, Uganda. In addition, the approach has been applied to quantify the impact of interventions (adaptation strategies) on enhancement of global UDS resilience to flooding. The developed GRA method provides a systematic and computationally efficient approach that enables evaluation of whole system resilience, where resilience concerns ‘beyond failure’ magnitude and duration, without prior knowledge of threat occurrence probabilities. The study results obtained by applying the developed method to the case studies suggest that by embedding the cost of failure in resilience-based evaluation, adaptation strategies which enhance system flexibility properties such as distributed storage and improved asset management are more cost-effective over the service life of UDSs.
List of publications


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“I searched everywhere, determined to find wisdom and to understand the reason for things” (Ecclesiastes 7:25a New Living Translation)
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Notation

\( \bar{R}_t \) Probability of occurrence of flooding during an UDS’s service life

\( A \) Link cross sectional area (m²)

\( C_L \) Pipe laying cost (£)

\( C_{OM} \) Operations and maintenance cost (£)

\( C_p \) Capital cost of pipes (£)

\( C_{TF} \) Cost of failure (£)

\( d_{f,p} \) Flood depth in a given flooded property (m)

\( D_p \) Pipe diameter (m)

\( d_p \) Pipe laying depth (m)

\( E_t \) Water saving efficiency (%)

\( f_c \) Direct tangible flooding cost (£/ m³ of flooding)

\( f_{D50} \) Number of days when \( f_{D50} > 50\% \) of average daily demand (days)

\( g \) Acceleration due to gravity (m/s²)

\( H \) Hydraulic head (m)

\( i \) Link failure level or number of failed links (-)

\( I_R \) Rainfall intensity (mm/hr)

\( j \) Design life of UDS (yrs)

\( L_c \) Length of open channel section (m)

\( L_p \) length of pipe (m)

\( M \) Number of manholes (-)

\( M_t \) Volume of required mains water (top-up) (m³)

\( n \) Manning’s roughness coefficient (-)
$N$   Total number of links (-)

$N_{B,y}$   Net benefit resulting from implementing a given strategy, $y$ (%)

$P_a$   Acceptable performance level of service (units$^1$)

$P_f$   Maximum system failure level (units)

$P_o$   Original (design) performance level of service (units)

$PVC_{T,BAU}$   Discounted total cost of the business as usual strategy (£)

$PVC_{T,y}$   Discounted total cost of a given adaptation strategy, $y$ (£)

$Q$   Flow rate (m$^3$/s)

$r$   Discount rate (%)

$R$   Hydraulic radius of flow cross section (m)

$R_{d,T}$   24 hr rainfall depth for a given return period, $T$ (mm)

$Res_o$   Resilience index (-)

$rs_i$   Number of random failure sequences (-)

$rs_x$   Minimum number of random failure sequences (-)

$S_A$   Sub catchment area (ha or m$^2$)

$S_{CW}$   Sub catchment width (m)

$Sev_i$   Volumetric severity (-)

$Sev_p$   Peak severity (-)

$S_f$   Friction slope (-)

$S_T$   Number of storage tanks (-)

$t$   Rainfall duration (hrs or minutes)

$T$   Rainfall return period (yrs)

$t_c$   Time of concentration (minutes)

$^1$ Takes on the units of specific system performance indicators for example total flood volume (m$^3$) or mean nodal flood duration (hrs)
\( t_e \)  
Time of entry (minutes)

\( t_f \)  
Mean nodal flood duration (hrs)

\( t_{fl} \)  
Time of flow (minutes)

\( t_{mf} \)  
Maximum nodal flood duration (hrs)

\( t_n \)  
Elapsed (simulation) time (hrs)

\( t_s \)  
Time period of operation (service years) of a given system (yrs)

\( t_t \)  
Total rainfall event duration (hrs)

\( V \)  
Flow velocity (m/s)

\( V_D \)  
Water demand volume (m\(^3\))

\( V_{SC} \)  
Additional tank volume required for storm water control (m\(^3\))

\( V_T \)  
Unit RWH tank volume (m\(^3\))

\( V_{TF} \)  
Total flood volume (m\(^3\))

\( V_{TI} \)  
Total inflow volume (m\(^3\))

\( \chi_{sim} \)  
Total number of simulations (-)

\( Y_R \)  
Rainfall yield from given roof catchment (m\(^3\))

\( \mu \)  
Mean (units)

\( \sigma \)  
Standard deviation (units)

Other notations\(^2\)

\(^2\)Other less frequently used notations are described in full in the paragraph(s) where they are first mentioned
## Abbreviations

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<th>Description</th>
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<td>ADB</td>
<td>Asian Development Bank</td>
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<tr>
<td>AMS</td>
<td>Annual Maximum Series</td>
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<tr>
<td>ARF</td>
<td>Areal Reduction Factor</td>
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<tr>
<td>BAU</td>
<td>Business as Usual</td>
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<td>BSI</td>
<td>British Standards Institution</td>
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<td>CBA</td>
<td>Cost Benefit Analysis</td>
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<tr>
<td>GRA</td>
<td>Global Resilience Analysis</td>
</tr>
<tr>
<td>GSA</td>
<td>Global Sensitivity Analysis</td>
</tr>
<tr>
<td>IDF</td>
<td>Intensity-Duration-Frequency</td>
</tr>
<tr>
<td>IPCC</td>
<td>Intergovernmental Panel on Climate Change</td>
</tr>
<tr>
<td>KCC</td>
<td>Kampala City Council (now KCCA)</td>
</tr>
<tr>
<td>KCCA</td>
<td>Kampala Capital City Authority</td>
</tr>
<tr>
<td>LID</td>
<td>Low Impact Development</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>LSA</td>
<td>Local Sensitivity Analysis</td>
</tr>
<tr>
<td>MLC</td>
<td>Maximised Likelihood Classification</td>
</tr>
<tr>
<td>MoWT</td>
<td>Ministry of Works and Transport</td>
</tr>
<tr>
<td>MS</td>
<td>Middle State</td>
</tr>
<tr>
<td>NIAC</td>
<td>National Infrastructure Advisory Council (USA)</td>
</tr>
<tr>
<td>NRCS</td>
<td>Natural Resources Conservation Service (USA)</td>
</tr>
<tr>
<td>NWSC</td>
<td>National Water and Sewerage Corporation</td>
</tr>
<tr>
<td>O&amp;M</td>
<td>Operations &amp; Maintenance</td>
</tr>
<tr>
<td>OAT</td>
<td>One-Factor-at-a-Time</td>
</tr>
<tr>
<td>Ofwat</td>
<td>Water Services Regulatory Authority</td>
</tr>
<tr>
<td>PD</td>
<td>Percentage Deviation</td>
</tr>
<tr>
<td>PIMP</td>
<td>Percentage Imperviousness</td>
</tr>
<tr>
<td>RDM</td>
<td>Robust Decision Making</td>
</tr>
<tr>
<td>RFSM</td>
<td>Rapid Flood Spreading Models</td>
</tr>
<tr>
<td>RWH</td>
<td>Rainwater Harvesting</td>
</tr>
<tr>
<td>SWMM</td>
<td>Storm Water Management Model</td>
</tr>
<tr>
<td>TRRL</td>
<td>Transport and Road Research Laboratory</td>
</tr>
<tr>
<td>UBOS</td>
<td>Uganda Bureau of Standards</td>
</tr>
<tr>
<td>UDS</td>
<td>Urban Drainage System</td>
</tr>
<tr>
<td>UGX</td>
<td>Uganda Shilling</td>
</tr>
<tr>
<td>UNPD</td>
<td>United Nations Population Division</td>
</tr>
<tr>
<td>UWS</td>
<td>Urban Water System</td>
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<tr>
<td>WDS</td>
<td>Water Distribution System</td>
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</tbody>
</table>
Chapter One

1. Introduction

1.1 Background and research justification

In recent years, an increasing number of natural and manmade catastrophic events have led to extreme flooding in various cities worldwide, for example in New Orleans, USA (2005), Dhaka, Bangladesh (2010), Bangkok, Thailand (2010), New York, USA (2012) and in various English cities in 2000, 2008 and 2014 (Djordjević et al., 2011; Hammond et al., 2014; IPCC, 2014a; MetOffice, 2014; Park et al., 2013). In addition, rapid population and urbanisation growth rates continue to add significant stress to the already inadequate and ageing water infrastructure particularly in developing country cities (IPCC, 2014b; UNPD, 2012; Vermeiren et al., 2012).

Occurrence of such catastrophic events has increased the recognition of the challenge of urban flooding at global, national and local levels and has underscored the need to build (enhance) the resilience of existing urban drainage and flood management systems as a key strategy to minimising resulting flooding impacts (Djordjević et al., 2011; Hammond et al., 2014; Park et al., 2013). Urban flooding is not only caused by external climate-related and urbanisation threats such as extreme rainfall and increasing urbanisation but also internal system threats for example equipment malfunction, sewer collapse and blockages (Dawson et al., 2008; Kellagher et al., 2009; Mugume et al., 2015a, 2015b; Ryu and Butler, 2008; Ten Veldhuis, 2010). System (component) failures can either be abrupt (unexpected) shocks for example pump, valve or sensor failure or chronic pressures such as asset aging, long term asset decay or sewer sedimentation (Mugume et al., 2015a, 2015b). The impact of such failures, either singly or in
combination on urban drainage infrastructure can significantly reduce the expected flood protection service levels in cities and lead negative consequences such as loss of lives, damage to properties and other critical infrastructure (Djordjević et al., 2011; Hammond et al., 2014; IPCC, 2014a; Mugume et al., 2015b; Park et al., 2013; Ryu and Butler, 2008; Ten Veldhuis, 2010).

It is therefore vital to enhance the resilience of urban drainage systems (UDSs), not only to increase their ability to maintain acceptable flood protection service levels during normal operating conditions but also to minimise the resulting flooding impacts and consequences during exceptional (unexpected) loading conditions that lead to system failure (Butler et al., 2014; Djordjević et al., 2011; Mugume et al., 2015a, 2015b). The concept of resilience provides a paradigm shift from a focus on prevention of failure (i.e. fail-safe approach) to emphasis on minimising the resulting loss of functionality magnitude and duration during unexpected failures and ensuring quick return to original functionality levels after failure, i.e. the safe-fail approach (Butler et al., 2014; Lansey, 2012; Mugume et al., 2015a; Park et al., 2013).

Considering the case of the UK water sector, the need to build resilience in urban drainage infrastructure is well understood and supported by a suite of promising intervention strategies (Cabinet Office, 2011; CIRIA, 2014; Hepworth, 2015; Ofwat, 2012). However, operationalisation of resilience in urban flood management is still constrained by lack of suitable quantitative evaluation methods (Butler et al., 2014; Mugume et al., 2015b).

Conventional reliability-based urban drainage design and rehabilitation approaches tend to focus on minimising the probability of occurrence of functional (hydraulic) failures resulting from a specified design storm of a given frequency (return period). The design storm return period determines the flood protection service level delivered by the system (Butler and Davies, 2011). Hydraulic reliability-based approaches place significant emphasis on accurate quantification of the probability of occurrence of extreme rainfall and minimising the probability of resulting
hydraulic failures (Ryu and Butler, 2008; Sun et al., 2011; Thorndahl and Willems, 2008). However, such approaches may be insufficient for ensuring acceptable flood protection levels in cities during unprecedented extreme events and also do not consider other causes such as structural failures\(^3\) that also significantly contribute to urban flooding (Dawson et al., 2008; Kellagher et al., 2009; Mugume et al., 2015a, 2015b; Ofwat, 2009; Ten Veldhuis, 2010).

Furthermore, it is argued that the direct application of reliability-based approaches for evaluation of the effect of structural failures on UDS performance may be significantly constrained by insufficient knowledge about the causes and mechanisms of system failure, complexity of existing probabilistic asset deterioration modelling techniques and limited existing and rehabilitated system condition data sets (Ana and Bauwens, 2010; Egger et al., 2013; Kellagher et al., 2009; Park et al., 2013; Ten Veldhuis, 2010).

To address these limitations, new and computationally efficient evaluation methods which consider ‘all possible threats’ including existing network capacity constraints and unexpected system failures and take into account vital interactions between a wide range of threats (loading conditions), system performance (structure & function) and resulting failure impacts are required (Mugume et al., 2015a, 2015b)

1.2 Research aim and objectives

1.2.1 Research aim

To investigate, develop and apply the Global Resilience Analysis (GRA) approach to systematically evaluate the resilience of urban drainage systems to exceptional (unexpected) threats.

\(^3\) The term structural failure is used to refer to internal system or component failures such as sewer collapse, blockage, bed load sediment deposition or equipment malfunction. In some parts of the thesis, the three terms (structural, system or component) may be used interchangeably.
1.2.2 Specific objectives

• To investigate and characterise potential failure modes that lead to pluvial or urban drainage system flooding

• To evaluate the effect of a large number and range of both functional and structural failure scenarios on UDS performance

• To develop a new resilience index that quantifies system residual functionality as a function of failure magnitude and duration

• To model and evaluate the effect of implementing potential adaptation strategies on enhancement of resilience in UDSs

• To develop a methodology that embeds the cost of failure in cost-benefit analysis of resilience enhancement (adaptation) strategies

A number of key research questions have guided the investigation carried out to address the aforementioned research aim and objectives. These questions and the chapters where they are investigated include:

a) How can resilience be defined in clear, consistent and meaningful ways? *(Chapter 3)*

b) What is the scope of resilience assessment? *(Chapters 3 & 4)*

c) Which performance indicators or metrics are most suitable for quantifying global UDS resilience to flooding? *(Chapter 3)*

d) How can functional and structural failures in UDSs be effectively characterized and modelled? *(Chapters 3, 5, 6 & 7).*

e) What is the effect of improving redundancy and flexibility properties of a given UDS (achieved through implementing various adaptation strategies) on enhancement of its global resilience to unexpected system failures? *(Chapters 6, 7 & 8)*

f) When the cost of failure is included in resilience analysis, how cost-effective are the proposed adaptation strategies over the system’s service life? *(Chapters 3, 6, 7 & 8)*
Furthermore, this PhD research is linked to an EPSRC Established Career Fellowship Project ‘Safe & SuRe: A new paradigm for water management’ that is led by Professor David Butler. The Fellowship Project is aimed at developing new thinking and new approaches to water management in UK cities in response to emerging global challenges.

1.3 Thesis outline

Chapter 2 provides a critical review of emerging external climate-related and urbanisation threats and internal system failures and their contribution to urban flooding. The chapter reviews conventional reliability-based approaches for evaluation of both functional and structural failures in UDSs and identifies areas for further research. The chapter further discusses resilience concepts, contrasting paradigms of resilience and strategies for building (enhancing) resilience in UDSs. Key limitations that currently constrain the operationalisation of resilience in UDSs are deliberated and the need for new resilience-based evaluation approaches is underscored.

Chapter 3 describes a new analytical approach based on Global Resilience Analysis (GRA) that has been developed and applied to investigate functional and structural resilience in UDSs. The chapter also describes a new convergence analysis technique that has been developed and applied to increase computational efficiency in resilience-based evaluation. Key components of the GRA method that is system failure modelling, determination of failure envelopes and computation of the flood resilience index are described. The chapter concludes by describing a developed cost-benefit analysis method that embeds the cost of failure (penalty cost) in whole life costing of proposed adaptation strategies.

Chapter 4 describes flow modelling concepts and the adopted approach to modelling of surface flooding from UDSs. The chapter also provides a detailed description of two case study UDSs that form the basis of the investigations carried out in this research that is; a synthetic UDS and the Nakivubo UDS in Kampala,
Uganda. For the Kampala case study, the data collection, analysis, model build and sensitivity analysis carried out as part of this research are discussed.

Chapter 5 develops and applies the GRA method to systematically evaluate the performance of the Nakivubo UDS when subject to a wide range of random functional failure scenarios resulting from extreme rainfall loading inputs of varying magnitude, rate and spatial distribution. Functional failures are modelled using block rainfall events which represent ‘engineering worst case’ hydraulic loading scenarios for various rainfall return periods. The resulting loss of system functionality during the simulated failure scenarios is quantified using total flood volume and mean flood duration. System residual functionality and hence the level of resilience of the existing UDS is estimated by computing the functional resilience indices for the considered block rainfall loading scenarios.

Chapter 6 develops and tests the GRA method to evaluate the ability of a synthetic UDS to minimise the magnitude and duration of flooding when subject to a wide range of cumulative pipe failure scenarios. In addition, the developed approach is extended to evaluate global resilience enhancement benefits and cost-effectiveness of both centralised and distributed storage strategies over a system service life of 50 years.

Chapter 7 further develops and extends the GRA method to systematically evaluate the performance of the Nakivubo UDS in Kampala when subject to a wide range of random structural failure scenarios resulting from cumulative link failure. The chapter also investigates the performance and cost effectiveness of a set of potential adaptation strategies in enhancing system resilience to cumulative link failure namely: centralised storage (CS), distributed storage (DS) and improved operation & maintenance (O&M) strategies.

Chapter 8 focuses on investigating the performance and cost effectiveness of implementing multifunctional (dual-purpose) RWH systems at a catchment scale with respect to enhancement of global UDS resilience to flooding and provision of alternative water supplies in the Nakivubo catchment in Kampala.
Chapter 9 presents the main conclusions drawn from the research. It also synthesises the research contribution to the field and presents key recommendations for practice and further research.
Chapter Two

2. Literature review

This chapter sets the scene through a discussion of emerging threats and the challenge of urban flooding in various cities worldwide. In addition, the need to give due attention to the challenge of urban flooding is underscored in section 2.1. Section 2.2 discusses external climate change and urbanisation threats which contribute to functional (hydraulic) failures in UDSs. The section also provides a critical review of conventional ‘predict-then-adapt’ (top-down) evaluation approaches for evaluation of climate change impacts on UDS performance and highlights their limitations. In section 2.3, structural failures in UDSs are discussed and broadly characterised as sewer failures (collapse, blockage and sediment deposition) or equipment malfunction (pump, valve or sensor failure).

In section 2.4, a critical review of conventional reliability-based approaches for evaluation of both functional and structural failures in UDSs is carried out and key areas that necessitate further research are identified. Following from this, a detailed discussion of resilience concepts which has been mainly developed in studies on complex dynamic systems is carried out in section 2.5. Two broad and fundamentally different views of resilience that is; engineering and ecological are discussed and key differences are pinpointed. In addition, resilience is characterised either as general (attributed-based) or specified (performance-based) resilience and (potentially) desirable attributes of resilient infrastructure are discussed. In sections 2.6, strategies for building resilience in UDSs are presented. Practical aspects entailed in operationalisation of resilience in UDSs are discussed and key research questions are drawn in section 2.7. Finally, in section 2.8 a justification of the need for further investigation and development of new resilience-based UDS evaluation approaches in view of emerging threats is highlighted.
2.1 Introduction

In recent years, an increasing number of extreme flooding events have occurred in various cities worldwide (Djordjević et al., 2011; Hammond et al., 2014; IPCC, 2014a; MetOffice, 2014). Urban flooding is caused by a multiplicity of threats such as climate change and variability, rapid urbanisation particularly in cities in developing countries, insufficient urban drainage infrastructure and long term asset degradation (Djordjević et al., 2011; Hammond et al., 2014; IPCC, 2014b; Kellagher et al., 2009; UNPD, 2012). Furthermore, occurrence of unexpected system failures such as sewer collapse, blockage or equipment malfunction can further threaten the performance of existing UDSs and further exacerbate the challenge of urban flooding (Dawson et al., 2008; Kellagher et al., 2009; Mugume et al., 2015b; Ryu, 2008; Ten Veldhuis, 2010).

Cities can be viewed as complex systems characterised by dynamic relationships between a wide range of city functions such as human and economic activities, transport or innovation (Batty, 2008). When left unchecked, occurrence of extreme flooding events can lead to catastrophic impacts and consequences such as loss of lives, damage to property or severe disruption other critical city services (e.g. electric power, transportation or water distribution) due to the high concentration of people, infrastructure, assets and economic activities exposed to the resulting flooding impacts (Hammond et al., 2014; IPCC, 2014a; NIAC, 2009). Consequently, as cities develop and in view of these emerging threats, new approaches and innovative flood management solutions are required in order to maintain acceptable flood protection levels in current and future cities (Djordjević et al., 2011; Mugume et al., 2015a, 2015b; Park et al., 2013).

2.1.1 Types and causes of urban flooding

Urban flooding can be categorised into a number of distinct types depending on the threat (cause), which could be a single threat or a combination of threats (Lancaster et al., 2004; Ryu, 2008). In Table 2.1, the main types (categories) of urban flooding are listed and their respective causes described.
Table 2.1: Types and causes of urban flooding (Butler and Davies, 2011; Hankin et al., 2008; Lancaster et al., 2004; Maksimović et al., 2009; Mugume et al., 2015b; Ryu and Butler, 2008; Ryu, 2008; Ten Veldhuis, 2010)

<table>
<thead>
<tr>
<th>Type of flooding</th>
<th>Description of cause (threat)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pluvial flooding</td>
<td>Caused by both external threats such as high intensity, short duration extreme rainfall, high urban imperviousness levels coupled with insufficient urban drainage network capacity (including inlets) which leads to hydraulic overloading, overflow operation, surcharging and surface flooding.</td>
</tr>
<tr>
<td>UDS (sewer) flooding</td>
<td>Caused by internal system threats (other causes) such as equipment malfunction, sewer collapse and blockages that also lead to flooding. UDS flooding may occur in either dry or wet weather flow conditions.</td>
</tr>
<tr>
<td>Fluvial flooding</td>
<td>Exceedance of the flow capacity of the channel of a river, stream or other natural water course, typically associated with heavy rainfall events in which the excess water spills on the flood plain.</td>
</tr>
<tr>
<td>Coastal and tidal flooding</td>
<td>Caused by either or a combination of high tides, storm surges and wave action.</td>
</tr>
<tr>
<td>Estuarial flooding</td>
<td>Estuarial and water courses affected by tide locking. This often involves high tidal levels and high fluvial flows in combination.</td>
</tr>
<tr>
<td>Ground water flooding</td>
<td>Caused by raised ground water levels, typically following prolonged rain and may result into increased overland flow flooding.</td>
</tr>
<tr>
<td>Overland flow flooding</td>
<td>Caused by water flowing over the ground surface before reaching a natural or artificial drainage channel due to extreme rainfall that exceeds the infiltration capacity of the ground, or when the ground is highly saturated with water.</td>
</tr>
<tr>
<td>Infrastructure failure</td>
<td>Caused by structural, hydraulic or geotechnical failure of infrastructure that retains, transmits or controls the flow of water e.g. dam failure.</td>
</tr>
<tr>
<td>Leakages and external overflows</td>
<td>Drinking water flows on the surface due to a pipe failure, leaking hydrants or values, discharge of water from other sources e.g. construction sites or emptying of swimming pools.</td>
</tr>
</tbody>
</table>

2.1.2 Scale of pluvial and urban drainage system flooding

Over the last decade, a substantial number of studies has investigated fluvial and coastal flooding in cities (Dawson et al., 2008, 2005; Hall and Solomatine, 2008; Hall et al., 2006). However, limited attention has been given to pluvial or UDS flooding and this has been attributed to the relatively smaller scale of individual events and computational complexity inherent in quantitative evaluation of large urban drainage networks (Blanc et al., 2012; Dawson et al., 2008; Kellagher et al.,...
2009; Sun et al., 2011). In addition, some stakeholders have tended to view pluvial flooding as occasional urban drainage system ‘failure’ that should be ignored (Blanc et al., 2012; Butler and Davies, 2011).

However, in the recent years, the contribution and impacts of pluvial flooding events in cities are increasingly recognised (Blanc et al., 2012; Butler and Davies, 2011; Dawson et al., 2008; Pitt, 2008). In the UK context, the scale of pluvial flooding has been highlighted in the 2008 review of the 2007 extreme flooding (Pitt, 2008) which caused an estimated damage valued at £3.2 billion (Blanc et al., 2012; Chatterton et al., 2010). In a more recent study, it is reported that 3 million properties are at some risk of pluvial (or surface water) flooding in England alone with an estimated annual average economic consequence of £290 million (Environment Agency, 2014).

As a result, a renewed focus on enhancing the resilience of UDSs (existing or planned) to both pluvial and UDS flooding is now well established (Hepworth, 2015; Mugume et al., 2015b, 2014). Pluvial flooding is caused by external threats such as extreme rainfall, increasing dry weather flows or excessive infiltration that lead to UDS functional (hydraulic) failures for example overflow operation, surcharging and flooding (Mugume et al., 2015b; Thorndahl et al., 2008). In contrast, UDS flooding may be caused by structural failures (internal system threats) for example sewer collapse, blockages or equipment malfunction which lead to inability of the failed component to deliver its desired function in full or in part (Kellagher et al., 2009; Mugume et al., 2015b; Ten Veldhuis, 2010). Functional and structural failures in UDSs are described in more detail in sections 2.2 and 2.3.

2.2 Functional failures in UDSs

Functional failures in UDSs typically occur when exceptional rainfall with intensities greater than 20 – 25mm/hr occurs over very short durations (≤ 3 hrs) leading to exceedance of the flow conveyance capacity of the minor system or if the inlet capacity is insufficient to capture resulting surface runoff (Houston et al., 2011;
Maksimović et al., 2009; Ten Veldhuis, 2010). It can also occur following lower intensity rainfalls (~ 10 mm/hr) over longer periods, especially if the ground surface is highly impermeable (Houston et al., 2011). Occurrence of local extreme rainfall is majorly influenced by two key factors that is: anthropogenic climate change or natural climate variability (IPCC, 2013). Generation of surface run-off (overland flows) or dry weather flows (DWF) in combined systems is influenced by the level of urbanisation, which determines the percentage imperviousness (PIMP) and DWF rates respectively (Butler and Davies, 2011). In sections 2.2.1 and 2.2.2, the effects of climate change and urbanisation on flooding in cities are discussed in more detail.

### 2.2.1 Climate change

Climate change may lead to changes in extreme rainfall frequency and intensity and these changes exhibit substantial spatial variations (Butler and Davies, 2011; Djordjević et al., 2011; IPCC, 2013). Considering Northern Europe and other high latitude regions, climate model projections indicate a very likely increase annual average precipitation by 2100 (IPCC, 2013; Jenkins et al., 2009; Willems et al., 2012b). In the UK, climate projections indicate an increase in heavy winter rainfall frequency and intensity of up to 33% (Western UK) and up to 40% reduction in summer rainfall particularly in South East UK (Butler and Davies, 2011; Jenkins et al., 2009). In contrast, recent climate model projections suggest a very likely increase in not only the annual average rainfall but also the frequency and intensity of extreme rainfall events over mid-latitude and wet tropical regions by 2100 (Bates et al., 2008; IPCC, 2013).

The impacts of climate change that could threaten the performance of existing UDSs with respect to flooding include (i) increase in frequency and intensity of extreme convective rainfall events (ii) occurrence of relatively low intensity rainfall events over prolonged periods and (iii) prolonged periods of dry weather (or drought) which leads to increased bed load sediment deposition in combined sewer systems (ADB, 2011; Campos and Darch, 2014; Houston et al., 2011;
Sliuzas et al., 2013; Willems et al., 2012a, 2012b). Conventional methods for evaluation of climate change impacts that are of relevance to UDSs are discussed in 2.2.1.1 and their key limitations are discussed in 2.2.1.2.

**2.2.1.1 Climate change impact evaluation methods**

Effective evaluation of climate change impacts on UDS performance requires high resolution spatial-temporal rainfall data (Willems et al., 2012b). To achieve this, downscaling techniques have been developed and applied in a number of recent studies to evaluate the effects of climate change on extreme rainfall at an urban catchment scale (e.g. Onof and Arnbjerg-Nielsen, 2009; Willems et al., 2012a, 2012b). Downscaling fits within conventional ‘predict-then-adapt’ or ‘top-down’ Drivers-Pressures-State-Impact-Response (DPSIR) frameworks in which evaluation starts from the drivers (scenarios of GHG emissions) to threats (climate change) then to impacts (extreme rainfall) and finally to system response (UDS performance) (Dawson et al., 2010; Wilby and Dessai, 2010). Figure 2.1 provides an illustration of a ‘top-down’ climate impact assessment framework.
Chapter 2

Figure 2.1: 'Top-down' climate change impact assessment framework. Adapted from Kendon et al., (2012); Onof and Arnbjerg-Nielsen, (2009); Sunyer et al., (2012); and Wilby and Dessai, (2010)

Two main approaches that have been applied to increase the spatial-temporal resolution of coarse Global Climate Model (GCM) results include dynamic and statistical downscaling. In dynamic downscaling (or Regional Climate Modelling), a full physical simulation of the atmospheric system of a specific region of interest is carried out, within (i.e. nested) a GCM. In comparison to GCM results, dynamic downscaling enables local scale climate features such as orographic precipitation, extreme climate events and regional scale climate anomalies to be simulated at higher spatial (between 12 – 50 km) and temporal (daily time step) resolutions (Fowler et al., 2007; Sunyer et al., 2012; Willems et al., 2012a). However, dynamic downscaling is largely constrained by immense computational resources inherent in running full global atmospheric model simulations (Willems et al., 2012b).

Statistical downscaling methods on the other hand use empirical relationships to convert coarse climate model results to finer urban spatial-temporal scales (Willems et al., 2012b). The most commonly applied statistical downscaling techniques of relevance to UDS studies include climate change factors (e.g.
Mugume et al., 2014; Olsson et al., 2012; Semadeni-Davies et al., 2008a) and stochastic rainfall models (Butler et al., 2007; Chen and Djordjević, 2012; Onof et al., 2000). Mugume et al., (2013) provides a critical review of statistical downscaling techniques, their interrelationships and relevance for application to case studies in tropical developing country cities.

2.2.1.2 Key limitations of conventional climate change impact evaluation approaches

Most climate change impacts studies employing statistical downscaling have been carried out using case study cities in developed, temperate and mid-latitude regions (Fowler et al., 2007; Mugume et al., 2013; Willems et al., 2012b). However, only a few studies of a similar nature have been carried out using case study cities in tropical developing regions (e.g. ADB, 2011; Rana, 2013). The key challenges that have constrained their direct application in studies on urban flooding in tropical developing country cities include: (i) very high computational resources i.e. computer power, time and human resources, (ii) inherent uncertainties in global climate model projections, (iii) less reliability in tropical regions where local convective processes greatly influence local climate and (iv) limited or incomplete observed continuous rainfall time series data with comparable spatial-temporal resolution as climate model results (Egger and Maurer, 2015; Fowler et al., 2007; Park et al., 2013; Wilby and Dessai, 2010; Willems et al., 2012a, 2012b).

In recent studies, attempts have been made to address a number of shortcomings in conventional ‘predict-then-adapt’ approaches. The Robust Decision Making (RDM) methods also denoted as ‘assess-risk-of-policy’ approaches have been developed applied in a limited number of studies to address uncertainties resulting from non-stationary threats such as climate change (Hall and Solomatine, 2008; Hall et al., 2012; Park et al., 2013; Urich and Rauch, 2014a; Wilby and Dessai, 2010). RDM approaches apply exploratory modelling in which various policy interventions are tested using a large number of future scenarios in systems influenced by high uncertain drivers (Urich and Rauch, 2014a). However, Park et
al., (2013) argue that RDM approaches are still limited in instances where (i) threats are unknown and highly stochastic (e.g. occurrence of a tropical storm or hurricane), (ii) in complex systems where failures emanate from non-linear interactions between threats, system components and processes and (iii) where system response to external threats is largely non-stationary.

These limitations underscore the need to develop new and context appropriate tools or techniques that do not over rely on climate model projections in order to enable effective evaluation of UDS performance when subject to unexpected functional loading scenarios (e.g. Butler et al., 2014; Park et al., 2013; Wilby and Dessai, 2010).

### 2.2.2 Urbanisation

Recent urbanisation trends have vital implications for water infrastructure in cities. The global urban population has been projected to increase by 72% from 3.6 billion in 2011 to 6.3 billion by 2050 (UNPD, 2012). The projected urban population growth rates however, are not even. Recent studies suggest that while most European cities will experience significant demographic shifts towards an ageing population (Butler et al., 2014) and negative (shrinking) city growth rates (Urich and Rauch, 2014a, 2014b) on the one hand, a significant number of developing country cities in Africa, Asia and South America will continue to experience rapid urban growth rates on the other hand (UNPD, 2012). Taking an example of recent urban development trends in Africa, the number of cities with more than 1 million inhabitants increased from 2 in 1950 to 48 in 2012; and is projected to increase to 65 by 2025 (Vermeiren et al., 2012).

Urbanisation is influenced by a range of factors such as economic activity, population growth, demographic changes or adopted spatial planning policies and regulations (Sitzenfrei et al., 2013; Urich and Rauch, 2014a) and may result in varying urbanisation effects that include: densification, urban sprawl or depopulation (shrinking cities). Densification may result from continued
construction of new buildings and other infrastructure (e.g. roads, parking areas, transport hubs etc.) within a finite city district or catchment area that leads to an increase in building compactness and, impervious areas and a reduction of green spaces (Skovbro, 2001) which in turn may lead to increased stresses on existing water supply (i.e. increased water demand) and UDSs (i.e. increased system hydraulic overloading). Urban sprawl on the other hand occurs when city development occurs in a spread out and often irregular and unregulated manner leading a 'dispersed' urban form (Batty, 2008; Catal et al., 2008). Urban sprawl leads to loss of green spaces and the rate of urban development often outstrips the level of investment in expansion of existing urban water distribution, sewerage and storm water infrastructure.

Furthermore, in the context of developing country cities, rapid urbanisation, which is characterised by inadequately planned and and/or regulated urban development often leads to both inner city densification and urban sprawl and thus presents a combined threat to the already inadequate and ageing city water infrastructure (Douglas et al., 2008; IPCC, 2014b; Lwasa, 2010; UNPD, 2012; Vermeiren et al., 2012). In respect to urban drainage, urbanisation leads to changes in urban catchment characteristics such as population, housing density, land use and land cover types and imperviousness levels among others (Butler and Davies, 2011; Sitzenfrei et al., 2013; Urich and Rauch, 2014a). In addition, insufficient solid waste management practices may contribute to structural failures for example blockage of storm water inlets or pipes. It could also contribute to loss of hydraulic capacity in existing open channel systems due to uncontrolled disposal of solid waste and inadequate cleaning and maintenance operations (Mugume et al., 2015a; Ten Veldhuis, 2010).

Consequently, the impacts of urbanisation, if left unchecked, may significantly contribute to exacerbation of urban flooding impacts. Furthermore, during flooding events, the level of urbanisation may influence (safe) conveyance of exceedance flows during flooding events and consequently the resulting flooding consequences.
(Maksimović et al., 2009). New approaches that enable inclusion of structural failures in performance evaluation of UDSs are required.

2.3 Structural failures in UDSs

In contrast to functional failures which are caused by external threats, structural failures (also referred to as system or component failures) are caused by internal threats. Internal threats can be defined as endogenous drivers or processes which may occur with a given system across varying time scales (Dawson et al., 2010). In UDSs, structural failures refer may result from malfunction of single or multiple components leading to the inability of the failed component to deliver its prescribed function in full or in part (i.e. system failure) (Mugume et al., 2015b). Occurrence of system failures can also contribute significantly to negative flooding impacts and consequences in cities (Kellagher et al., 2009; Mugume et al., 2015b; Ten Veldhuis, 2010).

2.3.1 Categorisation of system failures

System failures can be broadly categorised as abrupt (unexpected) shocks for example structural damage of a pipe (due to heavy traffic), blockage of inlets or sewers, pump or sensor failure or chronic (long term) stresses such as sewer collapse, asset ageing/decay and bed load sediment deposition (Ana and Bauwens, 2010; Butler and Davies, 2011; Mugume et al., 2015b).

In the UK, although media attention tends to focus on flooding caused by functional failures, it is estimated that approximately 200,000 blockages occur in the UK each year and account for more than 55% of sewer flooding incidents (Arthur et al., 2009; Rodríguez et al., 2012), with more than 3,000 properties affected each year (Dawson et al., 2008). Furthermore, it is estimated that up to 40% UK sewers have structural defects (Ellis et al., 2004). It is also estimated that critical sewers i.e. those for which the cost of failure would be significantly higher than upgrading costs comprise of up to 20% of the total number of links in any given system (Butler and Davies, 2011)
Such failures have traditionally been minimised through routine or ‘business as usual’ (BAU) asset management strategies such as periodic operations and maintenance activities, system rehabilitation and regulation (operational targets). However, in view of emerging external threats which go beyond a given system’s internal processes or control, additional response interventions such as mitigation and adaptation may be required to maintain expected or acceptable customer service levels in cities (Butler et al., 2014; Dawson et al., 2010; Gersonius et al., 2013; Wilby and Dessai, 2010). The effectiveness of such strategies on enhancement of system resilience to flooding is a key area that requires further investigation (e.g. Butler et al., 2014). In Table 2.2, a description of possible UDS failure modes is provided and examples of serviceability indicators of relevance to investigation UDS performance during failure conditions are listed. In section 2.3.2, a review of causes and mechanisms of sewer failure is carried out.

Table 2.2: System failure modes and example (serviceability) indicators in UDSs (Butler and Davies, 2011; Campos and Darch, 2014; Ofwat, 2009; Savić et al., 2006; Ten Veldhuis, 2010).

<table>
<thead>
<tr>
<th>System failure mode</th>
<th>Description</th>
<th>Example indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sewer collapse</td>
<td>Gravity sewers collapse, structural deterioration or accidental failure of rising mains</td>
<td>No. of sewer collapses</td>
</tr>
<tr>
<td>Blockages</td>
<td>Inlet or sewer obstruction leading to odour, surcharging, flooding &amp; overflows</td>
<td>No. of blocked sewers</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. of blocked inlets</td>
</tr>
<tr>
<td>Bed load sediment deposition</td>
<td>Build-up of settleable particulate material e.g. due to bed deposits during dry weather flows, decelerating flows (storm recession) or disposal of solid waste in UDSs</td>
<td>% of sewer cross sectional area filled by sediments</td>
</tr>
<tr>
<td>Equipment malfunction</td>
<td>Failure of system components such as pumps, valves or sensors that curtails the ability of a given component to deliver its prescribed function in full or in part</td>
<td>No. of failed pumps</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. of failed valves</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. of failed sensors</td>
</tr>
</tbody>
</table>
2.3.2 Causes and mechanisms of sewer failure

In so far as existing city topography allows, most UDSs are designed as gravity systems that mainly consist of sewers (links) draining in the same direction. This approach is aimed at minimising operation and maintenance (O&M) costs associated with running of electro-mechanical equipment such as pumping stations over the lifetime of a given system (Butler and Davies, 2011). In this research therefore, emphasis is placed on sewer failures that are caused by link collapse, blockages and bed load sediment deposition (Mugume et al., 2015a, 2015b).

Davies et al., (2001) provide a comprehensive review of the causes of rigid sewer pipe failures. From the review, the main causes of sewer failures are categorised as: (a) construction related factors such as quality of workmanship, sewer size, depth, bedding or material, (b) local external factors such as surface loading, ground conditions, root intrusion or ground water levels and (c) other factors such as pipe age, sediment level, inappropriate maintenance, and waste water flow characteristics. Based on evidence from studies of construction related factors and sewer age, occurrence of sewer failures has been characterised using a ‘bath tab’ type failure curve (Davies et al., 2001), as illustrated in Figure 2.2.

Figure 2.2: ‘Bath tab’ curve sewer failure occurrence curve
In Figure 2.2, three distinct parts of the ‘bath tab’ curve fit well within the basic stages of sewer failure described in Ana and Bauwens, (2010) and Davies et al., (2001). Table 2.3 lists and describes these stages that is; initial defect, deterioration and collapse. From the review, two resulting effects of sewer failure that have direct implications on the ability of the UDS to continue functioning during failure that is reduction of flow cross sectional area and increase in roughness are identified and will form the basis for modelling of sewer failures (described in chapter 3 and applied in chapters 6 and 7).

**Table 2.3**: Stages of sewer failure (Ana and Bauwens, 2010; Davies et al., 2001; Tran, 2007)

<table>
<thead>
<tr>
<th>Stage of sewer failure</th>
<th>Description of cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial defect</td>
<td>Caused by cracking due to excessive vertical load or bad bedding, shoddy construction or when making new connections</td>
</tr>
<tr>
<td>Deterioration</td>
<td>Caused by degradation of the sewer material itself leading to: (a) reduction of load bearing capacity (structural deterioration) e.g. due to erosion of joint material or concrete corrosion and (b) reduction of flow conveyance (hydraulic) capacity due to sediment deposition, tree root intrusion which lead to reduction in flow cross sectional area or increase in roughness</td>
</tr>
<tr>
<td>Collapse</td>
<td>Triggered by some random event after prolonged sewer deterioration</td>
</tr>
</tbody>
</table>

### 2.4 Reliability-based evaluation of urban drainage systems

The conventional urban water system (UWS) design and rehabilitation approach is to build reliable systems that can achieve expected customer service levels under normal (standard or design) loading conditions (Butler et al., 2014; Jung et al., 2014). Reliability focuses on prevention of failures when subject to standard range of loading conditions (Butler et al., 2014). In water distribution systems (WDSs), reliability-based approaches such as least cost design formulations seek to ensure that systems are designed to satisfy minimum acceptable service levels within a specified or normal range of operating conditions for example demand variations (Farmani et al., 2005; Todini, 2000). In recent WDS studies, analysis has been extended to investigate system performance when subject to both hydraulic
failures (e.g. demand variations) and component (structural) failures resulting from a narrow range of stresses such as single pipe failure and changes in pipe roughness (Atkinson et al., 2014; Jung et al., 2014; Lansey, 2012; Mugume et al., 2015a; Trifunovic, 2012; Yazdani and Jeffrey, 2012).

In UDSs, the conventional design and rehabilitation approach focuses on prevention of functional failures resulting from a specified design storm of a given frequency (i.e. return period). The specified design storm return period determines the flood protection level provided by the system (Butler and Davies, 2011). However, the frequency of flooding is undoubtedly not equal to that of the specified design storm because the system is able to accommodate considerable surcharge before onset of surface flooding (Butler and Davies, 2011; Fu et al., 2011).

In more recent quantitative studies, simulation-based hydraulic reliability-based\(^4\) methods such as the first order reliability method (FORM) or risk-based optimal storm sewer network design approaches has been developed and applied for evaluation of UDS performance and resulting functional failure impacts and consequences. Thorndahl and Willems (2008) applied a first order reliability method to quantify UDS failure probabilities using hydrodynamic simulations applying long term (continuous) local rainfall time series. Ryu and Butler, (2008) developed a risk-based methodology that utilises continuous simulations to estimate the probability of flooding impacts. Based on the results, the annual average flood risk for a given property was quantified as a function of flood probability, flood depths and damages (costs). In Sun et al., (2011) a risk-based approach was combined with genetic algorithm (GA) based optimisation to determine the optimal UDS design under several design storms.

In other recent studies, the effect of climate change (i.e. extreme rainfall) on the performance of UDSs is investigated through: (i) direct consideration of more

\(^4\) Reliability-based approaches are also referred to as ‘risk-based’ methods in literature. Risk-based approaches generally focus on minimising the probability of occurrence of failure (i.e. flooding) and the consequences resulting from occurrence of failure (Blanc et al., 2012; Butler and Davies, 2011; Dawson et al., 2008; Hartford and Baecher, 2004; Ryu and Butler, 2008; Sun et al., 2011)
extreme rainfall events with higher return periods \( e.g. T = 50 \) or 100 years, (ii) use of climate change factors to uplift standard design storms (Gersonius et al., 2012; Mailhot and Duchesne, 2010), (iii) upscaling of observed extreme events using climate change factors (Mugume et al., 2015a, 2014) and (iv) use of stochastic rainfall models (Butler et al., 2007; Chen and Djordjević, 2012). Other studies have investigated the combined effects of climate change and urbanisation on UDS performance (Semadeni-Davies et al., 2008b).

Furthermore, in recent work, the Three Points Approach (3PA) has been proposed to facilitate decision making involving different actors and stakeholders (Fratini et al., 2012). 3PA entails three key domains that is: (1) **technical optimisation** which focuses on design rainfall standards and guidelines for UDSs and the technical solutions to deal with the design storms in order to prevent flood damages and to meet agreed flood protection service levels; (ii) **spatial planning**, which focuses on improving urban resilience to future conditions (e.g. extreme rainfall caused by climate change) through engagement with architects and urban planners to create new spaces for water conveyance and storage within the urban area during extreme conditions and (iii) **day-to-day values** which focuses on enhancing awareness, acceptance and participation among stakeholders for maintenance of above ground multifunctional green infrastructure (Fratini et al., 2012).

These studies, however, focus on quantifying system **hydraulic reliability** when subject to threats such as extreme rainfall or increasing dry weather flows that lead to functional failures only, with no due consideration given to **structural failures**. The main shortcoming of such an approach is that the full failure scenario space that includes **structural failures** is not explored (Kellagher et al., 2009; Mugume et al., 2015a, 2015b).

In recent studies, however, the contribution of system failures to resulting flooding impacts is now being recognised (Dawson et al., 2008; Kellagher et al., 2009; Ten Veldhuis and Clemens, 2011; Ten Veldhuis, 2010). There is a rich literature on sewer deterioration modelling, where probabilistic reliability-based techniques are
applied to quantify the likelihood of occurrence of sewer failures, including locations in a given UDS that are more prone to such failures (Ana and Bauwens, 2010; Butler and Davies, 2011; Duchesne et al., 2013; Egger et al., 2013; Savić et al., 2006; Tran, 2007). The aim of sewer deterioration modelling is to predict future system failure rates (deterioration) in a given UDS, based on its current condition (Ana and Bauwens, 2010; Savić et al., 2006; Trifunovic, 2012). The main approaches used for modelling of sewer failures include: physically based, statistical and artificial intelligence techniques (Ana and Bauwens, 2010; Egger et al., 2013; Savić et al., 2006).

Physically based models are more appropriate where the cost of failure is significant enough to justify the cost of detailed surveys for example during investigations involving critical links in a given network (Butler and Davies, 2011; Duchesne et al., 2013; Savić et al., 2006). Statistical models on the other employ probabilistic approaches to relate the physical sewer condition rating data (model inputs) to deterioration (model outputs), and are the most frequently used sewer deterioration modelling techniques (Ana and Bauwens, 2010; Duchesne et al., 2013; Tran, 2007). Statistical models are subdivided into pipe group (which consider whole urban drainage networks or cohorts) and pipe level models (which consider parts of the network with similar characteristics e.g. pipe age, material, or size). Due to limited historical or current UDS condition data, most statistical models are based on pipe groups rather than individual pipes (Butler and Davies, 2011). Artificial intelligence techniques on the other hand apply various techniques such as neural networks, fuzzy set theory or, genetic algorithms to enhance the efficiency of prediction of sewer failures based on available (current) asset data and more efficient (Ana and Bauwens, 2010; Savić et al., 2006).

However, the use of probabilistic reliability based approaches for evaluation of sewer failures is significantly constrained by a multiplicity of factors. These include (i) limited understanding of complex sewer deterioration processes (i.e. causes and mechanisms of failure), (ii) limited historical and current condition data sets of high
quality, (iii) limited or lack of records on pipe rehabilitation or replacement, (iv) uncertainties and errors in acquisition of pipe condition data e.g. CCTV inspection and (v) high costs involved in physical condition data collection (Ana and Bauwens, 2010; Duchesne et al., 2013; Egger et al., 2013; Fenner et al., 2007; Savić et al., 2006; Tran, 2007; Tscheikner-Gratl et al., 2015).

It is therefore argued that the direct application of reliability-based approaches for evaluation of structural failures in UDSs could be insufficient mainly because causes and mechanisms of failure are largely unknown and difficult to quantify (Kellagher et al., 2009; Mugume et al., 2015b). Furthermore, in recent studies, it is demonstrated that direct application of reliability-based methods may be further be constrained by computational complexities associated with the need to simulate a very large number of potential sewer failure combinations that could occur in a given UDS (Dawson et al., 2008; Kellagher et al., 2009). New and computationally efficient resilience-based evaluation approaches that shift the object of analysis from accurate prediction of the probability of occurrence of sewer failures, to evaluating the effect of different sewer failures modes and extent (range), irrespective of their occurrence probability, on the ability of an UDS to minimise the resulting flooding impacts are required (Kellagher et al., 2009; Mugume et al., 2015b). In the next sections, a critical review of resilience theory is provided and a clear justification for the need for new quantitative resilience-based evaluation approaches is provided.

2.5 Resilience theory

The term resilience originated from materials science over a century ago in studies on elastic deformation of solid materials (Hoffman, 1948). The concept of resilience was later extensively developed by Canadian ecologist, C.S. Holling in his studies on the behaviour of complex dynamic ecological systems (Holling, 1973). Currently, the term resilience is used in diverse research disciplines with multiple definitions and interpretations (Folke, 2006; Francis and Bekera, 2014; Holling, 1996; Park et al., 2013).
2.5.1 Ecological versus engineering resilience

A critical review of resilience studies suggests that resilience can be broadly interpreted from two fundamentally different viewpoints that is; ecological and engineering system resilience (Butler et al., 2014; Holling, 1996; Park et al., 2013). Ecological system resilience has been investigated in a large number of studies (e.g. Cumming et al., 2005; Folke, 2006; Holling, 1973; Walker et al., 2004). It is interpreted as a measure of system integrity and is defined as a system’s ability to maintain its basic structure and patterns of behaviour through absorbing shocks or disturbances under dynamic (non-equilibrium) conditions (Holling, 1996).

This view of resilience has been used to explain the apparent stability (persistence) of systems in conditions far from any equilibrium steady state and their ability to recover from shocks or disturbances that are either internal or external to the system (Carpenter et al., 2001; Cumming et al., 2005; Holling, 1996, 1973). This apparent stability is attributed to the existence of multiple stability domains in complex dynamic systems (Holling, 1996, 1973).

However, although a significant number of studies have investigated ecological system resilience within the wider field of complex dynamic systems, its application to infrastructure systems is a more recent development (Butler et al., 2014; Mugume et al., 2015b; Park et al., 2013). In contrast to natural ecological systems, engineering systems are a product of intentional human invention and are designed to provide continued (uninterrupted) services to society in an efficient manner (Blackmore and Plant, 2008; Holling, 1996; Park et al., 2013). Furthermore, engineering systems are considered to have a single stable equilibrium; with resilience being measured by the system’s resistance to a given disturbance (stress) and the speed of return to a single equilibrium (Holling, 1996).

Figure 2.2 graphically illustrates the fundamental differences between engineering and ecological resilience. In engineering systems, emphasis is placed on resistance to a given disturbance (protection) and the speed of return to a single
equilibrium. In ecological systems which exhibit multiple equilibria, external shocks or disturbances can cause the system to shift from one equilibrium or stability domain to another. In some instances, the magnitude of the shock or disturbance may exceed a certain critical system specific threshold or tipping point (Kwadijk et al., 2010), leading to shift to a potentially irreversible stability domain especially when the system is not actively managed or intentionally adapted (Holling, 1996; Nystrom et al., 2000).

Figure 2.3: ‘Ball and topography analogy’ depicting two different view point of resilience in complex dynamic systems. (a) single globally stable equilibrium, (b) speed of return to stable equilibrium, (c) relatively high resilience/stability in one stability domain or phase and (d) low resilience and increased susceptibility to shocks or disturbances that could lead to a phase shift, (e) phase shift occurs and (f) new and potentially irreversible equilibrium established (Holling, 1996; Nystrom et al., 2000)
2.5.2 Defining infrastructure system resilience

Engineering (infrastructure) systems are intentionally designed to provide continuous (uninterrupted) services to society in an efficient manner (Blackmore and Plant, 2008; Holling, 1996; Park et al., 2013). In this research therefore, engineering system resilience is interpreted differently from ecological resilience and focuses on ensuring *continuity and efficiency of system function* during and after occurrence failure (Butler et al., 2014; Lansey, 2012; Park et al., 2013).

To operationalise resilience in infrastructure systems, a number of prominent definitions that are in line with the interpretation of engineering resilience above have been put forward by both academic and professional communities (Table 2.4). The definitions emphasize a number of desirable attributes (properties) of resilient systems for example *absorption* of shocks, *continuity* of function and rapid *recovery* in the event of extreme shocks that lead to unexpected system failures.

<table>
<thead>
<tr>
<th>Definition</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>A system’s “ability to reduce the magnitude and/or duration of disruptive events” or “the ability to absorb, adapt to, and/or rapidly recover from a potentially disruptive event”</td>
<td>(NIAC, 2009)</td>
</tr>
<tr>
<td>The ability to withstand shocks and continue to function so as to maintain appropriate customer service levels during extreme/disruptive events</td>
<td>(Ofwat, 2010)</td>
</tr>
<tr>
<td>The “ability of assets, networks and systems to anticipate, absorb, adapt to and / or rapidly recover from a disruptive event”</td>
<td>(Cabinet Office, 2011)</td>
</tr>
<tr>
<td>The “ability to gracefully degrade and subsequently recover from a potentially catastrophic disturbance that is internal or external in origin”</td>
<td>(Lansey, 2012)</td>
</tr>
<tr>
<td>The “capacity to maintain essential services under a range of circumstances from normal to extreme”</td>
<td>(Hepworth, 2015)</td>
</tr>
</tbody>
</table>

2.5.3 Resilience characterisation

In complex dynamic systems, resilience is considered an *emergent* system property, rather than a static property that a system has (Park et al., 2013). This is
based on the accepted view, that it is generally impossible to abstract the global system behaviour from the analysis of single components during unexpected failure events (Hassler and Kohler, 2014; Park et al., 2013; Vespignani, 2010). This view is also held by Alderson et al., (2015) who argue that the contribution of single (individual) components to the *global functionality* of a given system is dependent on interactions with other components.

In figure 2.3, two dimensions or categories of resilience that have been put forward in recent studies that is: *general* and *specified* resilience are compared (Butler et al., 2014; Carpenter et al., 2012; Hassler and Kohler, 2014; Hwang et al., 2015).

![Figure 2.4: Dimensions of resilience](image)

*General resilience* refers to the state of the system that enables it to limit failure duration and magnitude to *any threat* (i.e. all hazards including unknowns) while *specified* resilience is performance-based and refers to the agreed performance of the system in limiting failure magnitude and duration to a *given (known) threat* (Butler et al., 2014; Scholz et al., 2011).

In line with the aforementioned definition, a given system’s specified resilience can be characterised based on its behaviour or response to given failure scenario (Haimes, 2009; Hassler and Kohler, 2014; Park et al., 2013). Based on this understanding, it is argued that a given system exhibits attributes or properties (general resilience) than can be altered/influenced in order to enhance its behaviour/response (specified resilience) to a given threat or failure scenario (Hassler and Kohler, 2014). In Table 2.5, examples of general resilience attributes put forward in recent studies on infrastructure resilience are listed.
Table 2.5: Attributes or properties of resilient systems

<table>
<thead>
<tr>
<th>General resilience attribute</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Robustness (reliability); redundancy, resourcefulness and rapidity</td>
<td>(Lansey, 2012)</td>
</tr>
<tr>
<td>Resistance, redundancy, reliability, response and recovery</td>
<td>(Cabinet Office, 2011)</td>
</tr>
<tr>
<td>Diversity, adaptability, cohesion, flexibility, renewability, regrowth, innovation</td>
<td>(Park et al., 2013)</td>
</tr>
<tr>
<td>Robustness (reliability); resourcefulness, rapid recovery</td>
<td>(NIAC, 2009)</td>
</tr>
<tr>
<td>Redundancy, connectedness, flexibility</td>
<td>(Butler et al., 2014)</td>
</tr>
</tbody>
</table>

It is further argued that system resilience can be improved by implementing various strategies that enhance a given (or set of) attribute(s) during design, retrofit or rehabilitation so as to influence the ability of the system to withstand the level of service failure and to ensure rapid recovery from failure once it occurs (Hassler and Kohler, 2014; Vugrin et al., 2011). In the next subsection, strategies and general resilience attributes of relevance to building resilience in UDSs are discussed in more detail.

2.6 Building (enhancing) resilience in UDSs

Potential strategies for enhancing resilience in UDSs are widely known and practiced. Taking the UK water sector as an example, a number of recent studies have proposed a range of potential strategies or intervention options (Cabinet Office, 2011; CIRIA, 2014; Djordjević et al., 2011; Mcbain et al., 2010). These strategies are broadly categorized as: mitigation, adaption and coping (Butler et al., 2014).

*Mitigation* refers to long term actions carried out a local scale to reduce a given threat. Although carried out at a local scale, mitigation activities have a wider implications at a city, national or global scales (Butler et al., 2014). In urban drainage and flood management, examples of mitigation strategies may include reduction of operational greenhouse gas emissions and reduction of urban imperviousness.
In contrast, adaptation entails targeted actions or adjustments carried out in a specific system in response to actual or anticipated threats in order to minimize failure consequences (IPCC, 2014b; Jones and Preston, 2011). Coping strategies on the other hand focus on reduction of recipient vulnerability and enhancement of social capacities through improved protection and preparedness of customers (of water services) from system failure impacts, particularly in instances where mitigation and adaptation strategies are insufficient (Butler et al., 2014).

In this work, the focus is placed on adaptation as an intervention strategy for enhancing UDS resilience to flooding. Adaptation is interpreted in this research as local responses to increasing threats such as modifying specific attributes of a system to enhance its capacity to minimize the magnitude and duration of failure to both standard (i.e. to increase system reliability) and exceptional loading conditions (i.e. to increase general or design resilience) (Mugume et al., 2015a). It is argued that by implementing adaptation strategies in a specific urban water system, both reliability and resilience could be enhanced (Mugume et al., 2015a). This could be achieved by altering the system configuration to enhance its inherent flexibility and redundancy attributes as described in sub sections 2.6.1 and 2.6.2 below.

### 2.6.1 Flexibility

Flexibility is defined as inbuilt system capability to adjust or reconfigure so as to maintain acceptable performance levels when subject to multiple (varying) loading conditions (Spiller et al., 2015; Vugrin et al., 2010). Flexibility can be increased in a given system through intentional one-off or phased interventions that enhance inbuilt system attributes such as flatness (less system hierarchy), buffering capacity (head room), homeostasis (feedbacks) and omnivory (diversification) (Butler et al., 2014; Hassler and Kohler, 2014; Watt and Craig, 1986; Wildavsky, 1988). It could also be increased by ensuring that more resources (e.g. trained repair crews, emergency supplies) are readily available at any given time to facilitate rapid response to an unexpected failure event (Butler et al., 2014; Hassler and Kohler, 2014; Lansey, 2012).
Based on this interpretation, flexibility could in principle be increased in a given system through use of spatially distributed (decentralized) systems (e.g. Sitzenfrei et al., 2013), modular systems (e.g. Spiller et al., 2015) or through provision of back-up capacity (e.g. Ahern, 2011; Cabinet Office, 2011). In UDSs, flexibility can be enhanced through designing in future proofing options (Gersonius et al., 2013), use of distributed systems for example distributed storage tanks, rainwater harvesting systems, roof disconnection or through use of intentionally designed multifunctional urban spaces such as car parks, playgrounds, cycle routes and road sections (Ahern, 2013; DeBusk, 2013; Mugume et al., 2015a; Taylor, 2013).

### 2.6.2 Redundancy

Redundancy on the other hand refers to the degree of overlapping function in a system that permits it to change by allowing vital functions to continue while formerly redundant elements take on new functions (Hassler and Kohler, 2014; Watt and Craig, 1986; Wildavsky, 1988). In UDSs, redundancy is enhanced by introducing multiple components providing similar functions for example storage tanks or parallel pipes, in order to minimize failure propagation through the system or to enable operations to be diverted to alternative parts of the system during exceptional loading conditions (Ahern, 2011; Cabinet Office, 2011; Mugume et al., 2015a; NIAC, 2009). Table 2.6 provides examples of potential adaptation strategies that could in principle (a priori) improve UDS flexibility and redundancy properties.

**Table 2.6:** Examples of potential adaptation strategies in UDSs (Mugume et al., 2015a)

<table>
<thead>
<tr>
<th>General resilience attribute</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redundancy</td>
<td>Centralized storage tanks</td>
</tr>
<tr>
<td></td>
<td>Pipe replacement</td>
</tr>
<tr>
<td></td>
<td>Parallel pipes</td>
</tr>
<tr>
<td>Flexibility</td>
<td>Distributed source control</td>
</tr>
<tr>
<td></td>
<td>Roof disconnection</td>
</tr>
<tr>
<td></td>
<td>Rain water harvesting</td>
</tr>
<tr>
<td></td>
<td>Multi-functional urban spaces</td>
</tr>
</tbody>
</table>
However, it is still unclear how each of these adaptation options actually enhances the resilience a given UDS in the event of unexpected system failures (Butler et al., 2014; Ofwat, 2012; Park et al., 2013). Further investigation and development of new resilience-based evaluation approaches that can enable quantification of the effect of improving flexibility and redundancy attributes (through a range of adaptation strategies) on enhancement of UDS resilience to flooding is required.

2.7 Operationalising resilience

The need and importance of building resilience in a given system or infrastructure in view of emerging threats is now widely recognised in both academic and practitioner communities (Butler et al., 2014; Djordjević et al., 2011; Hepworth, 2015; Mugume et al., 2015a). In the UK water industry, recent studies have proposed outline guidelines (Ofwat, 2012) and regulations (Hepworth, 2015) for operationalising resilience in UWSs. In addition, resilience standards have been proposed. They can be broadly categorised into three types that is; **event-based**, **system-based**, or **service-based** standards (Butler et al., 2014; Ofwat, 2012). Table 2.7 provides examples of resilience standards of relevance to urban drainage and flood management (Ofwat, 2012).

Table 2.7: Examples of resilience standards

<table>
<thead>
<tr>
<th>Category of standard</th>
<th>Example standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>Event based</td>
<td>1 in 200 year flooding return period for all critical assets</td>
</tr>
<tr>
<td>System based</td>
<td>Duty/standby configuration of pumps</td>
</tr>
<tr>
<td></td>
<td>Dual power supplies for critical assets</td>
</tr>
<tr>
<td>Service based</td>
<td>10 l/p/d of emergency water supplies provided within 24 hours of loss of supply</td>
</tr>
<tr>
<td></td>
<td>No property flooding</td>
</tr>
</tbody>
</table>

However, further research is required to develop detailed guidelines and suitable quantitative evaluation methods to facilitate the operationalization of resilience in practical urban drainage and flood management (Hepworth, 2015; Mugume et al., 2015b). To achieve this, new evaluation approaches that enable consideration of
‘all possible threats’ or ‘combinations of threats’ that could influence the resilience of existing systems are required (Hepworth, 2015; Ofwat, 2012). In addition, new evaluation approaches than can facilitate evaluation of effectiveness of proposed adaptation strategies and which include the cost of failure in resilience assessments are required (Alderson et al., 2015; Hepworth, 2015; Mugume et al., 2015a, 2015b). However, to achieve this, a number of research questions still remain unanswered and necessitate further investigation:

a) How can the concept of resilience be defined in a clear, consistent and meaningful way?

b) What is the scope (threats, scale, and failure modes) of resilience assessment?

c) Which performance indicators and/or metrics are most suitable for quantifying UDS resilience to flooding?

d) How can functional and structural failures in UDSs be effectively characterized and modelled?

e) What is the effect of improving redundancy and flexibility attributes of a given UDS (through implementing various adaptation strategies) on enhancement of its global resilience to unexpected system failures?

f) When the cost of failure is included in resilience analysis, how cost effective are the proposed strategies over the system’s service life?

This research will therefore seek to address these underpinning research questions in order to enable effective resilience characterization in existing UDSs and evaluation of performance (resilience enhancement) and cost-effectiveness of a set of adaptation strategies than can be implemented to build resilience in UDSs.

2.8 Conclusions

In this chapter, a critical review of resilience concepts of relevance to UDSs is provided. The chapter sets the scene by describing and characterising emerging threats as either external (climate-related and urbanisation) or internal system failures that lead to urban flooding. System failures are further classified as either
abrupt (short term) shocks or chronic (long term) stresses. Reliability-based evaluation approaches of relevance to UDSs are reviewed. The review suggests that previous UDS studies have tended to focus on investigation of hydraulic reliability. However, existing hydraulic reliability approaches are still insufficient for evaluating UDS resilience because only a narrow range of functional failures is considered and no due consideration is given to structural or system failures that also significantly contribute to flooding in cities. In addition, the review identified that although considerable literature on probabilistic sewer deterioration modelling approaches exist, most approaches are insufficient for direct application in resilience evaluation due to inadequate understanding of causes and mechanisms of system failures and attendant difficulties in quantitative assessment.

Based on the review, it is suggested that new resilience-based evaluation approaches which shift the object of analysis from prediction of threat occurrence probabilities (i.e. extreme rainfall and system failures) to a focus on evaluation of UDS performance when subject to a wide range of both functional and structural failures, irrespective of their occurrence probabilities are required.

Following from the above, a critical review of resilience concepts, interpretations and definitions is carried out. Two fundamentally different views of resilience are identified that is; ecological and engineering resilience. Emphasis is placed on engineering system resilience which focuses on ensuring continuity and efficiency of system function during and after occurrence failure. Two dimensions of resilience that is general or specified resilience are used to facilitate resilience characterisation. General resilience emphasises the need for consideration of ‘all possible threats’ while specified resilience focuses on enhancing inherent system attributes to improve its ability to limit the resulting failure magnitude and duration when subject to given threats. Finally, key research questions that necessitate further investigation are identified and will form the basis for the research carried out within the scope of this thesis. In table 2.8, the specific chapters where the identified research questions are tackled are highlighted.
Table 2.8: Identified knowledge gaps and/or research questions

<table>
<thead>
<tr>
<th>#</th>
<th>Research question</th>
<th>Addressed in:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>How can the concept of resilience be defined in a clear, consistent and meaningful way?</td>
<td>Chapter 3</td>
</tr>
<tr>
<td>2.</td>
<td>What is the scope (threats, scale, and failure modes) of resilience assessment?</td>
<td>Chapters 3 &amp; 4</td>
</tr>
<tr>
<td>3.</td>
<td>Which performance indicators and/or metrics are most suitable for quantifying UDS resilience to flooding?</td>
<td>Chapter 3</td>
</tr>
<tr>
<td>4a.</td>
<td>How can functional failures in UDSs be effectively characterized and modelled?</td>
<td>Chapters 3 &amp; 5</td>
</tr>
<tr>
<td>4b.</td>
<td>How can structural failures in UDSs be effectively characterized and modelled?</td>
<td>Chapters 3, 6 &amp; 7</td>
</tr>
<tr>
<td>5.</td>
<td>What is the effect of implementing various adaptation strategies or options on enhancement of resilience in a given UDS during unexpected system failures?</td>
<td>Chapters 6, 7 &amp; 8</td>
</tr>
<tr>
<td>6.</td>
<td>When the cost of failure is included in the analysis, how cost-effective are the proposed adaptation strategies over the system’s service life?</td>
<td>Chapters 3, 6, 7 &amp; 8</td>
</tr>
</tbody>
</table>
3. Quantitative resilience-based evaluation methods

This chapter describes the Global Resilience Analysis (GRA) approach which has been developed and extended to investigate the effect of a wide range of random functional and structural failure scenarios on the ability of an UDS to minimise the magnitude and duration of flooding. The developed GRA approach shifts the object of analysis from the threats themselves which has dominated most conventional probabilistic reliability-based approaches to explicit consideration of system performance when subject to large number of failure scenarios.

Section 3.1 sets the context and provides justification for the need for new resilience-based evaluation approaches. Section 3.2 describes the ‘Safe & SuRe’ approach that forms the basis for definition and interpretation of resilience in this research. Section 3.3 introduces the Middle-State based GRA method and describes its implementation in a MATLAB environment linked to the Storm Water Management Model, SWMMv5.1. The section also describes a convergence analysis method developed in this research for minimising computational complexity inherent in resilience-based evaluation of UDSs. Section 3.4 and 3.5 specifically describe the main steps in applying the GRA for evaluation of UDS performance when subject to a wide range of random functional (extreme rainfall) and structural (link) failure scenarios that lead to surface flooding.

In section 3.6, a procedure for determining failure envelopes that provide a means of graphically illustrating the range of resulting failure impacts at each considered failure level is presented. In section 3.7, the resilience index which is used to link the resulting loss of system functionality to the residual functionality and hence the level of resilience at given failure levels is developed. In addition, the section
describes the approach for determination of resilience envelopes based on the computed resilience indices. Lastly but not least, a methodology based on discounted cost benefit analysis (CBA) for evaluation of whole life costs is described in section 3.8. The developed CBA enables effective comparison of proposed adaptation strategies by including the cost of failure in resilience analysis.

3.1 Introduction

Building resilience in urban drainage systems (UDSs) requires consideration of a wide range of threats that contribute to urban flooding. Urban drainage system flooding is not only caused by external climate-related and urbanisation threats such as extreme rainfall and increasing urbanisation but also internal system threats for example equipment malfunction, sewer collapse and blockages (Kellagher et al., 2009; Mugume et al., 2015a, 2015b; Ryu and Butler, 2008; Ten Veldhuis, 2010).

Current hydraulic reliability-based design and rehabilitation approaches place significant emphasis on identifying and quantifying the probability of occurrence of extreme rainfall or increase in dry weather flows that lead to hydraulic overloading of a given system i.e. the fail-safe approach (Butler and Davies, 2011; Dawson et al., 2008; Ryu and Butler, 2008; Sun et al., 2011; Thorndahl and Willems, 2008). However, such approaches fail to consider other causes of failure that is; structural failures which also lead to flooding (Dawson et al., 2008; Kellagher et al., 2009; Möderl et al., 2014; Mugume et al., 2015a, 2015b; Sitzenfrei et al., 2011; Ten Veldhuis, 2010).

As result, hydraulic reliability-based approaches are insufficient for resilience analysis because the full failure scenario space that ranges from normal or identifiable failures to unexpected or exceptional conditions is not explored (Mugume et al., 2015b). Taking the example of Kampala city, Uganda, in addition to extreme rainfall, other causes of flooding include: inadequate investments in system cleaning and maintenance, disposal of solid waste in open channels and
long term bed load sediment deposition which leads to blockage and loss of hydraulic capacity in existing storm water systems (Douglas et al., 2008; KCC, 2002a; Lwasa, 2010; Mugume et al., 2015b; Sliuzas et al., 2013). Table 3.1 provides a formal categorisation of failure modes in UDSs adopted in this research.

**Table 3.1**: Failure modes in urban drainage systems (Mugume et al., 2015b)

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Description</th>
<th>Examples/Causes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Functional failure</td>
<td>Hydraulic overloading due to changes in inflows leading to failure e.g. overflow operation, surcharging and surface flooding</td>
<td>Increase in dry weather flows, extreme rainfall events, excessive infiltration</td>
</tr>
<tr>
<td>Structural failure</td>
<td>Malfunctioning of single or multiple components in the system such as pumps, tanks or pipes leading to the inability of the failed component to deliver its desired function in full or in part</td>
<td>Pipe collapse, blockages, sediment deposition, solid waste, pump failure, rising main failure</td>
</tr>
</tbody>
</table>

It is further argued that the direct application of hydraulic reliability-based approaches for evaluation of structural failures in UDSs could be insufficient mainly because *mechanisms* of failure are largely unknown and difficult to quantify (Ana and Bauwens, 2010; Kellagher et al., 2009; Park et al., 2013; Ten Veldhuis, 2010). More so, it is recognised that different threats or *combinations* of threats such as extreme rainfall or sewer collapse could lead to the same failed state (i.e. surface flooding). Therefore, by only considering a narrow range of hydraulic failures, current approaches take a limited view of functional resilience with no due consideration given to structural resilience (Mugume et al., 2015b).

In order to operationalise resilience in urban drainage and flood management systems, new approaches that seek to ensure that UDSs are designed to not only be reliable during *normal* (standard) loading conditions but also to be resilient to *unexpected* (exceptional) conditions are required i.e. the *safe-fail approach* (Mugume et al., 2015a, 2015b; Park et al., 2013). New and effective evaluation approaches are required to enable explicit consideration of *all possible failure scenarios* for example insufficient network capacity and system or asset failures in
order to holistically evaluate resilience in UDSs (Dawson et al., 2008; Kellagher et al., 2009; Mugume et al., 2015b; Ofwat, 2012; Ten Veldhuis, 2010).

3.2 The ‘Safe & SuRe’ approach

The ‘Safe & SuRe’ approach to Water Management has been developed in Professor David Butler’s EPSRC Established Career Research Fellowship Project entitled “Safe & SuRe: A new paradigm for water management”. The Fellowship Project is aimed at developing new thinking and new approaches to water management in UK cities in response to emerging global challenges. The developed approach forms the basis for the adopted definitions of reliability, resilience and sustainability, provides the underpinning conceptual framework for linking emerging threats through to their impacts and consequences, and underscores the role and place for mitigation, adaptation and coping interventions (Butler et al., 2014). The key definitions, relationships between reliable, resilient and sustainable systems are described in detail in section 3.2.1.

3.2.1 Definitions

**Reliability** is defined as: “the degree to which the system minimises the level of service failure frequency over its design life when subject to standard loading” (Butler et al., 2014). The goal of a reliable system (Rel) is to avoid or prevent failure:

\[
Rel = \min (\text{failure: probability}) \tag{3.1}
\]

**Resilience** is defined as “the degree to which the system minimises level of service failure magnitude and duration over its design life when subject to exceptional conditions” (Butler et al., 2014). The goal of a resilient system (Res) is therefore to both withstand service failure as much as possible and to recover from it if and when it occurs:

\[
Res = \min (\text{failure: magnitude, duration}) \tag{3.2}
\]
Exceptional conditions refer to uncertain threats or disturbances that lead to system failure for example climate change induced extreme rainfall events, sewer collapse or blockage. Based on this definition, the goal of resilience in engineering systems is therefore to maintain acceptable functionality levels (by withstanding level of service failure) and to rapidly recover from failure once it occurs (Butler et al., 2014; Lansey, 2012; Park et al., 2013).

Resilience is further classified into two broad categories: a) general (attribute-based) resilience which refers to the state of a system that enables it to limit resulting failure duration and magnitude when subject to any threat (i.e. all hazards including unknowns) and b) specified (performance-based) resilience which refers to the agreed performance of the system in limiting failure magnitude and duration to a given (known) threat (Butler et al., 2014; Scholz et al., 2011).

Sustainability on the other hand is defined as “the degree to which the system maintains levels of service in the long-term whilst maximising social, economic and environmental goals” (Butler et al., 2014). The goal of a sustainable system (Sus) is therefore to continue functioning over the long-term whilst balancing agreed societal goals:

\[
\text{Sus} = \max(\text{capital: social, economic, environmental})
\]  \hspace{1cm} (3.3)

### 3.2.2 Relationships between reliability, resilience and sustainability

Reliability is considered the foundational building block of ‘Safe & SuRe’ water systems. Reliability seeks to ensure that a given system functions well with respect to a given (expected or regulated) customer level of service (Butler et al., 2014). Intuitively, it is argued that reliability and resilience are related with the latter extending and building on the former. It is consequently postulated that if resilience builds on reliability, by improving the former, the latter can also be improved (Butler et al., 2014; Mugume et al., 2015a). Sustainability on the other hand focuses on the long term perspective and hence necessitates the design of resilient systems than can cope with threats or disturbances that may occur in the future (Scholz et
al., 2011). It is also argued that resilient systems contribute to achievement of sustainability through recovery, renewal (adaptation) and innovation (Park et al., 2013; Seager, 2008). Figure 3.1 illustrates the relationship between reliability, resilience and sustainability. A ‘Safe & SuRe’ system must be reliable, built upon by resilience and topped off with sustainability (Butler et al., 2014).

Figure 3.1: Relationships between Safe, Resilient and Sustainable systems (Butler et al., 2014)

### 3.3 Middle-State based Global Resilience Analysis (GRA) approach

In this research, a new Global Resilience Analysis (GRA) approach is developed, that shifts the object of analysis from the threats themselves to explicit consideration of system performance when subject to large number of failure scenarios (Johansson, 2010; Mugume et al., 2015a, 2015b). This research builds on and extends recent work in which an allied approach referred to as Global Vulnerability Analysis (GVA) has been applied to evaluate the effect of a wide range of progressive structural failure scenarios in various systems such as water distribution systems (WDSs) and electrical power systems (EPSs) on resulting failure impact magnitude (Johansson and Hassel, 2012; Johansson, 2010). Global Vulnerability Analysis applies graph theoretic approaches to quantify the resulting failure impact magnitude only (for example the number of customers without water...
or power supply) when the system is subjected to random and progressive removal of system components i.e. middle states (Johansson and Hassel, 2012; Johansson, 2010).

In contrast, the developed GRA method applies a physically based modelling approach to quantify both the failure impact magnitude and duration, when the system is subjected to random and progressive structural and functional failure scenarios (i.e. middle or failed states). The novelty of the GRA method is that potential interactions between system structure and function are explicitly considered during system failure modelling. Furthermore, the GRA method applies the resilience index to link the resulting loss of functionality to system residual functionality which provides a measure of the headroom in a given system (Mugume et al., 2015b). In subsection 3.3.1, the concept of middle states that forms the basis for the developed GRA method is described in more detail. Section 3.3.2 describes an approach adopted for characterisation of system resilience based on GRA results.

3.3.1 Concept of Middle (Failed) States

A Middle state is a point in the phase plane of a system (e.g. loss of a component) that can result from various initiating events (threats) (such as component malfunction, extreme weather or malicious attack) and lead to different end states as illustrated in Figure 3.2 (Johansson, 2010).
In conventional reliability-based approaches, a lot of emphasis is placed on quantifying the probability of occurrence of initiating events (and their attendant consequences), with limited emphasis or interest in middle states (Johansson, 2010). In this research, middle states (removal of components) are used to represent system performance during failure conditions (i.e. failed states). The novelty of the concept of the concept of a middle or failed state is that it enables the effect of failure of a given component on the global performance of a system to be evaluated without the need to quantify the probability of occurrence of the cause (threat) (Johansson, 2010; Kellagher et al., 2009; Mugume et al., 2015b). Based on this premise, resilience of a given system can be investigated by carrying out model simulations to quantify whole (global) system performance during failure conditions (Mugume et al., 2015a). Table 3.2 lists examples of middle (failed) states and impacts or level of service performance indicators in UDSs.

**Figure 3.2:** Illustration of the Middle State concept. $S_0$ is the system as planned/designed (Initial State), $IE$ the Initiating Event (threat), $MS$ the Middle (Failed) State and $ES$ the End State (Johansson, 2010).
### Table 3.2: Examples of middle states and impacts/levels of service indicators

<table>
<thead>
<tr>
<th>Middle States</th>
<th>Impacts /Levels of service indicators</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stress (functional):</strong></td>
<td></td>
</tr>
<tr>
<td>• x% rainfall increase</td>
<td>• Surface flood volume (m$^3$)</td>
</tr>
<tr>
<td>• y% DWF increase</td>
<td>• Surface flood duration (hrs)</td>
</tr>
<tr>
<td><strong>Stress (structural):</strong></td>
<td></td>
</tr>
<tr>
<td>• x% sewers (links) failed</td>
<td>• Flood extent (no. of flooded nodes)</td>
</tr>
<tr>
<td>• y% pumps failed</td>
<td>• Flood extent (no. of flooded properties)</td>
</tr>
</tbody>
</table>

### 3.3.2 Resilience characterisation

Based on the results of the GRA, systems are characterised as less resilient (i.e. more vulnerable) if the failure of a single or small fraction of components leads to disproportionately large impacts and consequences (Johansson, 2010). On the other hand, systems are characterised as more resilient if the failure of a large fraction of components does not significantly degrade their ability to maintain acceptable customer service levels (Butler et al., 2014; Mugume et al., 2015b). In Figure 3.3, the resulting loss of system functionality (strain) is plotted against percentage of failed components (stress). The figure contrasts the theoretical behaviour of: (a) less resilient and (b) more resilient systems.
Figure 3.3: Global resilience curves showing the behaviour of less resilient and more resilient systems when subject to progressive stresses. Butler et al., (2014) adapted from Johansson, (2010).

It is therefore argued that by carrying out model simulations involving a wide range of random failure scenarios, inbuilt system properties or attributes that influence its performance can revealed. Based on this, the resilience of a given system to a specific threat can be characterised.

3.3.3 Scope of resilience assessment

In this research, the proposed GRA method is further developed and extended to investigate the effect of a wide range of random functional and structural failure scenarios on the performance of an UDS. The developed methodology is also applied to test the effect of implementing various potential adaptation strategies that include; centralised storage, distributed storage, improved asset management and multi-functional rainwater harvesting systems on the global system’s ability to minimise resulting loss of functionality during considered the failure scenarios.
The key strengths of the developed GRA method is that emphasis is shifted from accurate quantification of the probability of occurrence of functional (extreme rainfall) failures (e.g. Sun et al., 2011; Thorndahl et al., 2008) and structural (sewer) failures (e.g. Egger et al., 2013), to evaluating the effect of different sewer failures modes and extent, irrespective of their occurrence probability, on the ability of an UDS to minimise the resulting flooding impacts (Kellagher et al., 2009; Mugume et al., 2015b). Figure 3.4 illustrates the proposed Middle-State based Global Resilience Analysis framework which is applied in this research.

Figure 3.4: Developed Middle-State Global Resilience Analysis framework for UDSs (Adapted from Butler et al., 2014; Johansson, 2010). The figure on the right that illustrates the desired resilient UDS is adopted from Mehaffy and Salingaros, (2013)

3.3.4 Minimising computational complexity inherent in GRA

Resilience-based evaluation of a given urban drainage network requires consideration of all possible failure scenarios in order to holistically quantify system performance during both normal conditions and unexpected conditions (Kellagher
et al., 2009; Mugume et al., 2015b). Taking the example of link (sewer) failure in a given UDS and considering two states that is; *non-failed* and *complete failure* for each link, $C_i: i = 1, 2, 3, N$, the total number of link failure scenarios in the entire solution space which should in principle be evaluated to quantify the maximum possible flooding impacts can be determined using Equation 3.4.

$$F(N, C_i) = \sum_{i=1}^{N} \frac{N!}{(N-i)!i!}$$

(3.4)

Where $F$ is the total number of failure scenarios; $N$ the total number of links and $i$ the link failure level (number of failed links).

Taking Nakivubo UDS in Kampala, which consists of 81 links as an example (described in detail in chapter 4), and assuming two link states (*non-failed* or *completely failed*), the total number of link failure scenarios within the full failure scenario space would be $2^{81} (2.4 \times 10^{24})$ failure scenarios.

Analysis of the distribution of the number of failure scenarios at each link failure level indicates a normal (Gaussian) distribution (Figure 3.5). The total number of scenarios involving random failure of a single link $(N - 1)$ is 81. The total number of scenarios involving random failure of two $(N - 2)$, three $(N - 3)$, four $(N - 4)$ links would be 3,240, 85,320 and 1,663,740 respectively. The highest number of potential failure scenarios occurs at the mid-point i.e. $N - 40$. Simulating such a large number $(2.4 \times 10^{24})$ of link failure scenarios in order to *fully explore* the entire failure scenario space would require huge computational resources in terms of computer power, cost and simulation time, which would limit the practicability of the GRA method (Kellagher et al., 2009; Mugume et al., 2015b).
In order to minimise the computational requirements associated with considering all possible link failure combinations, a minimum number of random failure sequences, $rs_x$ (and hence a minimum number of random failure scenarios) that should be analysed so as to achieve consistent GRA results, while covering as many failure states as possible needs to be determined (Mugume et al., 2015b).

In this research, preliminary investigation using Critical Component Analysis (CCA) is carried out to determine the number and location of links (i.e. critical link failure set) which when failed result in the most significant impacts for a given UDS (refer to 3.3.4.1). Results of CCA provide a good indication of the critical link failure set and hence the total number of simulations that would be required to quantify the most significant impacts. Thereafter, a method based on convergence analysis (Mugume et al., 2015b; Trelea, 2003) is developed and applied to determine the minimum number of pseudo-random cumulative link failure sequences, $rs_x$ that should be analysed to achieve consistent GRA results, while covering as many failure states as possible (Mugume et al., 2015b) (refer to 3.3.4.2).
3.3.4.1 Critical Component Analysis (CCA)

Previous studies that employ CCA in networked systems suggest that failure of only a small fraction of components results in the significant impacts on level service delivered by the system (Church and Scaparra, 2007; Johansson and Hassel, 2012; Johansson, 2010). CCA involves an exhaustive exploration of a system state to estimate negative consequences of failure of a single or set of components (Johansson and Hassel, 2012). In UDSs, critical sewers that is; sewers for which the cost of failure would be significantly higher than upgrading costs make up 20% of the system on average (Butler and Davies, 2011).

In this study, the single component CCA (*N-1 analysis*) is carried out using *targeted failure* (as opposed to *random failure*) of each individual link in the system. The results of CCA give a good estimate of the number and location of components (links) which when failed lead to the highest failure impacts. Emanating from this, it is suggested that for a given network, a certain minimum number of random failure sequences, \(rs_x\) is necessary to achieve convergence of the GRA results. This ensures that the set of links that are critical are covered in global resilience analysis (e.g. Johansson & Hassel, 2012).

3.3.4.2 Convergence analysis

*Convergence analysis* is carried out as described in the following steps to determine \(rs_x\) (Mugume et al., 2015b):

a) GRA is carried out using 5 random sequences (*i.e. 5(N+1) failure scenarios*) and the mean values of the total flood volume are determined.

b) The procedure is repeated for 10, 25, 50, 100, 150 and 200 random failure sequences that is: \(10(N+1); 25(N+1); 50(N+1); 100(N+1); 150(N+1)\) and \(200(N+1)\) failure scenarios respectively.

c) The percentage deviation, \(PD\) between the computed mean values is computed for each step-wise increase in \(rs_x\) for \(i: i = \{5,10\}; \{10, 25\}; \{25, 50\}, \{50; 100\}, \{100;150\} and \{150;200\}. If \(PD\) reduces to less than 5%, after the specified maximum number of random failure sequences (i.e. 200 in this
case), then the GRA results are considered to have converged. If this is not the case, then $rs_x$ is increased and the procedure is repeated.

### 3.3.5 GRA implementation

The GRA method is implemented in the MATLAB environment linked to the Storm Water Management Model, SWMMv.5.1; a physically based discrete time hydrological and hydraulic model that can be used for single event and continuous simulation of run-off quantity and quality and which is primarily built for urban areas (Rossman, 2010). Using a physically-based modelling approach as opposed to use of generic approaches such as surrogate models or graph theory (e.g. Johansson, 2010; Yazdani and Jeffrey, 2012; Yazdani et al., 2011) enables realistic modelling of vital interactions between system structure (components) and function (available hydraulic capacity) during failure, which enables a more accurate evaluation of its resilience to given threats (Alderson et al., 2015; Mugume et al., 2015b). Two distinct but complementary approaches are implemented to characterise UDS performance when subject to a wide range of functional and structural failures respectively (sections 3.4 and 3.5).

### 3.4 GRA considering functional failures

Global resilience analysis is applied to characterise the performance of an existing UDS when subject to a wide range of random functional failure scenarios resulting from extreme rainfall. Functional failure is modelled by random and cumulative loading of the sub catchments with the derived block rainfall events to represent system hydraulic overloading that leads to surface flooding. In contrast to application of uniform spatial rainfall loading over the whole catchment, the adopted approach of random and cumulative ‘failure’ of sub catchments models the effect of spatial rainfall distribution (variation) over the catchment, which leads to spatially non-uniform hydraulic loading in the UDS. The developed GRA method for evaluation of the effect on functional failure scenarios with varying magnitude, rate and spatial distribution presents a novel approach to describe a mechanistic
phenomenon (i.e. extreme rainfall with varying spatial distribution) that has previously been evaluated using other methods i.e. spatial rainfall interpolation methods, stochastic rainfall models and use of radar rainfall data. The developed GRA method is particularly suitable for use in urban catchments with minimal spatial rainfall data. In the developed GRA method, for each sub catchment, 2 system states are considered:

(a) **Non-failure**: The sub catchment is loaded with an insignificant (dummy) block rainfall event (constant $I_R = 6 \text{ mm/hr}$, $t = 100 \text{ minutes}$) that does not cause flooding at any of the nodes in the UDS.

(b) **Failure**: The sub catchment is loaded (‘failed’) with a specified block rainfall event that leads to hydraulic overloading of the links and flooding in parts of UDS.

Block rainfall events are derived from Intensity-Duration-Frequency (IDF) curves for the case study area and have a constant intensity over their duration, $t ; t \geq t_c$, where $t_c$ is the time of concentration for the catchment. As opposed to using design storm profiles whose intensity varies with time, block rainfall events with a duration greater than the time of concentration are chosen for GRA because each (block rainfall event) represents the maximum functional loading scenario for each considered rainfall return period (Butler and Davies, 2011). Consequently, it is argued that using block rainfall events for UDS model simulations enables evaluation of maximum loss of system functionality (i.e. hydraulic overloading) resulting from extreme rainfall of a given return period and duration.

A large number of model simulations is carried out and whole (global) UDS performance is quantified at each failure level, using total flood volume and mean nodal flood duration as system performance indicators. A time period of 7 hours which is significantly higher than the computed time of concentration (65 minutes) is used for the wet weather simulation to ensure that system performance is evaluated at steady state conditions so as to quantify maximum possible system failure impacts. The main steps taken in applying the GRA approach include:
a) A simulation is run to quantify the *initial state* performance of the UDS i.e. with all sub catchments in a *non-failed* state.

b) A randomly selected sub-catchment, $S_i : i = 1, 2, 3, ..., S_N$ is ‘failed’ and a simulation is run to quantify the UDS performance, where $S_N$ is the total number of sub catchments.

c) In the next iteration, two randomly selected sub catchments are ‘failed’ and a second simulation is run.

d) The procedure is repeated by running simulations at each failure level until all the sub-catchments, $S_N$ in the catchment area have been failed.

e) *Convergence analysis* is carried out by repeating the procedure in (a) – (d) is repeated for a range of random sub-catchment failure sequences $rs_i$ for $i = 1, 2, 3, ..., m$; where $m$ is the minimum number of $rs_i$ that should be evaluated to achieve consistent GRA results. In this research, (chapter 5) the study results suggest that at least 200 random failure sequences are sufficient to obtain consistent GRA results (Mugume and Butler, 2015).

f) The minimum, mean and maximum values of all model solutions (total flood volume and mean nodal flood duration) are computed at each considered sub catchment failure level and used to derive the resulting sub catchment *failure envelopes*. The envelopes represent the upper and lower limits of the resulting loss of functionality.

g) The procedure described in (a) – (d) and (f) is then carried out for other block rainfall events with $T = 5, 25, 50 & 100$ years.

A detailed description of the application of this approach for evaluation of functional resilience of an existing real-world UDS in Kampala, Uganda is described in Chapter 5.
3.5 GRA considering structural failures

Global resilience analysis is applied to characterise the performance of an existing UDS when subject to a wide range of random structural failure scenarios resulting from sewer (link) failure. Random and cumulative (progressive) removal of links represents the inability of the removed component to deliver its prescribed function (Mugume et al., 2015a, 2015b). In this research, links in an UDS are randomly and cumulatively failed and the resulting impacts on the global performance of the system are investigated for each failure level, until all the links in the system have been failed.

This process of pseudo random cumulative link failure represents structural failure modes such as sewer collapse, blockages and sediment deposition in closed systems and blockage resulting from deposition of solid waste and washed-in sediments in open channel systems (Mugume et al., 2015b). The approach of failing links randomly ensures that all links, \( N \) in the system have an equal probability of being removed (Johansson and Hassel, 2012). In addition, a step by step increase in sewer failure levels enables the exploration of the full sewer failure scenario space that ranges from predictable or commonly occurring failure scenarios such as single component (\( N - 1 \)), two component (\( N - 2 \)) or three component (\( N - 3 \)) failure modes to other unexpected (unpredictable) failure scenarios (\( N - i \)) involving simultaneous failure of a large number of components, \( i \) (Johansson, 2010; Mugume et al., 2015b; Park et al., 2013). In this research, link failure is modelled either by significantly reducing pipe diameters in the model (Möderl et al., 2014; Mugume et al., 2015a) or increasing the Manning’s roughness coefficient, \( n \) to a very high value (Mugume et al., 2015b).

A large number of model simulations are carried out at each randomly generated link failure level and system performance is quantified using the total flood volume and mean duration of nodal flooding as key performance indicators. Surface flooding is simply modelled using the ponding option inbuilt in SWMM which allows exceedance flows to be stored atop of the nodes and to subsequently re-enter the
UDS when the capacity allows (Rossman, 2010). The flooding extent at each node is modelled using an assumed ponded area of 7,500 m². A more detailed description of the adopted approach to modelling of surface flooding is provided in Chapter 4. Figure 3.6 further illustrates the adopted modelling framework.

The main steps in implementing the GRA include:

a) A simulation is run to assess UDS performance in its initial (non-failed) state using a given single extreme rainfall event as functional loading input.

b) A randomly selected single link $c_i: i = 1, 2, 3,...N$, in the UDS is failed and a simulation is run using the same extreme rainfall loading. This step represents single link failure mode and is denoted as $N - 1$.

c) Two randomly selected links, in the UDS are failed (denoted as $N-2$ failure mode) and the simulation is repeated.

d) The procedure is repeated for all $N - i: i = 1, 2, 3,...N$ failure modes until all the links in the system have been failed.
e) Convergence analysis is carried out by repeating the procedure in (a) – (d) to determine the minimum number of random failure sequences $\text{rs}_x$ that ensures convergence of results.

f) Using the determined $\text{rs}_x$, the procedure in (a) – (d) above is repeated to investigate the effect of the proposed adaptation strategies on minimising the loss of system functionality resulting from the considered cumulative link failure scenarios.

The developed method is tested in chapter 6 using a small synthetic UDS and then implemented in chapters, 7 and 8 for evaluation of structural resilience of the Nakivubo UDS in Kampala, Uganda and for testing effectiveness of modelled adaptation strategies.

### 3.6 Determination of failure envelopes

The use of average values in reliability and resilience analysis simplifies results’ interpretation but can potentially hide key information about the range of possible failure impacts and consequences (e.g. Trifunovic, 2012). The process of determining failure envelopes provides a means of graphically illustrating the range of failure impacts at each considered failure level (e.g. Church and Scaparra, 2007).

In this research, failure envelopes are determined by computing the minimum and maximum values of all model solutions (total flood volume and mean duration of nodal flooding) obtained at each considered failure level for the existing UDS and for the considered adaptation strategies. The resulting envelopes represent the upper and lower limits of the resulting loss of system functionality (impacts) that therefore provide vital information about the resilience properties of the system being tested. If the resulting envelope covers solutions with lower impacts at all failure levels, then the resulting loss of system functionality is minimised during the considered failure scenarios. If the resulting envelope covers solutions with higher impacts and with a larger range between the minimum and maximum values, the
tested system effectively exhibits higher loss of system functionality to the considered failure scenarios (e.g. O’Kelly and Kim, 2007).

### 3.7 Computation of the flood resilience index

The resilience index, $\text{Res}_o$, is used to link the resulting loss of functionality to the system’s residual functionality and hence the level of resilience at each failure level. The resulting loss of system functionality is estimated using the concept of volumetric severity, $\text{Sev}_i$ (Hwang et al., 2015; Lansey, 2012), which is interpreted as a function of maximum failure magnitude (peak severity) and failure duration (Figure 3.7). Volumetric severity provides a measure of the level of consequences such as injury, property of system damage that could result from the simulated failure impacts (Hwang et al., 2015).

![Figure 3.7: Theoretical system performance curve for an UDS. The block solid line, $P_o$, represents the original (design) performance level of service. The blue dotted line, $P_a$, represents a lower but acceptable level of service. $P_f$ represents the maximum system failure level resulting from the considered threat.](image)

Figure 3.7 illustrates the theoretical response of an UDS to a single extreme rainfall loading scenario. In Figure 3.7, severity can be estimated as the (shaded) area between the original system performance level, $P_o$ and the actual system...
performance curve, $P_i(t)$, at any time $t$ after occurrence of a given threat that lead to system failure (Equation 3.5).

$$Sev_i = f[Sev_p, t_f] = \frac{1}{t_o} \int_{t_o}^{t_n} (P_o - P_i(t)) dt$$  \hspace{1cm} (3.5)

Where $t_f$ is the failure duration, $t_o$ the time of occurrence of the threat, and $t_n$ the total elapse time. Equation 3.5 above is further simplified by assuming that the system failure and recovery curve is rectangular (Equation 3.6)

$$Sev_i = \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_o} = \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_n}$$  \hspace{1cm} (3.6)

The resilience index, $Res_o$, which is a measure of system residual functionality, is estimated as one minus the computed volumetric severity and is computed at each link failure level (Equation 3.7).

$$Res_o = 1 - Sev_i = 1 - \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_n}$$  \hspace{1cm} (3.7)

Where $V_{TF}$ is the total flood volume, $V_{TI}$ the total inflow into the system, $t_f$ the mean duration of nodal flooding and $t_o$ the total elapsed (simulation) time.

For a given threat (i.e. percentage of failed links or sub catchments), the proposed index quantifies the residual functionality of the UDS as function of both the failure magnitude (total flood volume) and duration (mean nodal flood duration). $Res_o$ values ranges from 0 to 1; with 0 indicating the lowest level of resilience and 1 the highest level resilience to the considered failure scenarios. Resilience envelopes are then derived by plotting the minimum and maximum values of $Res_o$ computed at each failure level against the percentage of failed links. The resulting envelopes graphically illustrate the system residual functionality at each considered failure level.

### 3.8 Cost-benefit analysis

Enhancement of resilience in UDSs by implementing various adaptation strategies cannot be achieved at any cost (Hepworth, 2015). It is therefore vital to investigate the trade-offs between operational performance benefits of implementing a given
Methodology

adaptation strategy and attendant whole life costs over the design life of the UDS. By taking into account the cost of failure (penalty cost) that could arise from the inability of the system to deliver its expected or prescribed customer level of service in resilience analysis, a more complete view of cost-effectiveness of proposed adaptation strategies is gained (Alderson et al., 2015; Hepworth, 2015).

In this research, discounted cost analysis (HM Treasury, 2011) is applied to evaluate net benefits achieved by implementing a set of proposed adaptation strategies considering a design (service) life of 50 years. The discounted total cost of each strategy is computed using Equations 3.8 and 3.9. The approach of discounting enables the comparison of costs and benefits that occur in different time periods (HM Treasury, 2011).

\[ \sum_{t=0}^{t=j} C_T / \left(1 + \frac{r}{100}\right)^{t_s} \]  \hspace{1cm} (3.8)

\[ C_T = C_P + C_M + C_L + C_{ST} + C_{OM} + C_{TF} \]  \hspace{1cm} (3.9)

Where \( C_T \) is the total cost (£); \( r \) the discount rate (\( r = 3.5\% \) for initial 30 yrs, then \( r = 3.0\% \)); \( t_s \) a given time period (service years) during an UDS’s design life, \( j \) (yrs); \( C_P \) the unit cost of pipes (£/m); \( C_M \) the cost of links (£); \( C_L \) the pipe laying cost (£); \( C_{ST} \) the cost of storage tanks (£), \( C_{OM} \) the system operation and maintenance (O&M) cost (£) and \( C_{TF} \) the cost of failure (direct tangible flooding costs in £).

### 3.8.1 Capital and O&M costs

The capital and O&M costs are computed using the cost equations in Table 3.3. Given the significant local city specific factors that influence land acquisition, land cost is excluded from the capital cost calculations (Mugume et al., 2015a; Swan and Stovin, 2007).
Table 3.3: Capital and O&M cost functions for an urban drainage network (Mugume et al., 2015a)

<table>
<thead>
<tr>
<th>Cost ('000 £)</th>
<th>Cost Equation</th>
<th>Remarks/References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe cost ( (C_p) )</td>
<td>( \sum_{i=1}^{N} 0.455D_p^{1.72}L_p )</td>
<td>Adapted from Barreto, (2012)</td>
</tr>
<tr>
<td>Pipe laying cost ( (C_L) )</td>
<td>( \sum_{i=1}^{N} 70L_pD_p d_p )</td>
<td>Unit cost of £70/m length/m diameter/m depth</td>
</tr>
<tr>
<td>Manhole cost ( (C_M) )</td>
<td>( \sum_{i=1}^{M} 300d_m )</td>
<td>Unit cost of £300/manhole</td>
</tr>
<tr>
<td>Storage tank cost ( (C_ST) )</td>
<td>( \sum_{i=1}^{S_T} 738.33V_{ST}^{0.88} )</td>
<td>Adapted from Barreto (2012)</td>
</tr>
<tr>
<td>O &amp; M cost ( (C_{OM}) )</td>
<td>( 0.1 \times (C_p + C_L + C_M + C_ST) ) 10% of total capital costs</td>
<td></td>
</tr>
</tbody>
</table>

Where \( D_p \) is the pipe diameter (in mm); \( L_p \) the pipe length (m); \( d_p \) the pipe depth (m); \( d_m \) the manhole depth (m); \( V_{ST} \) the storage volume (m\(^3\)); \( V_{TF} \) the total flood volume (m\(^3\)); \( N \) is the number of links (pipes), \( M \) the number of manholes; and \( S_T \) the number of storage tanks.

For open channel systems, in Kampala, a unit cost function (Equation 3.10) is derived from UDS capital costs presented in KCC, (2002a, 2002b). The cost function includes excavation and channel construction costs but exclude land costs (Equation 3.10). An annual inflation factor of 6.5% is applied to estimate the 2015 costs based on the 2002 cost data described in the reports. For conversion of costs from Uganda Shillings (UGX) to Great Britain Pounds (GBP), the 2015 average exchange rate (1GBP = 4611.5 UGX) is applied.

\[
C_c = 260.2kL_c \tag{3.10}
\]

Where \( C_c \) is the cost per m length of open channel section (link) and \( L_c \) the length of each link, and \( k \) is a factor dependent of the channel cross sectional area.

Furthermore, it is noted that O&M costs for open channel systems are significantly lower than comparable costs for piped systems (Parkinson and Mark, 2005). In this research, O&M costs for open channel systems are estimated as 1.0% of the capital costs (KCC, 2002b). Furthermore, for the improved Operations and Maintenance strategy (investigated in Chapter 7), the O&M costs are increased to 5% of the total capital costs.
3.8.2 Direct tangible flooding costs

The direct tangible flooding costs associated with a given strategy, $C_{TF}$ are computed as a function of total flood volume, flood damage cost per unit volume of flooding, and the probability of occurrence of flooding during the considered UDS design life using Equations 3.11 and 3.12 (Mugume et al., 2015a)

\[ C_{TF} = \sum_{t=0}^{t=j} V_{TF} \cdot f_c \cdot \bar{R}_t \]  \hspace{1cm} (3.11)

\[ \bar{R}_t = 1 - \left( 1 - \frac{1}{T} \right)^t \]  \hspace{1cm} (3.12)

Where $j$ is the design life of the UDS (yrs); $V_{TF}$ the total flood volume (m$^3$); $f_c$ the unit direct tangible flooding cost (£/m$^3$ of flooding); $\bar{R}_t$ the probability of flood occurrence during a service life of $t : t \leq j$ (yrs); and $T$ the rainfall return period (yrs).

The unit direct tangible flooding cost, $f_c$ is obtained from existing flood depth damage data presented in the UK Multi-coloured Manual (Penning-Rossell et al., 2005). In Figure 3.8, $f_c$ is computed by multiplying the flood damage per square meter, $D_{c,p}$ with the flood depth, $d_{f,p}$.

Model simulations are carried out in SWMM to estimate the mean nodal flood volume. Surface flooding of the minor system is modelled using the ponding option which allows exceedance flows to be temporality stored atop the nodes as virtual flood cones and to subsequently re-enter the system when capacity allows (Butler and Davies, 2011; Rossman, 2010). In this research, the flood extent at each node is assumed to cover an area of 7,500 m$^2$. The average flood depth is computed as a ratio of the total flood volume to the sum of flood areas at all nodes. From the model simulations, the average flood depth, $d_{f,p}$ is estimated. Using the Figure 3.8, and having estimated the average flood depths, the unit direct tangible flooding cost, $f_c$ is estimated.

In this research, the computed average flood depths, $d_{f,p}$ range from 0.6 – 0.7 m and the computed values of $f_c$ ranges from 450 – 580 £/m$^3$ of flooding.
Consequently, in this research, an average $f_c$ value of 500 £ /m$^3$ of flooding is applied.

![Typical depth-damage functions. Hammond et al., (2015) adapted from Penning-Rossell et al., (2005)](image)

**Figure 3.8:** Typical depth-damage functions. Hammond et al., (2015) adapted from Penning-Rossell et al., (2005)

### 3.8.3 Computation of net benefits

To convert all costs to 'present values', discount rates of 3.5% and 3.0% are applied to the first 30 years and the subsequent 20 years respectively as recommended in HM Treasury, (2011). The difference between computed total discounted costs for the BAU strategy ($PVC_{T,BAU}$) and each tested adaptation strategy, ($PVC_{T,y}$) represents the net benefit, $N_{B,y}$ attributed to the strategy. The net benefit, $N_{B,y}$ (%) is expressed as percentage of $PVC_{T,BAU}$ using Equation 3.13.

$$N_{B,y} (%) = \frac{PVC_{T,y} - PVC_{T,BAU}}{PVC_{T,BAU}} \times 100$$  \hspace{1cm} (3.13)
3.9 Conclusions

In this chapter, a new Middle State based Global Resilience Analysis (GRA) approach for evaluation of UDS performance when subject to a wide range of random functional and structural failure scenarios in urban drainage and flood management systems is described. The ‘Safe & SuRe’ approach which provides a basis for definition and characterisation of interrelationships between reliability, resilience and sustainability concepts is introduced. The GRA method presents a new and promising resilience-based UDS performance evaluation technique that shifts emphasis from prediction of the probability of occurrence of threats that lead to flooding to explicit evaluation of system performance when subject to a wide range of failure scenarios for example extreme rainfall, insufficient urban drainage network capacity and system failures which also contribute to flooding in cities. The developed methodology will be applied in Chapters 5, 6, 7 and 8 for evaluation of resilience in existing UDSs and for testing the effectiveness of a set of proposed adaptation strategies.
4. UDS modelling and description of case studies

This chapter describes the adopted urban drainage system (UDS) modelling approach that has been implemented in the Storm Water Management Model, SWMMv5.1. The chapter also describes the proposed case studies that are developed and applied in subsequent chapters for resilience based evaluation of UDSs under unexpected loading conditions. In section 4.1, the theoretical basis that underpins modelling of surface flooding from UDSs is described. Furthermore, two case study UDSs that will be applied in this research that is; a synthetic UDS and a real-world UDS in Kampala, Uganda are described.

Section 4.2 describes the synthetic UDS (Case study 1) which is applied for preliminary testing of the GRA method for investigation structural resilience in UDSs in Chapter 6. Section 4.3 describes the data collection, analysis and model build for the Nakivubo UDS in Kampala (Case study 2). In subsection 4.3.2, extreme rainfall frequency analysis is applied to generate Intensity-Duration Frequency (IDF) curves for Kampala that are used for derivation of block rainfall events. In subsection 4.3.3, a Local Sensitivity Analysis (LSA) method is applied to investigate the most influential model input parameters that significantly affect UDS performance (model outputs). The developed real world case study is applied for investigation of global UDS resilience to both functional and structural failure scenarios and for testing effectiveness of a set of proposed adaptation strategies in Chapters’ 5, 7 and 8.

4.1 Flow modelling in urban drainage systems

Mathematical modelling provides a valid and well established approach that facilitates description, simulation and analysis of complex water related interactions
between threats (causes), system performance and the resulting failure impacts. To achieve the objectives of a given urban drainage design, rehabilitation or performance analysis task, a robust urban drainage model, is required to represent complex physically based processes of transforming rainfall into storm water, collection and transport of flows (storm, foul or combined) to wastewater treatment plants and/or receiving water bodies. The main features which require representation include: estimation of run-off from rainfall, modelling of overland flow, part-full flow in the sewer system, sewer surcharging and surface flooding (Butler and Davies, 2011; Ryu, 2008).

In this research, the storm water management model (SWMM v5.1) is applied. SWMM is a physically based discrete time hydrological and hydrodynamic model that can be used for single event and continuous simulation of run-off quantity and quality and is primarily built for urban areas (Rossman, 2010). In the next subsections, the governing equations for simulation of flows in an urban drainage network in SWMM are described. In addition, a review of existing surface flood modelling approaches is carried out and a justification for the adopted approach for modelling of surface flooding is provided.

### 4.1.1 Governing equations

In this research, the full dynamic wave model (Saint-Venant equations) in SWMM is used to route flows through the network of links (pipes or open channels) and storage units. The dynamic wave model enables modelling of various flow regimes such as backwater, surcharging, reverse flow and surface ponding (Butler and Davies, 2011; Rossman, 2010). Considering flow along an individual link (conduit), the Saint-Venant equations for solving unsteady flow of water are given in Equations 4.1 (conservation of mass/continuity) and Equation 4.2 (conservation of momentum) (Rossman, 2006).

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0
\]  

(4.1)

\[
\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA \frac{\partial H}{\partial x} + gAS_f + gAh_L = 0
\]  

(4.2)
\[ S_f = \frac{n^2 V |V|}{k^2 R^{4/3}} \quad (4.3) \]

\[ h_L = \frac{k V^2}{2gL} \quad (4.4) \]

Where \( x \) is the distance along the link (m); \( t \) the time (s); \( A \) the cross-sectional area \((m^2)\); \( Q \) the flow rate \((m^3/s)\); \( H \) the hydraulic head of water in the conduit i.e. elevation head and any possible pressure head \((m)\); \( S_f \) the friction slope i.e. head loss per unit length \((-)\); \( h_L \) the local energy loss per unit length of conduit \((-)\); \( g \) the acceleration due to gravity \((m/s^2)\); \( n \) the Manning’s roughness coefficient \((-)\); \( V \) the flow velocity \((m/s)\); \( R \) the hydraulic radius of the flow’s cross section \((m)\); \( K \) a local loss coefficient at location \( x \) \((-)\) and \( L \) the conduit length \((m)\).

In addition, when analysing a network of links, an additional continuity relation (Equation 4.5) is required for the junction nodes that connect two or more links together (Rossman, 2006). Figure 4.1 illustrates the node-link representation in SWMM and the key terms in Equations 4.5.

\[ \frac{\partial H}{\partial t} = \frac{\Sigma Q}{A_{store} + \Sigma A_s} \quad (4.5) \]

Where \( A_{store} \) is the nodal surface area \((m^2)\); \( \Sigma A_s \) is the surface area contributed by the links connected to the node \((m^2)\) and \( \Sigma Q \) is the net flow into the node \((inflow - outflow)\) contributed by all links connected to the node including externally imposed flows \((m^3/s)\).
UDS modelling and case studies

Figure 4.1: Node link representation of a drainage system in SWMM (Roesner et al., 1992).

The Saint-Venant equations described above are valid only when the following assumptions are met: hydrostatic pressure distribution is hydrostatic, very small sewer bed slope, uniform velocity distribution at a channel cross section, prismatic channel, friction losses estimated by steady flow equations valid for unsteady flow and negligible lateral flow (Butler and Davies, 2011). These equations described above can be solved using the finite difference or finite volume numerical methods (Begnudelli and Sanders, 2006; Butler and Davies, 2011).

4.1.2 Modelling of surface flooding

The main approaches for modelling of surface flooding can be broadly categorised as: one dimensional or 1D (Mark et al., 2004); dual drainage or 1D-1D (Djordjević et al., 1999; Maksimović et al., 2009; Schmitt et al., 2005); 2D rapid flood spreading models (Blanc et al., 2012; Ryu, 2008) and coupled 1D-2D overland flow models (Digman et al., 2014; Leandro et al., 2009; Néelz and Pender, 2010).
4.1.2.1 1D modelling of surface flooding

In 1D modelling, only surface flooding from the minor system is modelled. Flooding will occur when the water surface at a node exceeds the maximum defined depth or if more flow volume enters a node than can be stored or released during a given time step (Rossman, 2010). Such an approach provides a robust and simplified surface flood modelling approach that provides a good indication of the total flood volume and the critical points in an urban drainage network (Butler and Davies, 2011; DHI, 2012). In this approach, exceedance flows are assumed to be temporality stored atop of the nodes (e.g. manholes) as ‘virtual flood cones’ or ‘reservoirs’ and to subsequently re-enter the UDS when the capacity allows (Butler and Davies, 2011; Maksimovic and Prodanovic, 2001; Maksimović et al., 2009; Rossman, 2010). Figure 4.2 illustrates the principle of the ‘virtual flood cone’ for modelling of surface flooding of the minor system.

![Figure 4.2](image)

*Figure 4.2 Modelling of surface flooding using a virtual flood cone (Maksimovic and Prodanovic, 2001)*

The simplified 1D approach however, could be less realistic for modelling of surface flooding during unexpected or extreme conditions because overland flood flows that occur in the major system are excluded from the analysis. Consequently, the 1D approach may lead to inaccuracies in the estimation of the global (whole)
4.1.2.2 Rapid flood spreading models

Rapid flood spreading models (RFSMs) build on and extend the 1D approach through a realistic distribution of the total flood volume generated by the minor system model over the catchment surface (Blanc et al., 2012; Butler and Davies, 2011; Ryu, 2008). RFSMs therefore enable delineation of the resulting flood catchment as illustrated in Figure 4.3. However, although such an approach is fast and relatively easy to use, it is limited by its inability to represent flood flow movement over time (Butler and Davies, 2011).

Figure 4.3: Flood catchment delineation (Ryu, 2008)

4.1.2.3 Dual drainage (1D-1D) modelling

In contrast to the 1D or RFSM approaches, the 1D-1D modelling approach aims at providing an accurate description of the transitions from free surface flows in the minor system to surcharging (pressurised flow) and finally to occurrence of
overland flows in the major system (network of ponds and pathways). The 1D-1D approach thus enables distinct consideration of vertical interactions between the major and minor system flows (Djordjević et al., 1999; Maksimović et al., 2009; Schmitt et al., 2005). The major and minor systems are linked via weirs or orifice type elements representing inlets and holes on manhole covers (e.g. Figure 4.4) through which a direct interaction between the two systems take place (Mark and Djordjević, 2006; Mark et al., 2004). Sewer surcharging is modelled using the Preissman open slot concept which allows the link (pipe) to exceed its original diameter by overflowing into the open slot and thereby representing the effect of pressurized flow (Butler and Davies, 2011; Hunt et al., 2012; Ryu, 2008).

![Figure 4.4: Illustration of the interaction of surface and sewer flows in dual drainage (1D-1D) modelling (Schmitt et al., 2005)](image)

### 4.1.2.4 Coupled 1D-2D modelling

Coupled 1D-2D models provide a more recent and advanced approach that can be used to achieve a much more realistic analysis of urban flooding. In 1D-2D models, the 1D minor system model is coupled with a 2D surface flow model (Chen et al., 2005; Mark and Djordjević, 2006). Interactions between the two models take place between an underground network of nodes and the surface computational grids (Maksimović et al., 2009; Mark and Djordjević, 2006). Coupled 1D-2D approaches
have been applied to simulate complex urban flood flows for example physically based representation of flood depths and extent (flood inundation maps) in urban areas and consideration of the storage effects of underground infrastructures (Chen and Djordjević, 2012; Chen et al., 2005). In comparison to the 1D-1D approach, the 1D-2D approach enables a more exact representation of buildings and urban structures and facilitates more realistic analysis of overland (major system) flows during extreme events where surface flows are not confined to streets or road profiles (Maksimović et al., 2009; Mark and Djordjević, 2006).

However, application of 1D-2D modelling approaches is significantly limited by the need for high level of spatial detail (i.e. high resolution Digital Elevation Models) and attendant computational complexity that is; high computational time, computer power and cost. Consequently, using 1D-2D approaches may be impractical for quantification of flooding impacts and consequences in: (a) large urban catchments, (b) studies involving continuous simulations using high spatial-temporal resolution rainfall data, and (c) resilience-based approaches that require a large number of model simulations to represent a wide range of failure scenarios (Chen and Djordjević, 2012; Dawson et al., 2008; Digman et al., 2014; Kellagher et al., 2009; Maksimović et al., 2009; Mugume et al., 2015b).

4.1.2.5 Adopted surface flood modelling approach

In this research, given the significant computational burden of the proposed global resilience analysis methodology, the simplified 1D approach to modelling of surface flooding (of the minor system) is applied rather than using more complex 2D overland flow models (Mugume et al., 2015b). Such an approach has also been applied successfully in other recent studies to minimise the computational complexity inherent in considering a large number of simulations or in evaluation of a very large number of virtual (synthetic) case studies with varying system characteristics (Egger and Maurer, 2015; Möderl et al., 2009; Sitzenfrei et al., 2013).
4.2 Case study 1: synthetic UDS

The modelled network represents an existing UDS that requires rehabilitation because its current configuration leads to unacceptable surface flooding (Mugume et al., 2015a). This system is a baseline configuration i.e. business as usual (BAU) case in which the pipes provide the hydraulic capacity of the system with no storage devices (Figure 4.5). The UDS is designed to convey flows generated by an observed extreme rainfall event with a total depth of 66.2 mm and duration of 100 minutes with no flooding at any node. The extreme rainfall event occurred on 25th June 2012 and was measured at Makerere University rain gauge station, Kampala (Figure 4.6). The system consists of 26 nodes and 25 links with diameters ranging from 600 mm to 1500 mm, and slopes ranging from 0.5 – 2.25%. The UDS drains a total catchment area of 22.5 hectares with an average slope of 0.5% and percentage imperviousness of 25%. Table A.1 presents the hydraulic data for the synthetic UDS.

![Figure 4.5: Layout of (a) modelled hypothetical UDS with no storage (BAU)](image-url)
In addition, two adaptation strategies are modelled and used to test their effect on enhancement of system resilience to structural failures (Refer to Chapter 6 for the detailed analysis and results discussion).

4.3 Case study 2: Nakivubo UDS in Kampala City, Uganda

Kampala is the political and economic capital of Uganda, with a total population of 1.72 million and an estimated annual population growth rate of 5.6% (UBOS, 2012). It is located in central Uganda on the shores of Lake Victoria (Figure 4.8). The city experiences a tropical climate and receives a relatively high annual average rainfall of 1,292 mm (Figure A.1). The rainfall regime is bimodal, with seasonal convective rainfall occurring mainly during the months of March - May (main rainy season) and October – December (secondary rainy season). Typical rainfall events are characterised by high intensities of short duration and high temporal and spatial variability, which is attributed to the effect of storm movement (KCC, 2002b; Kigobe et al., 2011; START, 2006)
However, over the last decade, the number of (reported) catastrophic flooding incidences that occur during extreme convective rainfall events has doubled; that is, from an average of 5 in 1993 to 10 in 2014, with the later having an average duration of 2 – 4 hours (Figure 4.8).

**Figure 4.8:** Number of reported flooding incidences in Kampala. Compiled from Lwasa, (2010) and various daily Newspaper articles that is; New Vision [http://www.newvision.co.ug/](http://www.newvision.co.ug/) and Monitor [http://www.monitor.co.ug/](http://www.monitor.co.ug/) newspapers.
In addition to occurrence frequency, the *magnitude* and *duration* of flooding events have increased and led to negative consequences such as property damage, traffic and business disruption, shallow ground water contamination and structural failure of the existing paved road network (Lule, 2014; Lwasa, 2010; Sliuzas et al., 2013; UN-Habitat, 2009) and in some unfortunate instances loss of life (Tumwine, 2014). The increased risk of urban flooding in Kampala is attributed to a multiplicity of factors. The effect of climate change and variability has led to an increase in the magnitude and intensity of extreme rainfall events that lead to flooding incidences in Kampala (Tenywa et al., 2008; UN-Habitat, 2009).

Secondly, Kampala has experienced rapid urbanisation trends that have led to very high increase urban imperviousness levels. In a recent study, it is estimated that between 1989 and 2010, the built-up area in Kampala quadrupled (444% increase) and will continue to present a significant challenge to flood management if left unchecked (Vermeiren et al., 2012). In addition, other key factors that significantly contribute to urban flooding include insufficient design of open channels and culverts, frequent disposal of solid waste in open channels leading to blockage and inadequate investments in system cleaning, maintenance and expansion (Douglas et al., 2008; KCC, 2002b; Lwasa, 2010; Sliuzas et al., 2013).

Figure 4.9 shows pictures of open channel sections blocked by solid waste (Figure 4.9a) and the recently constructed Lubigi flood alleviation channel in which sediments have covered a significant portion of the design hydraulic capacity (Figure 4.9 b & c). The pictures also illustrate that due to the nature of convective rainfall in Kampala, the required capacities (i.e. pipe diameters, open channel cross sectional areas, storage tank volumes) of storm water infrastructure are significantly large\(^5\) when compared to comparable infrastructure in temperate regions such as the UK.

---

\(^5\)Open channel cross sectional areas for typical storm water infrastructure in Kampala are considerably large due to the nature of convective tropical storms. As an example, the dimensions of a typical road side channel with a depth = 0.5 m, bottom width = 0.5 m, side slope (vertical: horizontal) = 1.5, draining a small catchment area has an equivalent pipe diameter of 728 mm.
Figure 4.9: (a) A drainage channel in Kampala blocked by solid waste (Davies, 2013) (b) Recently constructed Lubiigi flood alleviation channel filled with sediments and solid waste (Photo taken during field visit to Kampala in April 2014) (c) Same channel section as in (b) conveying storm water after a morning (normal) rainfall event on 15th April 2014.
Figure 4.10 shows photographs of recent flood incidences that have been reported in the media highlighting the scale of urban flooding in Kampala. Therefore, new approaches and solutions to urban flood management in Kampala necessitate detailed investigation.

In this research, the Nakivubo UDS, which drains a highly urbanised central business district in Kampala is selected for further investigation. The system which was designed for a flooding return period of 10 yrs (KCC, 2002) requires rehabilitation due to the increasing frequency, magnitude and duration of flooding mainly during extreme convective rainfall events (Mugume et al., 2015b). Figure 4.11 shows a satellite image of the case study area and the extent of the Nakivubo catchment.
4.3.1 Model build in SWMMv5.1

A model of the existing UDS that drains the Nakivubo catchment has been built in SWMM v5.1. The data needed to build the model has been obtained from a Digital Elevation Model (DEM) for Kampala (2 m horizontal resolution), a 2011 satellite image for Kampala (0.5m horizontal resolution), as-built drawings and from existing reports (KCC, 2002a, 2002b). The main steps undertaken during the model build are graphically illustrated in Figure 4.12.
4.3.1.1 Computation of sub-catchment width

GIS based spatial analysis of the Digital Elevation Model (DEM) is applied to delineate the entire catchment into sub catchments which represent areas whose run-off flows towards a single outlet. In the research, the ArcHydro tool box in ArcGIS v10.1 is used for terrain pre-processing and analysis. Using this approach, the Nakivubo catchment which drains a total area of 2,793 hectares is delineated into 31 sub-catchments (Figure 4.13). Based on the results of the sub-catchment delineation, parameters i.e. sub catchment area, $S_A$ and at least three values of possible maximum overland flow path lengths, $S_{MOL}$ are determined for each sub-catchment. Sub- catchment width, $S_{CW}$, is computed as the ratio of average maximum overland flow path length, $S_{MOL, avg}$ and the sub-catchment area (Equation 4.6).

$$S_{CW} = \frac{S_A}{S_{MOL, avg}}$$
Figure 4.13: Digital elevation model and delineated sub catchments in the Nakivubo catchment

4.3.1.2 Sub catchment slope analysis

GIS based terrain pre-processing is carried out in Arc-Hydro tool box to generate a raster file with the slope data for the entire catchment area. The resulting raster file is used to create a sub-catchment slope grid ‘Nakivubo_Slopes’ using ‘Zonal Analysis’ in the ‘Spatial Analyst’ (‘extract by mask’) tool in ArcGIS v10.1 (i.e. catchment slope data is subdivided into the respective sub-catchments). Based on the results the minimum, average and maximum slope values of each of the delineated sub-catchments are determined. The computed sub catchment slopes range from 0.034 – 0.172 (Figure 4.14)
Computation of impervious areas

In this research, GIS spatial analysis of satellite imagery is applied to compute the percentage imperviousness (PIMP) for the Nakivubo catchment. In this approach, image classification (extraction of information classes from multi-band raster images) is carried out using the iterative supervised classification (ISC) method, which is a form of Maximum Likelihood Classification MLC approach (Han and Burian, 2009). The ISC method automatically classifies a selected image layer representing a given land-use category using pre-created training samples. Table 4.1 lists the land use categories used for creating of the training samples.

<table>
<thead>
<tr>
<th>Land cover type</th>
<th>Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impervious</td>
<td>Tarmac road, gravel road, parking lot, roofs (grey tiles, red/maroon) tiles, blue iron sheets, galvanized iron sheets), bare soil</td>
</tr>
<tr>
<td>Pervious</td>
<td>Grassed areas, trees</td>
</tr>
</tbody>
</table>
The main steps involved in the ISC method are illustrated in Figure 4.15. The computed sub-catchment percentage imperviousness (PIMP) ranges from 52.3 – 85.7 (Table A.2).

**Figure 4.15:** Flow chart showing the main steps involved in GIS based land cover classification

### 4.3.1.4 Modelling of the hydraulic network

The existing primary and secondary conveyance system consists of trapezoidal open channel sections constructed using reinforced concrete in upstream sections and gabion walls in the downstream sections. Open channels are modelled as regular trapezoidal shapes with equal side slopes and horizontal bases. Nodes are placed at every change of flow direction, slope, bridge or culvert crossings. The
resulting hydraulic model of the system consists of 81 links, 81 nodes and 1 outfall, with a total conduit length of 22,782 m (Figure 4.16).

![Figure 4.16: Layout of the modelled Nakivubo urban drainage network](image)

The modelled system drains a total catchment area of 2,793 hectares and drains into the Nakivubo wetland and finally into Lake Victoria. The gradients of the open channel sections range from 0.001 to 0.0124. The existing system is not always clean in a ‘business as usual’ case. This is reflected in the SWMM model by taking the initial value of Manning’s n as 0.020 which is the upper limit of the recommended range (i.e. 0.010 – 0.020) for concrete lined channels.

### 4.3.2 Kampala rainfall data

In resilience analysis, emphasis is placed on investigation of system performance when subject to unexpected loading conditions that exceed normal or standard loading conditions (Butler et al., 2014; Mugume and Butler, 2015; Park et al., 2013). This requires evaluation of system performance during extreme events that lead to flooding (e.g. Kellagher et al., 2009). Therefore, as opposed to using continuous rainfall data or synthetic design storms, two types of rainfall data are chosen:
(a) Observed event-based extreme rainfall (Figure 4.6).

(b) Block rainfall events (Discussed in detail in Chapter 5)

Observed high temporal resolution single extreme events are chosen for resilience analysis because they represent actual conditions that led to flooding in Kampala (Sliuzas et al., 2013). Use of single events also eliminates the need for filtering of less significant events in continuous data (Chen and Djordjević, 2012) and hence improves computational efficiency. In addition, block rainfall events, which have constant rainfall intensity, $I_R$ over their duration $t$ such that $t \geq$ time of concentration, $t_c$, are derived from intensity-duration-frequency (IDF) curves and are applied in this research. The derived block rainfall events, with a duration greater than $t_c$, represent the maximum functional loading scenarios for the considered rainfall return periods (Butler and Davies, 2011). The methodology for derivation of block rainfall events is described in detail in Chapter 5.

4.3.2.1 Extreme rainfall frequency analysis

Intensity-duration-frequency (IDF) relationships provide a widely used form of conveying rainfall information for a given location (Butler and Davies, 2011). In this chapter, IDF curves for Kampala are derived from observed daily rainfall data using the Annual Maximum Series (AMS) extreme rainfall frequency analysis method (Butler and Davies, 2011; Madsen et al., 1997). In the AMS method, annual maximum daily rainfall depths are abstracted from the observed daily rainfall time series. The annual maximum values are ranked from 1 to $x$ in decreasing order of magnitude. Weibull’s plotting position formula (Equation 4.7) is applied to estimate the rainfall return periods, $T$ (Butler and Davies, 2011).

$$T = \frac{x+1}{m}$$

(4.7)

The total number of available daily rainfall observation years for the considered rain gauge stations is as follows: Makerere University (19), City Hall (30) and Kampala municipality (51). Because the observations have been recorded over a relatively short period for reliable estimation of extreme rainfall with higher return
periods, the AMS method is applied to determine the $T = 2$ yr rainfall depths where the prediction accuracy is high (Figure 4.17).

![Figure 4.17: Results of rainfall frequency analysis using the Annual Maximum Series Method for three rain gauge stations in Kampala (a) Gauge 1: Makerere University (1991-2009) (b) Gauge 2: City Hall (1963 – 1992) and (c) Gauge 3: Kampala Municipality (1942 – 1993)](image)

Thereafter, a generalised Gumbel relationship between rainfall of any frequency and the $T = 2$ yr values in East Africa (Fiddes et al., 1974) is used to determine the 24 hour point rainfall for higher return periods i.e. $T = 5, 10, 25, 50$ and 100 years (Equation 4.8).

$$R_{d,T} = a + c \log \log \frac{T}{T-1}$$  \hspace{1cm} (4.8)

Where $R_{d,T}$ is the predicted rainfall depth (mm), with $a$ and $c$ being location specific constants.

The main steps involved in applying this approach include are described in Fiddes et al., (1974). The results of this analysis are presented in Table 4.2. When compared to results of previous studies (i.e. Fiddes et al., 1974 and KCC, 2002), the rainfall depths for the respective return periods are under estimated on average.
by 3.8 – 11.7%. However, in resilience analysis, the main goal is to quantify system response to exceptional conditions. Consequently, the TRRL study results (Fiddes et al., 1974) which are higher are adopted and used for deriving the IDF curves.

Table 4.2: Results of rainfall frequency analysis for Kampala

<table>
<thead>
<tr>
<th>Return period, T</th>
<th>AMS frequency analysis results</th>
<th>Previous studies</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>64.4</td>
<td>63.1</td>
</tr>
<tr>
<td>5</td>
<td>86.3</td>
<td>84.6</td>
</tr>
<tr>
<td>10</td>
<td>99.8</td>
<td>97.8</td>
</tr>
<tr>
<td>25</td>
<td>112.7</td>
<td>110.4</td>
</tr>
<tr>
<td>50</td>
<td>135.2</td>
<td>132.5</td>
</tr>
<tr>
<td>100</td>
<td>148.1</td>
<td>145.1</td>
</tr>
</tbody>
</table>

4.3.2.2 Derivation of IDF curves and design storm profiles

For a given rainfall return period, the IDF curve provides the graphical illustration of the relationship between rainfall intensity, \( I_R \) (mm/hr) and duration, \( t \) (hrs). Having estimated the daily (24 hr) rainfall depths for the various return periods a second step in which the 24 hour rainfall is converted to given shorter durations is required (Butler and Davies, 2011; Fiddes et al., 1974; Shaw, 1994). Equation 4.9 relates the average rainfall intensity, \( I_R \) and duration, \( t \) for a given return period for Kampala (Fiddes et al., 1974; MoWT, 2010).

\[
I_R = \frac{a}{(t+b)^c} \tag{4.9}
\]

Where \( a, b \) and \( c \) are constants \( (b = 0.33, c = 0.95) \)

By eliminating \( a \), Equation 4.9 can be simplified into Equation 4.10

\[
R_t = \frac{t}{24} \left( \frac{24+b}{b+t} \right)^c \times R_{d,T} \tag{4.10}
\]

Where \( R_T \) is the rainfall depth for any duration, \( t \), \( R_{d,T} \) is the 24 hour rainfall.
The derived IDF curves (Figure 4.18) confirm a well-known phenomenon that short duration convective rainstorms are associated with very high rainfall intensities (Butler and Davies, 2011). In addition, the curves show that for all $T$, large variations in rainfall intensities occur at rainfall durations less than 4 hours.

![IDF curves for Kampala](image)

**Figure 4.18:** Derived intensity-duration-frequency curves for Kampala

In addition, design storm profiles (Figure 4.19) are derived from the IDF curves using a procedure described in Fiddes et al., (1974) and MoWT, (2010).
4.3.3 Sensitivity analysis of UDS model input parameters

Sensitivity analysis is generally used to characterize the effect of changes in model inputs (e.g. model parameters, initial conditions, boundary conditions) on the resulting model outputs (Tang et al., 2007). There are two prominent methods that have been applied to perform sensitivity analysis of model inputs that is; global and local sensitivity analysis (Saltelli et al., 2008). In Global sensitivity analysis (GSA), all model parameters are varied within predefined regions to quantify not only their importance but also the potential importance of parameter interactions (Tang et al., 2007).

GSA attempts to explore the full parameter space within the predefined ranges and is more appropriate for use in highly non-linear models with a large number of model parameters or in instances when interactions between various model parameters greatly influence the model outputs (Sweetapple et al., 2014; Tang et al., 2007). In contrast, Local Sensitivity Analysis (LSA) entails a univariate (i.e. one-
factor-at-a-time, OAT) analysis of model parameter impacts on model outputs (Tang et al., 2007). In designing OAT sensitivity analysis experiments, only one parameter changes values between successive simulations (Saltelli et al., 2008), and potential interaction effects are not considered (Tang et al., 2007).

4.3.3.1 OAT sensitivity analysis of UDS model parameters

In this chapter, the OAT method is selected and applied to quantify the effect of changing key UDS model parameters on the performance of the modelled UDS. OAT local sensitivity analysis is chosen for this type of analysis because it enables: (i) changes in model outputs to be unambiguously attributed to a specific model input parameter and (ii) rapid identification of the most influential model input parameters with minimum computational requirements (e.g. Sweetapple et al., 2014). The sensitivity of the UDS model parameters is tested using the single observed rainfall event (Figure 4.6) as functional loading input into the model. The lower and upper bounds of the model parameters that are selected and tested using the proposed approach are presented in Table 4.3.

<table>
<thead>
<tr>
<th>Model parameter</th>
<th>Base case</th>
<th>Lower bound (50% lower)</th>
<th>Upper bound (50% higher)</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Imperviousness (PIMP)</td>
<td>52.3 - 85.7</td>
<td>26.1 - 42.9</td>
<td>78.5 - 100</td>
</tr>
<tr>
<td>Average sub catchment slope</td>
<td>0.033 - 0.171</td>
<td>0.017 - 0.086</td>
<td>0.051 - 0.258</td>
</tr>
<tr>
<td>Manning's n for impervious areas (N-Imp)</td>
<td>0.01</td>
<td>0.005</td>
<td>0.015</td>
</tr>
<tr>
<td>Manning's n for pervious areas (N-Perv)</td>
<td>0.10</td>
<td>0.050</td>
<td>0.150</td>
</tr>
<tr>
<td>Manning's n for links</td>
<td>0.02</td>
<td>0.010</td>
<td>0.030</td>
</tr>
</tbody>
</table>

Two UDS model simulations are carried out for each model parameter while keeping all the other model parameters the same. The first simulation is carried out using the lower bound, which is calculated by reducing the base case UDS model parameter by 50%. Thereafter a second simulation is carried out using the upper bound which is calculated by increasing the base case UDS model parameter by 50%. For both the lower and upper bounds, the percentage change in model...
outputs (i.e. total flood volume, mean nodal flood duration) is calculated. The results are used to gain an initial understanding of the most influential model parameters that influence the performance of the Nakivubo UDS and hence the resulting flooding impacts.

### 4.3.3.2 OAT sensitivity analysis results

The results of the OAT sensitivity analysis of the modelled system are presented in Figure 4.20. The figure shows the percentage changes in model outputs for total flood volume and mean nodal flood duration for the existing UDS.

![Graph showing the sensitivity analysis results for total flood volume and mean nodal flood duration](image)

**Figure 4.20:** Results of OAT sensitivity analysis for the existing UDS

The results of OAT sensitivity analysis suggest that the total flood volume is predominantly influenced by Manning’s roughness coefficient, $n$ and percentage imperviousness ($PIMP$). Setting Manning’s $n$ to the lower bound results in the...
highest decrease in total flood volume of 51%, while setting Manning’s \( n \) to the upper bound increases total flood volume by 28%. On the other hand, setting \( \text{PIMP} \) to the lower bound decreases the total flood volume by 35%, while setting \( N\text{-Perv}, N\text{-Imp} \) and \( \text{slope} \) to both the lower and upper bounds has no significant effect on total flood volume.

When compared to total flood volume, the results of the OAT sensitivity analysis reveal that only Manning’s \( n \) has the most influence of the mean flood duration. Setting Manning’s \( n \) to the lower bound results in a 45% decrease in mean flood duration, while setting Manning’s \( n \) to the upper bound increases the mean flood duration by 52%.

The results of the OAT sensitivity analysis for both total flood volume and mean duration of nodal flooding reveal that Manning’s \( n \) for links and percentage imperviousness (\( \text{PIMP} \)) are the most sensitive model input parameters while Manning’s \( n \) for impervious areas (\( N\text{-Imp} \)), Manning’s \( n \) for pervious areas (\( N\text{-Perv} \)), \( N \) and subcatchment slope (\%) are the least sensitive.

It is however noted that the chosen Manning’s \( n \) for impervious areas (\( n = 0.010 \)) is lower than the non-failed state Manning’s \( n \) value for the links (\( n = 0.20 \)). A lower value of Manning’s \( n \) for impervious areas was chosen in this case because the links in the existing Nakivubo UDS are not always clean in a business as usual scenario (i.e. the Manning’s \( n \) value for the links takes into consideration the effect of bed load sediments and solid waste that leads to an increase in the friction losses in the links). In addition, it noted that the lower bound of the Manning’s \( n \) for impervious areas is rather low because it is smaller than 0.01 which corresponds to smooth turbulent flow.

### 4.3.3.3 Implications on choice of resilience enhancement strategies

The results suggest that for Nakivubo UDS, in addition of occurrence of extreme rainfall, the current condition (\( \text{initial state} \)) of the links in the system and the level of catchment imperviousness also have a significant influence on the resulting flooding impacts. The state of each link in the system (for example \( \text{clean} \) or
blocked) is influenced by adequacy of both asset management initiatives such as frequency of cleaning and maintenance operations and solid waste management in the city (e.g. Douglas et al., 2008; Lwasa, 2010).

On the other hand, catchment imperviousness is influenced by urban development factors such as construction of buildings and car parks, paving of green/open spaces, widening of existing road network among others. These results suggest that in addition to conventional strategies such as addition of storage tanks to enhance whole system flow attenuation properties, asset management strategies such as improved cleaning and maintenance operations should be considered in subsequent investigations on effectiveness of potential resilience enhancement strategies.

4.4 Conclusions

In this chapter, the adopted urban drainage system modelling approach is described. Two case study UDSs that will be applied for investigation of both functional and structural resilience are described. In Case study 1, a simplified synthetic UDS is proposed and will be applied in Chapter 6 for preliminary testing of the proposed GRA method for investigation of system resilience to structural failures. The chapter also describes the data collection, model build and preliminary performance evaluation of the Nakivubo UDS in Kampala, Uganda (Case study 2) using One Factor-at-a-Time (OAT) sensitivity analysis method. The results of the preliminary performance evaluation of the Nakivubo UDS, using the local sensitivity analysis (LSA) suggest that in addition to occurrence of extreme rainfall, the initial condition of the UDS (i.e. clean or blocked), and the level of catchment imperviousness also greatly influence the resulting magnitude and duration of flooding. The Kampala case study will be applied in chapters 5, 7 and 8 for investigation of global resilience to random functional and structural failures scenarios and for evaluation of the effectiveness of a range of proposed adaptation strategies.
It is noted that the accuracy of the model outputs could be further improved through model calibration. However, in the Kampala case study, due lack of suitable calibration data sets (i.e. lack of a dense network of rain gauges for recording continuous short duration rainfall and corresponding flow data at key sections in the UDS) and the cost of calibration data acquisition (automatic rain gauges, flow data loggers and field assistants), model calibration was not undertaken. Instead, Local Sensitivity Analysis, which also provides a valid approach to identify the most influential model input parameters that could have an influence on the simulated flooding impacts, was applied. Furthermore, to address the limitation of using a non-calibrated model, in Chapters 7 and 8 (evaluation of effectiveness of adaptation strategies on enhancement of global UDS resilience of the Nakivubo UDS), percentage changes in the simulated flooding impacts will be computed as opposed to use of absolute values.
5. Global resilience to functional failures

This chapter develops and applies the GRA method to systematically evaluate the performance of the Nakivubo UDS when subject to a wide range of random functional failure scenarios resulting from extreme rainfall. Section 5.1 provides an introduction and justification for the use of the GRA method for investigation of functional resilience in UDSs. In section 5.2.1 the time of concentration, \( t_c \) for the case study catchment is estimated and block rainfall events that are used as extreme loading inputs for the subsequent functional resilience evaluation are derived. The section also describes the main steps involved in applying the GRA method to quantify the resulting loss of functionality of the case study UDS when subject to extreme rainfall loading scenarios. The section further describes the computation of the functional resilience index, \( \text{Res}_f \) which is applied to link the resulting loss of functionality to the system’s residual functionality (and hence the level of resilience) for each considered block rainfall loading scenario. The subsequent sections discuss the results and present the main chapter conclusions.

5.1 Introduction

The main objective of this chapter is to develop and apply the GRA method to investigate the effect of a wide range of functional failure scenarios resulting from extreme rainfall on the ability of an UDS to minimise resulting loss of system functionality magnitude and duration (i.e. pluvial flooding). Pluvial flooding typically occurs when exceptional rainfall with intensities greater than 20 – 25mm/hr occurs over very short durations (≤ 3 hrs) and leads to functional failure of an UDS due to exceedance of the flow conveyance capacity of the minor system or if the inlet capacity is insufficient to capture the surface runoff (Houston et al., 2011; Maksimović et al., 2009; Ten Veldhuis, 2010). It can also occur following lower
intensity rainfalls (~ 10 mm/hr) over longer periods, especially if the ground surface is highly impermeable (Houston et al., 2011).

In order to reliably and realistically evaluate the effect of a wide range of functional failure scenarios on the resulting magnitude and duration of surface flooding, a computationally efficient method of modelling the effect of spatial rainfall distribution (variation), which leads to spatially non-uniform system hydraulic loading mostly during convective rainstorms is required (Butler and Davies, 2011; Chen and Djordjević, 2012; Kellagher et al., 2009). However, most urban drainage design/modelling studies apply point rainfall as uniform input over the catchment or use areal reduction factors (ARFs) to account for the differences between point and catchment averaged rainfall volumes. Such an approach may lead to inaccurate quantification of the resulting flooding impacts particularly in large urban catchments where the effect of spatial rainfall variation is considered to be significant (Achleitner et al., 2009; Butler and Davies, 2011; Einfalt et al., 2004; Kellagher et al., 2009; Vaes et al., 2005).

Prior to the development of the digital computer and attendant hydroinformatic tools, the most commonly applied approach for assessing the effect of spatial rainfall distribution in a given catchment is the use of multiple rain gauges. The most commonly used spatial rainfall interpolation methods include the Thiessen’s polygon’s method (which is simple and straightforward) and other interpolation methods such as isohyetal and kriging’s methods (Ball and Luk, 1998; Shaw, 1994; Svensson and Jones, 2010; Vaes et al., 2005). However, spatial rainfall interpolation methods are limited by lack of dense network of rain gauges in urban catchments particularly when the inter distance between the rain gauges is larger than 2 – 3 km. These limitations can lead to significant differences between point rainfall and the areal rainfall over the catchment (Vaes et al., 2005).

In a limited number of more recent urban drainage modelling studies, the effect of spatial rainfall distribution on the resulting flooding impacts has been investigated using two main approaches: (a) use of radar rainfall data and (b) stochastic rainfall
models (Achleitner et al., 2009; Blanc et al., 2012; Chen and Djordjević, 2012; Einfalt et al., 2004; Kellagher et al., 2009). On the one hand, widespread use of radar rainfall data in real-world applications is still constrained by insufficient (i.e. short) observed radar rainfall data sets, uncertainties or biases in radar estimates of extreme rainfall, heterogeneities in recorded radar data sets (due to continuous improvements in data processing algorithms) and other organisational constraints such as data safety and need for additional staff training (Einfalt et al., 2004; Svensson and Jones, 2010).

On the other hand, although arguably more promising when compared to radar data, the direct use of stochastic rainfall model data (continuous spatial rainfall data) in urban flood modelling studies, especially for large catchments, has also been constrained by significant computational burden (time/resources) required to run the simulations, need for additional pre-processing of the generated rainfall data to identify/filter significant events and unresolved inaccuracies in mathematical modelling of non-stationary local convective rainstorms patterns (Chen and Djordjević, 2012; Kellagher et al., 2009; Willems et al., 2012a). Consequently, new and computationally efficient approaches that enable the practical use of spatially varying rainfall in real-world UDS resilience evaluation are required.

In this chapter, the developed GRA method applies block rainfall events derived from observed extreme rainfall data (IDF curves) to evaluate the effect of spatial rainfall distribution on UDS performance during extreme rainfall loading conditions.

The developed GRA method presents a novel approach to describe a mechanistic phenomenon i.e. extreme rainfall with varying spatial distribution that has previously been evaluated using other methods that include: manual spatial rainfall interpolation methods, stochastic rainfall models and use of radar rainfall. The developed GRA method is particularly suitable for use in urban catchments with minimal spatial rainfall data. Using the developed methodology, the following key research questions are investigated:
a) What is the effect of a change in the functional loading magnitude on the ability of the UDS to minimise the resulting flooding impacts?

b) What is the effect of a change in the functional loading rate on the ability of the UDS to minimise the resulting flooding impacts?

c) How does spatial rainfall distribution (variation) affect the performance behaviour of an UDS during extreme rainfall events?

To address these research questions, the developed GRA method is applied to quantify the effect of a large number of pseudo random cumulative functional failure scenarios on UDS performance. Block rainfall events with varying magnitudes and intensity are used to represent the functional loading scenarios at various return periods. Individual sub-catchments are randomly and increasingly loaded (i.e. ‘failed’) with the selected block rainfall events until all the sub-catchments in the case study have ‘failed’. The process of random and cumulative extreme rainfall loading of the sub-catchments simply represents the stochastic and distributed nature of rain cell arrivals over the catchment and thus enables the effect of spatial rainfall variation over the catchment to be modelled. It also models the effect of storm movement across the catchment (e.g. due to changes in wind direction in a convective storm) on the performance of the UDS (Vaes et al., 2005).

A large number of pseudo random cumulative functional failure scenarios (51,200 sub-catchment failure scenarios derived from 1,600 random cumulative sub-catchment failure sequences, $rs_i$) is simulated and system performance (loss of functionality) at each sub-catchment ‘failure’ level is quantified using the total flood volume and mean nodal flood duration indicators. Based on the results of the analysis, sub-catchment failure envelopes which represent the resulting loss of system functionality (impacts) at each sub-catchment ‘failure’ level are determined by computing the upper and lower limits of the model solutions.

Finally, the resilience index, $Res_i$, which quantifies system residual functionality as a function of the failure magnitude and duration, is computed for each considered block rainfall loading scenario. By comparing computed values of $Res_i$, to the
design functional resilience index of the existing UDS, the effect of extreme rainfall on degradation of system residual functionality (headroom) and hence the level of resilience to extreme rainfall is quantified.

5.2 Methods

In contrast to application of uniform spatial rainfall loading over the whole catchment (i.e. using design rainstorms in Chapter 4, Figure 4.19 in which ARFs are applied), block rainfall events are applied randomly and progressively to the sub-catchments using the GRA method that is described in detail in chapter 3, section 3.4. The block rainfall events have a constant intensity over their duration, \( t \) that is greater than or equal to the time of concentration, \( t_c \) and are consequently chosen for subsequent resilience analysis. For a given duration and return period, each block rainfall event represents an engineering ‘worst case’ functional loading scenario (Butler and Davies, 2011). Consequently, it is argued that using block rainfall events for UDS model simulations enables assessment of maximum loss of system functionality (i.e. hydraulic overloading) for a given return period and duration. The main steps taken to derive the block rainfall events include: (a) computation of the time of concentration, \( t_c \), and (b) derivation of block rainfall events as a function of constant rainfall intensities (read off the IDF curves) and time \( t : t > t_c \).

5.2.1 Computation of time of concentration for Nakivubo UDS

For a given rainfall intensity, \( I_R \), the critical storm duration that causes the catchment to operate at steady state (equilibrium) and to generate maximum flows equals to the time of concentration, \( t_c \) (Butler and Davies, 2011). The time of concentration (Equation 5.1), is defined as the time required for surface run-off to flow from the remotest part of the catchment area to a point under consideration (Butler and Davies, 2011).

\[
t_c = t_e + t_{f_l}
\]  

(5.1)

Where \( t_e \) is the time of entry (overland flow time) and \( t_{f_l} \) the time of flow
In this work, the TR-55 method, which is recommended for urban catchments, is used (NRCS, 1986). This approach subdivides the time of entry, $t_e$, into two components i.e. sheet flow, $t_{sf}$, and shallow concentrated flow, $t_{sc}$. Having identified the longest channel flow path and hence the remotest sub catchment ($S_3$), the two components are computed and $t_e$ estimated. To estimate the time of flow, $t_f$, average velocities in the UDS links along the longest channel flow path are required. Model simulations are carried out in SWMM model using the 2 yr 24 hr design storm with a total depth of 70 mm (Figure 4.19a) to compute the link velocities, $v_i$ at each 5 minute simulation time step. The average link velocity, $v$, is calculated using the simulated $v_i$ for the middle 80% of the time steps (i.e. $v_i$ computed at the lower and upper 10% of the time steps respectively are excluded to avoid underestimation of $v$). This is illustrated in Figure 5.1 for selected links $C_3$, $C_{24}$, $C_{45}$, $C_{76}$ and $C_{81}$.

![Simulated link velocities](image)

**Figure 5.1:** Simulated link velocities in the Nakivubo UDS resulting from a 2 yr 24 hour design rainstorm
The results of the computed average link velocities are presented in Figure 5.2. The results indicate that flow velocities range from a minimum of 1.7 m/s to 3.7 m/s and show an increasing trend along the channel length i.e. upstream to downstream links in the UDS. Although relatively high, the computed average velocities are comparable to observed flow conditions during extreme rainstorms in Kampala (e.g. Sliuzas et al., 2013). Based on the average velocities, the individual times of flow, $t_{c,i}$ are computed in each link and the time of flow for the entire catchment computed by summing up the individual $t_{c,i}$ for all links along the longest channel flow path.

![Computed average link velocity](image)

**Figure 5.2:** Computed average link velocities along the longest flow path in the Nakivubo UDS

The *time of entry* and average *time of flow* are computed as 13.1 minutes and 52.1 minutes respectively. The *time of concentration* is computed as 65.2 minutes. The computed value of $t_c$ for the Nakivubo catchment is rather short considering a total contributing area of 2,793 ha. However, this is attributed to the steep sub catchment slopes, high imperviousness levels (52.3 – 85.7%) and urbanisation effects that have increased channelization of the previously natural drainage system leading to high channel flow velocities (Sliuzas et al., 2013). Based on
these results, a duration of 70 minutes is taken as the critical storm duration for subsequent functional resilience analysis.

5.2.2 Derivation of block rainfall events

Two sets of block rainfall events are chosen that is: $t = 2t_c$ (140 minutes) and $t = t_c$ (70 minutes). The two sets of block rainfall events are derived by reading off corresponding intensities, $I_R$ from the IDF curves at $t = 140$ minutes and $t = 70$ minutes respectively for $T = 5$, 25, 50 and 100 years (Figure 5.3). The block rainfall durations, $t$ are chosen such that $t > t_c$ to ensure that UDS performance is assessed at steady state conditions (Butler and Davies, 2011) and thus to enable simulation of maximum system failure impacts. The derived 70 minute block rainfall events have higher rainfall intensities (63%) but slightly lower total rainfall depths (19%) when compared to the 140 minute block rainfall events.
5.2.3 GRA and Convergence Analysis

To fully explore the sub-catchment failure scenario space, a large number of simulations is required. For example for a catchment with 31 sub catchments, and assuming the two system states above, the full failure scenario space would be $2^{31} = 2.15 \times 10^9$ failure combinations.

Given the significant computational burden involved in simulating such a large number of scenarios (i.e. considering all sub catchment failure scenarios), the
minimum number of sub catchment failure sequences, \( rs_x \), necessary to achieve consistent GRA results is determined using *convergence analysis* (Mugume et al., 2015). In addition, a simple 1D modelling of surface flooding (i.e. nodal flooding of the minor system) is applied, rather than using more complex 2D overland flow models. The study results suggest that at least 200 random sub catchment failure sequences, i.e. \( 200 \times 32 = 6,400 \) sub catchment failure scenarios should be simulated for each block rainfall event (Figure 5.4).

![Figure 5.4: Convergence of GRA results after 200 random cumulative sub catchment failure sequences](image)

Based on this, GRA is carried out following the steps described in chapter 3, section 3.4, by simulating 6,400 random sub catchment failure scenarios for considered each block rainfall event. In addition, the GRA results are compared with simulation results obtained using design storm profiles for various return periods in which an areal reduction factor (ARF = 0.90) computed for the Nakivubo catchment is applied (Figure 4.19)
5.2.4 Computation of functional resilience index

The functional resilience index, $Res_f$, is used to link the resulting loss of functionality (severity, $Sev_i$) to the system’s residual functionality when the system is subjected to the considered block rainfall loading scenarios. The details of the derivation of the resilience index are provided in chapter 3, section 3.7.

In this chapter, volumetric severity, $Sev_i$ is estimated as a function of maximum surface flooding magnitude and duration (Equation 5.2).

$$Sev_i = \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_{mf}}$$ \hspace{1cm} (5.2)

Where $V_{TF}$ is the total flood volume; $V_{TI}$ the total inflow into the system; $t_f$ the mean nodal flood duration and $t_{mf}$ the maximum nodal flood duration.

However, it is noted that using the simulated surface flood duration (obtained using the 1D surface flood model), does not consider the duration of flooding that occurs in the major system (i.e. overland flow paths such as roads, paths and grass ways) during extreme events which could lead to underestimation of the mean flood duration. In addition, it is noted that the simulated surface flood duration represents the ‘failure impact’ time and does not include other factors that affect recovery time such as ‘system repair’ time and the ‘failure consequence’ time such as the time required to repair a property affected by flooding (Mugume et al., 2015b).

The functional resilience index, $Res_f$, is estimated using Equation 5.3 and ranges from 0 to 1; with 0 indicating the lowest level of functional resilience and 1 the highest level functional resilience to each considered extreme rainfall loading scenarios (Mugume et al., 2015b). It is computed at 100% (full) sub catchment failure level and hence represents the most severe functional loading scenario for each considered block rainfall event.

$$Res_f = 1 - Sev_i = 1 - \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_{mf}}$$ \hspace{1cm} (5.3)
In addition, Equation 5.3 is used to compute the design functional resilience, $Res_{f,d}$ for the Nakivubo UDS (designed for a 10 yr flooding return period). For the computation, it is assumed that the 10yr design flooding return period corresponds to a 2 yr design rainstorm (Figure 4.19a). Consequently, $Res_{f,d}$ is computed by simulating the effect of the 2 yr design rainstorm on the resulting loss of system functionality.

5.3 Results

5.3.1 Effect of spatial rainfall distribution on flooding

5.3.1.1 Effect on total flood volume

Simulation results obtained using the 140 minute block rainfall events indicate the effect of increasing spatial rainfall distribution on the ability of the UDS to minimise the loss of system functionality is less pronounced at lower return periods (e.g. $T = 5$ yrs) but increases with increasing rainfall return periods. This can be observed in Figure 5.5 b, c and d where the simulated total flood volume at higher sub-catchment failure levels significantly with increasing rainfall return periods, implying that increased spatial loading of the sub catchments leads to disproportionately high loss of system functionality magnitude.

Secondly, the results show that applying uniform, areally reduced rainfall over the catchment (i.e. use of design storms with ARFs applied) over estimates the total flood volume at spatial rainfall loading levels less than 70% and that the overestimation increases with increasing $T$ (Figure 5.5). On the other hand, the results also indicate that use of ARFs could lead to underestimation of the total flood volume at higher spatial rainfall loading levels; for example in Table 5.1, the total flood volume at a spatial rainfall loading level of 90% (which corresponds to the applied ARF factor of 0.9 for the Nakivubo catchment) underestimates the total flood volume by 15.9 – 33.9% (Table 5.1) at higher $T (T \geq 25$ yrs).
Table 5.1: Comparison between simulated total flood volume results for (a) 90% spatial rainfall loading (b) uniform rainfall loading with ARF factor applied

<table>
<thead>
<tr>
<th>Return period, $T$ (yrs)</th>
<th>Total flood volume ($\times 10^3$ m$^3$)</th>
<th>90% spatial rainfall loading</th>
<th>Uniform loading, ARF applied</th>
<th>Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T = 25$</td>
<td></td>
<td>404.8</td>
<td>340.5</td>
<td>15.9</td>
</tr>
<tr>
<td>$T = 50$</td>
<td></td>
<td>685.7</td>
<td>453.0</td>
<td>33.9</td>
</tr>
<tr>
<td>$T = 100$</td>
<td></td>
<td>856.1</td>
<td>640.4</td>
<td>25.2</td>
</tr>
</tbody>
</table>

5.3.1.2 Effect on mean flood duration

The results generally suggest that for all rainfall return periods, ‘failure’ of about 20% of the sub-catchments results in the highest increase in the mean flood duration. When the sub catchment ‘failure’ exceeds 20%, minimal variation in the mean flood duration is observed for all considered $T$ (Figure 5.5). A slight reduction in the mean flood duration is observed at higher sub catchment ‘failure’ levels, which is due to the effect of ‘averaging’ i.e. the number of flooded nodes increases with increasing total flood volume. Subsequently, the effect of ‘averaging’ leads to more stable results and in some instances lower mean values of the flood duration. The results also suggest for higher $T$ (i.e. 25, 50 and 100 years), that use of ARFs (with the assumption of uniform loading) slightly underestimates the mean flood duration (by 3.4 – 10.1%) when sub catchment ‘failure’ levels exceed 15% (Table 5.2).

Table 5.2: Comparison between simulated mean flood duration results for (a) 90% spatial rainfall loading (b) uniform rainfall loading with ARF applied

<table>
<thead>
<tr>
<th>Return period, $T$ (yrs)</th>
<th>Mean flood duration (minutes)</th>
<th>90% spatial rainfall loading</th>
<th>Uniform loading, ARF applied</th>
<th>Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T = 25$</td>
<td></td>
<td>136.2</td>
<td>130.0</td>
<td>4.5</td>
</tr>
<tr>
<td>$T = 50$</td>
<td></td>
<td>152.4</td>
<td>147.3</td>
<td>3.4</td>
</tr>
<tr>
<td>$T = 100$</td>
<td></td>
<td>163.0</td>
<td>146.5</td>
<td>10.1</td>
</tr>
</tbody>
</table>
Figure 5.5: Generated UDS failure envelopes showing the effect of spatial rainfall distribution on total flood volume (a-d) and mean nodal flood duration (e-h) for 140 minute block rainfall events with various rainfall return periods. The red dashed dot horizontal line (ARU) shows computed values of total flood volume and mean nodal flood duration using corresponding areally reduced uniform rainfall.
5.3.2 Effect of a rapid increase in rainfall intensity on flooding

5.3.2.1 Effect on total flood volume

To model the effect of a rapid increase in rainfall intensity, GRA is carried out using the 70 minute block rainfall events as functional loading inputs. The GRA results indicate that when compared to the 140 minute block rainfall events, the 70 minute block rainfall events result in higher loss of system functionality magnitude at all considered rainfall return periods (Figure 5.6). The effect on total flood volume is more pronounced for when the spatial rainfall loading exceeds 40%. The results indicate that the 70 minute block rainfall events result in a significant increase of 41 – 135% (Table 5.3) in the simulated total flood volume (at 90% sub catchment ‘failure’ level) when compared to the 140 minute block rainfall events for all considered T.

Table 5.3: Effect of short duration, high intensity block rainfall events on total flood volume

<table>
<thead>
<tr>
<th>Return period, T (yrs)</th>
<th>Total flood volume (x10^3 m^3)</th>
<th>140 minute block rainfall events</th>
<th>70 minute block rainfall events</th>
<th>Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T = 5</td>
<td></td>
<td>149.6</td>
<td>351.7</td>
<td>135.0</td>
</tr>
<tr>
<td>T = 25</td>
<td></td>
<td>404.8</td>
<td>694.5</td>
<td>71.6</td>
</tr>
<tr>
<td>T = 50</td>
<td></td>
<td>685.7</td>
<td>1,025.8</td>
<td>49.6</td>
</tr>
<tr>
<td>T = 100</td>
<td></td>
<td>856.1</td>
<td>1,208.1</td>
<td>41.1</td>
</tr>
</tbody>
</table>

5.3.2.2 Effect on mean flood duration

In contrast to the flood volume results, the 70 minute block rainfall events result in slightly lower mean flood duration values when compared to corresponding 140 minute block rainfall events. The effect is pronounced when sub catchment ‘failure’ levels exceed 10% (Figure 5.6). GRA results show that the simulated mean nodal flood duration values for the 70 minute block rainfall events are lower than corresponding values for the 140 minute block rainfall events (at 90% sub catchment ‘failure’ level) by 25 – 40.8% (Table 5.4)
Table 5.4: Effect of short duration, high intensity block rainfall events on mean flood duration

<table>
<thead>
<tr>
<th>Return period, $T$ (yrs)</th>
<th>Mean flood duration (minutes)</th>
<th></th>
<th></th>
<th>Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>140 minute block rainfall events</td>
<td>70 minute block rainfall events</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T = 5$</td>
<td>131.5</td>
<td>77.8</td>
<td></td>
<td>40.8</td>
</tr>
<tr>
<td>$T = 25$</td>
<td>147.8</td>
<td>88.2</td>
<td></td>
<td>40.3</td>
</tr>
<tr>
<td>$T = 50$</td>
<td>145.6</td>
<td>103.4</td>
<td></td>
<td>28.9</td>
</tr>
<tr>
<td>$T = 100$</td>
<td>152.9</td>
<td>114.7</td>
<td></td>
<td>25.0</td>
</tr>
</tbody>
</table>
Figure 5.6: Mean values of GRA results obtained using 140 minute and 70 minute block rainfall events showing the effect of increased rainfall intensity on total flood volume (a-d) and mean duration of nodal flooding (e-h) for various rainfall return periods.
5.3.3 Functional resilience index

The computed functional resilience indices for the considered block rainfall loading scenarios are presented in Figure 5.7. The computed design functional resilience index (0.91) represents the design flood protection level of service delivered by the existing UDS. The results also indicate that occurrence of shorter duration, high intensity rainstorms with higher return periods, significantly reduces the residual functionality of the UDS and hence it’s functional resilience to extreme rainfall. For example occurrence of the 50yr70 minute and 100yr70 minute block rainfall events result in degradation of the system’s design functional resilience of 24% and 32% respectively.

![Figure 5.7: Computed functional resilience indices for the existing UDS at various block rainfall loading scenarios](image)

5.4 Discussion of results

The results of the study suggest that the resulting loss of functionality of the existing UDS increases with increasing block rainfall event magnitudes. In addition, the study results indicate that the loss of system functionality is more sensitive to functional loading resulting from the short duration, high intensity block rainfall
events when compared to corresponding lower intensity block rainfall events and that this sensitivity is higher when the spatial rainfall loading extent exceeds 40%. This therefore suggests that the existing UDS exhibits low levels of resilience to extreme rainfall that could result from anticipated future climate change or climate variability.

Secondly, the study results also suggest that current approaches which use uniform rainfall loading inputs (with ARFs applied) may lead to overestimation of the magnitude of flooding resulting from a given rainfall event when the actual spatial rainfall extent is less than 70% of the total catchment area. However, for rainfall events that cover that entire catchment, use of uniform spatial rainfall loading underestimates the resulting magnitude of flooding. These results suggest that effective design (or sizing) of catchment scale resilience enhancement strategies in large urban catchments such as distributed storage or rainwater harvesting systems should apply spatially distributed rainfall inputs to achieve accurate results.

Thirdly, the generated sub catchment ‘failure’ envelopes suggest that in addition to the areal rainfall extent, storm movement, which may result from a change of wind direction (e.g. Vaes et al., 2005) during a given extreme rainfall event affects UDS performance and hence its functional resilience. This is confirmed in this study by targeted (as opposed to pseudo random) failure of sub catchments which effectively, simulates the effect of increasing spatial rainfall loading on UDS performance. Results obtained by progressive sub catchment loading from upstream to downstream parts of the catchment, results in higher failure impacts. On the other hand, increasing spatial rainfall loading from downstream to upstream parts of the catchment, leads to lower flooding impacts. These results are attributed to the spatially non-uniform system hydraulic loading during non-stationary rainstorms. As the spatial rainfall loading is gradually extended to cover downstream parts of the catchment, the generated flows from upstream parts of the catchment reach downstream links just when the local (downstream) storm run-offs are entering the UDS leading to higher flooding impacts.
5.5 Conclusions

In this chapter, the developed GRA method has been developed and applied to evaluate the functional resilience of an existing UDS in Kampala, Uganda when subject to a wide range of extreme rainfall loading conditions. The developed methodology facilitates improved understanding of the hydraulic performance behaviour of existing UDSs during unexpected extreme events. It also enables the effect of spatial rainfall distribution to be explicitly considered in UDS resilience evaluation with reduced computational complexity. From the study, the following conclusions specific to the Kampala city are drawn:

- Occurrence of short duration, high intensity rainfall events leads to significant loss of system functionality magnitude but has less effect on failure duration when compared to corresponding lower intensity rainfall events. Globally, it is concluded that short duration, high intensity rainfall events (i.e. 70 minute block rainfall) result in more significant reduction of the existing UDS’s functional resilience of 24 – 32%.

- Because the short duration events lead to higher loss of functionality magnitude but less effect of duration, it is suggested that implementation of innovative multifunctional infrastructure for example multifunctional rainwater harvesting systems and intentional design of specific road network sections to enable safe conveyance of exceedance flows could provide a promising option for enhancing global resilience to extreme events in Kampala.

- Use of areal reduction factors can lead to overestimation of the magnitude of flooding resulting from extreme rainfall events with higher return periods \((T > 25 \text{ yrs})\) when the actual spatial rainfall extent is less than 70% of the total catchment area, but underestimates the magnitude of flooding for rainfall events that cover entire catchments (higher spatial rainfall extents).

- Future planning and design of resilience enhancement strategies should apply spatially distributed rainfall inputs to enable effective design/size of potential adaptation strategies and therefore to minimise erroneous and costly adaptation decision making (e.g. Gersonius et al., 2013).
Furthermore, the following general conclusions on evaluation of functional resilience in UDSs are drawn:

- For large urban catchments, the effect of *spatial rainfall variation* can lead to spatially non-uniform system hydraulic loading, which significantly influences the hydraulic performance behaviour during failure. New resilience based evaluation guidelines such encourage the use of spatially distributed rainfall inputs for when assessing functional resilience to extreme rainfall in large urban drainage networks.

- The use of block rainfall events provides a practical and computationally efficient method that can be applied by water utilities/companies for diagnostic assessment of functional resilience in existing or planned UDSs particularly in developing country cities that may lack high resolution spatial temporal rainfall data sets.
Chapter Six

6. Global resilience to structural failure

This chapter develops and tests the GRA method to evaluate the effect of structural failure on the ability of an urban drainage system (UDS) to minimise the magnitude and duration of flooding. Using a simplified UDS (Case study 1) described in section 4.1, the system’s resilience to cumulative pipe failure is tested and characterised. Furthermore, the effect of implementing two potential adaptation strategies on minimising the loss of system functionality and whole life cost of UDSs is investigated.

Section 6.1 introduces the chapter, describes the study objectives and adopted methodology. Section 6.2 describes the modelling steps, initial system state performance assessment, GRA implementation and cost benefit analysis of the proposed adaptation strategies. Subsequent sections present the results (Section 6.3), results discussion (Section 6.4) and chapter conclusions (Section 6.5).

6.1 Introduction

Building resilience in UDSs requires explicit consideration of ‘all possible threats’. Therefore, in addition of functional failures (investigated in chapter 5), the contribution of structural failures to flooding in cities requires further investigation. Current evaluation approaches take a limited view of functional resilience and no view of structural resilience. Consequently, there is a need to systematically evaluate the global resilience of UDSs to structural failures so as to facilitate holistic understanding of UDS performance during unexpected failures and to provide a basis for evaluating effectiveness of potential adaptation (resilience-enhancing) strategies.
The main objective of this chapter is to further develop and extend the GRA method to evaluate the effect of structural failures on the ability an UDS to minimise the resulting flooding magnitude and duration. Using a simplified synthetic UDS, which requires rehabilitation, the effect of cumulative pipe failure is investigated. Pipe failure is used to model potential structural failures in UDSs such as pipe (sewer) collapse, blockages and bed load sediment deposition (Butler and Davies, 2011; Mugume et al., 2015a, 2015b). In addition, two adaptation strategies in which redundancy and flexibility may be enhanced are investigated, namely; downstream centralised storage (CS) and upstream distributed storage (DS) strategies respectively. The effect of the strategies on minimising the resulting loss of system functionality is evaluated and the discounted total cost of each strategy is evaluated considering a design life of 50 years.

6.2 Simulations and global resilience analysis

6.2.1 Initial state system performance assessment

The existing UDS (Figure 6.1a) requires rehabilitation because its current configuration leads to unacceptable surface flooding during extreme rainfall events (Figure 6.1a). This system is a baseline configuration in which the pipes provide the hydraulic capacity of the system with no storage devices i.e. business as usual (BAU) strategy

In addition, two adaptation strategies are modelled and their effect on enhancement of system resilience is investigated:

a) **Centralised storage (CS) strategy**: a large storage tank is introduced upstream of pipe C24 to minimise downstream flooding by enhancing peak flow attenuation effects (Figure 6.1b). It is noted that the most in urban drainage practice, the most widely chosen location of centralised storage tanks is usually further downstream i.e. downstream of pipe C24 in Figure 6.1. However, in this case study, a more optimal location of the storage tank i.e. upstream of pipe C24 was chosen to minimise the flooding impacts at confluence node where the three branches of the UDS meet.
b) **Distributed storage (CS) strategy**: 9 spatially distributed upstream storage tanks (same total storage volume as the CS strategy) are introduced at the outlets of each sub catchment to enhance flexibility at critical points in the network at sub-catchment level (Figure 6.1c).

![Figure 6.1: Layout of (a) Existing UDS with no storage (BAU strategy); (b) UDS with centralised (downstream) storage (c) UDS with upstream distributed storage](image)

Model simulations are carried out in SWMM v5.1 to investigate the effect of cumulative pipe failure on resulting loss of functionality (surface flooding) for the three system configurations in their respective initial states. The initial state (i.e. non-failed state) of a system refers to the ‘planned’, ‘as built’ or ‘desired’ condition before occurrence failure (Johansson, 2010). Flooding is simply modelled using a flood cone with all surface flows returning to the node from which they discharged (Rossman, 2010). In order to test the performance of the *BAU strategy* during failure conditions, model simulation is carried out using an up scaled extreme rainfall event with a total depth of 141 mm and duration of 100 minutes (i.e. change factor of 2.14 is applied to scale the observed extreme event return period, $T$ from 2 to 50 years).
6.2.2 Global Resilience Analysis

The system is subjected to increasing (cumulative) pipe failure scenarios (stress) in order to evaluate global performance (loss of system functionality) using total flood volume and mean nodal flood duration as key performance indicators. Pipe failure is modelled by significantly reducing pipe diameters, $D_p$ in the model from their initial values (non-failed state) to a very small value of 1 mm in order to model pipe failure (failed state), while maintaining the hydraulic connectivity required for the solution of the flow equations (Mugume et al., 2015a). A simulation run is carried out for the first random pipe failure scenario, then additionally, a second pipe is randomly failed and a second simulation run carried out. This is done cumulatively until all pipes in the network have failed.

Having considered the global performance of the existing system, the effect of implementing proposed adaptation strategies (in which the system configuration is altered to enhance its redundancy and flexibility properties) on improvement of its global resilience are investigated. The total number of simulations, $X_{sim}$ for a given GRA can be estimated using Equation 6.1. Overall, a total of 78 simulations are carried out.

$$X_{sim} = r_s i (N + 1)$$

(6.1)

6.2.3 Cost benefit analysis

For comparison of the strategies, the discounted cost, $PVC_{T,y}$ for each strategy is computed considering a design life of 50 years, using cost functions presented in chapter 3, section 3.8.

6.3 Results

6.3.1 Initial state system performance assessment

Model simulations are carried in SWMM v5.1 to test the performance of the BAU strategy and the proposed adaptation strategies in their respective initial states. For the BAU strategy, a high peak flow rate of 6.94 m$^3$/s is attained in downstream pipe C25 after a simulation period of 90 minutes (Figure 6.2a). Implementing the
CS strategy leads to a significant reduction in the peak flow rates in pipes located on the downstream end of the tank and/or network. Taking pipe C25 as an example, the CS strategy results in a higher reduction in the peak flow rate of 11.5% when compared the DS Strategy (2.4%). On the other hand, implementing the DS strategy results in significant reduction in flow rates in upstream pipes for example 15.3% and 8.3% in pipes C8 (Figure 6.2c) and C1 (Figure 6.2d) respectively. In some instances, the CS strategy results in slight increases in the peak flows rates in pipes located upstream of the tank; for example increases of 9.9% and 2.3% result in pipes C9 (Figure 6.3b) and C8 (Figure 6.2d) respectively.

Figure 6.2: Simulated flows in pipes C25, C9, C8 and C1. The flows are simulated in the initial system states i.e. for the BAU, CS and DS strategies before the UDS configurations are subjected to stresses resulting from cumulative pipe failure
6.3.2 Global resilience to cumulative pipe failure

The results of the GRA analysis are presented in Figure 6.3. The results indicate that the BAU strategy minimises the resulting loss of system functionality with respect to both total flood volume and flood duration at lower pipe failure levels (< 28%). However, further increase in the percentage of failed pipes lead significantly high and stepwise loss of system functionality when pipe failure levels exceed 28%. Implementing the CS strategy results in a slight reduction in the flood volume and mean nodal flood duration; with the reduction being effective at low (<48%) pipe failure levels (Figure 6.3a).

![Figure 6.3: Effect of cumulative pipe failure on (a) flood volume (b) mean duration of nodal flooding for the existing UDS (BAU), centralised storage and distributed storage strategies](image)

In contrast, implementing the DS strategy results in a significant reduction in total flood volume at all pipe failure levels implying the effect of cumulative pipe failure (stress) on loss of system functionality magnitude (strain) is reduced. In the case of the flood duration, the strain on the system increases to a maximum of 0.66 hours and remains almost constant when the cumulative pipe failure level exceeds 32% (Figure 6.3b).

However, in real-world urban drainage systems, it is unlikely that large percentage of the pipes would fail simultaneously. In Figure 6.4 below, the resilience analysis
results are plotted only for pipe failure levels up to 48%. Furthermore, the percentage reduction in the simulated flooding impacts (total flood volume and mean nodal flood duration) are computed at each link failure level and presented in Table 6.1.

Figure 6.4: Effect of cumulative pipe failure on (a) flood volume (b) mean duration of nodal flooding for the existing UDS (BAU), centralised storage and distributed storage strategies for pipe failure levels less than or equal to 48%.

The results indicate that the CS strategy leads to significant reduction in the total flood volume and mean nodal flood duration that ranges from 27.9 – 40.1% and 14.3 – 16.6% respectively at pipe failure levels less than 20%. However, the effectiveness of the CS strategy significantly reduced by increasing pipe failure levels.

In contrast, the DS strategy is the most effective and leads to very high reduction in the total flood volume that range from 104.8 – 566.6% for pipe failure levels less than 48%. The strategy also leads to a reduction in the mean nodal flood duration that ranges from 35.6 – 57.4%.
The results described above can in principle be combined with results of sewer deterioration modelling (which is focused on predicting the probability of occurrence of future sewer failures based on current system condition) to further provide a more rational basis for prioritising future UDS adaptation that includes both capital and asset management investments.

### Table 6.1: Mean values of GRA results for the BAU, CS and DS strategies for cumulative pipe failure levels ≤ 48%.

<table>
<thead>
<tr>
<th>% of failed links</th>
<th>Total flood volume ($x10^3 m^3$)</th>
<th>Mean nodal flood duration (hrs)</th>
<th>% reduction in total FV</th>
<th>% reduction in mean FD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BAU</td>
<td>CS</td>
<td>DS</td>
<td>BAU</td>
</tr>
<tr>
<td>0</td>
<td>10.9</td>
<td>6.5</td>
<td>1.6</td>
<td>0.71</td>
</tr>
<tr>
<td>4</td>
<td>11.5</td>
<td>8.3</td>
<td>2.3</td>
<td>0.81</td>
</tr>
<tr>
<td>8</td>
<td>11.5</td>
<td>8.3</td>
<td>2.3</td>
<td>0.81</td>
</tr>
<tr>
<td>12</td>
<td>11.5</td>
<td>8.3</td>
<td>2.3</td>
<td>0.81</td>
</tr>
<tr>
<td>16</td>
<td>11.5</td>
<td>8.3</td>
<td>2.3</td>
<td>0.81</td>
</tr>
<tr>
<td>20</td>
<td>12.3</td>
<td>10.6</td>
<td>3.9</td>
<td>0.87</td>
</tr>
<tr>
<td>24</td>
<td>12.3</td>
<td>10.6</td>
<td>3.9</td>
<td>0.87</td>
</tr>
<tr>
<td>28</td>
<td>14.4</td>
<td>13.6</td>
<td>5.9</td>
<td>0.90</td>
</tr>
<tr>
<td>32</td>
<td>14.4</td>
<td>13.6</td>
<td>5.9</td>
<td>0.90</td>
</tr>
<tr>
<td>36</td>
<td>14.4</td>
<td>13.6</td>
<td>5.9</td>
<td>0.90</td>
</tr>
<tr>
<td>40</td>
<td>14.4</td>
<td>13.6</td>
<td>5.9</td>
<td>0.90</td>
</tr>
<tr>
<td>44</td>
<td>16.0</td>
<td>15.7</td>
<td>7.8</td>
<td>1.01</td>
</tr>
<tr>
<td>48</td>
<td>16.0</td>
<td>15.7</td>
<td>7.8</td>
<td>1.01</td>
</tr>
</tbody>
</table>

### 6.3.3 Cost benefit analysis

The results of the cost-benefit analysis are presented in Figure 6.4. Plausibly, the computed capital costs of the CS and DS strategies are higher than the BAU strategy by 27% and 35% respectively, which is attributed to the addition of storage devices. When the costs of failure (direct tangible flooding costs) are taken into consideration, the CS strategy leads to a net benefit of 14.5% considering a system’s service life of 50 years. In contrast, the DS strategy generates a significantly high net benefit of 39.2% (Figure 6.5).
6.4 Discussion

The GRA results for the BAU strategy suggest that the system minimises (buffers) the resulting loss of system functionality and hence maintains considerable residual capacity and at lower pipe failure levels. The results also suggest that resulting loss of system functionality increases rapidly with progressive increase in pipe failure levels. The occasional ‘plateaus’ in the simulated flooding impacts (Figure 6.3) are attributed to failure of less critical pipes (e.g. an upstream pipe conveying low flows) with leads to minimal increase in the simulated flood volume and duration despite further increase pipe failure levels. The rapid increase in simulated flooding impacts is attributed to failure of more critical pipes. Globally, the results suggest that the existing system exhibits low levels of resilience to cumulative pipe failure.

The effect of introducing a large storage tank only minimises the flow rates in the pipes connected on the downstream end of the tank. Therefore, by implementing the CS strategy, the effect on reduction of flow rates is limited to only to downstream pipes C24 and C25. This however, leads to sub-optimal use of the total installed tank volume which in turn results in minimal flow attenuation effects at a whole (global) UDS level. Consequently, the CS strategy leads to minimal
Structural resilience evaluation

reduction of the resulting loss of functionality magnitude and duration. As a consequence, only a slight improvement in global resilience to cumulative pipe failure is achieved by the strategy.

In contrast, the results of the DS strategy suggest that increasing the spatial distribution of control options in upstream parts of the network ensures optimal global performance during pipe failure scenarios. The spatially distributed storage devices reduce the peak flow rates of storm water entering all upstream pipes (i.e. pipes receiving direct inflows from the sub catchments). This strategy consequently enables optimal use of the total installed tank volume for reduction storm water inflow rates into the UDS. As a result, the UDS, although structurally degraded is able to maintain higher residual functionality due to the reduced functional loading of the UDS. This strategy is therefore more resilient to cumulative pipe failure when compared to the other two strategies as demonstrated by the significant mean reduction in the total flood volume and duration of nodal flooding that ranges from 104.8 – 566.6% and 35.6 – 57.4% respectively (for pipe failure levels ≤ 48%).

Taking costs into consideration, the capital costs of the CS and DS strategies are higher by 27% and 35% respectively due to the addition of storage devices. When direct tangible flooding costs are taken into consideration, the CS strategy leads to 14.5% reduction in discounted total costs (net benefit) over the considered UDS service life of 50 years. On the other hand, the DS strategy results in higher net benefits (39.2% reduction in discounted total costs), over the UDS’s service life of 50 years which is attributed to its effectiveness in minimising the magnitude and duration and flooding and hence a reduction in attendant direct tangible flooding costs.

6.5 Conclusions

In this chapter, the GRA approach has been developed and applied to investigate the performance of a simplified synthetic UDS when subject to random cumulative pipe failure scenarios. The methodology is also applied to test the effect of implementing centralised and distributed storage strategies (which enhance
redundancy and flexibility properties of the tested UDS) on minimisation of resulting loss of system functionality and cost when subject to increasing pipe failure levels. From the study results, the following conclusions specific to the tested synthetic UDS are drawn:

- The existing UDS (BAU strategy) is less resilient to cumulative pipe failure and this may be attributed to hydraulic overloading of the UDS in its initial state coupled with the dendritic network topology.
- Introducing a large centralised storage tank (CS strategy) minimally reduces the resulting loss of system functionality with respect to total flood volume and mean nodal flood duration by 4.8% and 1.9% respectively. It is concluded that the CS strategy is less effective for enhancing the global UDS resilience to cumulative pipe failure.
- The use of smaller distributed storage tanks (DS strategy) significantly reduces the resulting loss of system functionality with respect total flood volume and mean nodal flood duration by 50.1% and 46.7% respectively. It is concluded that in contrast to the CS strategy, the DS strategy is more effective for enhancing global UDS resilience to cumulative pipe failure.
- Although the implementing DS strategy is associated with higher initial capital costs due to the additional cost of smaller distributed storage tanks, the strategy generates a positive net benefit of 39.2% when direct tangible surface flooding costs over a system design life are taken into consideration.

The study has demonstrated that that loss of system functionality and costs caused by exceptional structural loading conditions can be minimised if failure scenarios are taken into consideration during UDS design or rehabilitation. It is further concluded that enhancement of system flexibility properties (attributes), for example through increasing the spatial distribution of control options enhances the UDS’s ability to minimise the resulting loss of functionality and hence increasing its global resilience to unexpected system failures.
It is noted that the results and conclusions of this chapter have been based on performance assessment of a synthetic case study UDS. In addition, pipe failure has been modelled by reduction of the pipe diameter of the each randomly failed pipe. In chapter 7, to further validate its suitability and to achieve more consistent results, the GRA method is extended to evaluate a wide range of pseudo random cumulative link failures scenarios using a case study of an existing real-world UDS in Kampala (chapter 7). Furthermore, link failure is simulated by increasing the Manning’s roughness coefficient, $n$ in each randomly selected link to a very high value.

It is further postulated that by consideration of large number of random failure scenarios, the full link failure scenario space that includes ‘all possible’ link failure combinations that range from predictable (identifiable) single link failures ($N-1$), to less predictable (unexpected) multiple link failure combinations ($N-i$); (where $i$ is the number of failed links) is explored and thereby increasing the accuracy and consistency of the GRA results (Johansson and Hassel, 2012; Mugume et al., 2015b).
Chapter Seven

7. Global analysis of structural resilience and effectiveness of adaptation strategies in Kampala

This chapter further develops and applies the GRA method to systematically evaluate the performance of an existing UDS in Kampala when subject to a wide range of random structural failure scenarios resulting from cumulative link failure and to investigate the effectiveness of a set of potential adaptation strategies in enhancing system resilience to cumulative link failure. Sections 7.1 and 7.2 provide an introduction and describe the main steps involved in applying the GRA approach to investigate the structural resilience to flooding in both existing and adapted UDSs using a case study of the Nakivubo UDS (Case study 2) that has been described in detail in Chapter 4.

Section 7.3 describes results of GRA for the existing UDS and for the tested adaptation strategies namely: centralised storage (CS), distributed storage (DS) and improved operation & maintenance (O&M) strategies with respect to minimisation of the resulting loss of system functionality when subject to cumulative link failure. In sub section 7.3.4, the resulting link failure envelopes are presented. In sub sections 7.3.5, the results of computed structural resilience index, $Res_o$, and the generated resilience envelopes are presented. Sub section 7.3.6 presents the results of both discounted total cost calculations and net benefits generated by implementing the respective adaptation strategies. The subsequent sections discuss the results (section 7.4) and present the main chapter conclusions (section 7.5).
7.1 Introduction

The main objective of this chapter is to: (i) apply the GRA method to quantify the performance of an existing UDS in Kampala, Uganda when subject to a wide range of random structural failure scenarios resulting from cumulative link failure and (ii) to evaluate the effectiveness of a set of potential adaptation strategies in minimising the magnitude and duration of flooding in the case study area.

Random and cumulative link failure is used to represent possible structural failure modes such as sewer collapse, blockages and sediment deposition in closed systems and blockage resulting from deposition of solid waste and washed-in sediments in open channel systems (Mugume et al., 2015b). Failing links randomly ensures that all links, $N$ in the system have an equal probability of being removed (Johansson and Hassel, 2012). Furthermore, a step by step increase in sewer failure levels enables the exploration of the full sewer failure scenario space that ranges from normal (predictable or commonly occurring) failure scenarios such as single component ($N-1$), or two component ($N-2$) failure modes but also other unexpected (unpredictable) scenarios ($N-i$) involving simultaneous failure of a large number of components, $i$ (Johansson, 2010; Mugume et al., 2015b; Park et al., 2013).

The developed methodology is then applied to test the effect of implementing a set of potential adaptation strategies on minimizing loss of functionality during the considered structural failure scenarios. The tested strategies include: (a) introducing a large centralised detention pond ($CS$ strategy) (b) use of several spatially distributed storage tanks ($DS$ strategy); and (c) improved system operation and maintenance ($O&M$ strategy).

The performance of existing and adapted UDSs during the considered failure scenarios is quantified using the total flood volume and mean nodal flood duration as failure magnitude and failure duration indicators respectively. Based on the results of the analysis, link failure envelopes which represent the extent (range) of resulting loss of system functionality (impacts) at each link failure level are determined by computing the minimum and maximum values of the total flood.
volume and mean nodal flood duration results generated by running a large number of model simulations involving a wide range of pseudo random and cumulative link failure scenarios.

The generated link failure envelopes, which show the upper and lower limits (bounds) of the resulting loss of functionality for each considered link failure level are determined based on the hydraulic simulation results from a total of 65,600 scenarios. The failure envelopes reflect vital system resilience properties that determine the resulting loss of functionality when the system is subjected to increasing failure levels. Finally, the structural resilience index, $Res_o$ is used to quantify system residual functionality as a function of failure magnitude and duration. $Res_o$ is computed at each failure level for both the existing system and for the tested adaptation strategies.

The key strengths and novelty of the developed GRA method is that emphasis is shifted from accurate quantification of the probability of occurrence of sewer failures, to evaluating the effect of different sewer failure modes and extent, irrespective of their occurrence probability, on the ability of an UDS to minimise the resulting flooding impacts (Mugume et al., 2015b).

### 7.2 Simulations and global resilience analysis

#### 7.2.1 Existing Nakivubo UDS

The Nakivubo UDS (described in detail in Chapter 4) in its initial state or ‘business as usual’ (BAU) condition is not always clean due to insufficient maintenance and cleaning operations and inadequate solid waste management in the city (Sliuzas et al., 2013). This is represented in SWMM by setting the initial value of Manning’s $n$ to 0.020 in all the links. The value is chosen because it corresponds to the upper limit of the recommended range of $n$ values for concrete lined channels (Butler and Davies, 2011). In this study, a single extreme rainfall event (non-areally adjusted) described in chapter 4 is used as functional loading input for the GRA simulations. In addition, three proposed adaptation strategies are modelled and investigated. These are described in subsection 7.2.2.
7.2.2 Modelling the effect of adaptation strategies on UDS performance

Enhancing the resilience of an UDS during design or retrofit can be achieved by altering its configuration in order to enhance its redundancy and flexibility properties. In this chapter, three adaptation strategies are modelled and tested using the GRA methodology. These include:

i) **Centralised storage (CS) strategy:** A large centralised detention pond with a total storage volume of $3.15 \times 10^5$ m$^3$ is introduced upstream of link C47 (Figure 7.1b) to enhance system redundancy. This location of the detention pond, was chosen based on two main criteria; land availability and flow rates in the downstream links in the primary Nakivubo channel.

ii) **Distributed storage (DS) strategy:** 28 spatially distributed storage tanks with a combined total storage volume of $3.15 \times 10^5$ m$^3$ are introduced at the outlets of the sub catchments to enhance flexibility in crucial points in the network (Figure 7.1b). The DS strategy models the effect of upstream distributed source control. The volumes of the individual DS tanks are presents in Appendix Table A.4.

iii) **Operations and maintenance (O&M) strategy:** This strategy models the effect of improved asset management achieved through investments in system maintenance and cleaning so as to maintain its as-built hydraulic capacity and to improve flow conditions in the individual links (e.g. Butler and Davies, 2011). The strategy is modelled by changing the initial state Manning’s roughness coefficient, $n$ from 0.020 to 0.015 to represent a clean and well maintained system.
7.2.3 Initial state system performance assessment

In order to test the performance of the modelled existing UDS, simulations are carried out and flows are investigated at selected links in the system (refer to Figure 4.16 in Chapter 4). In addition, model simulations are also carried for the adapted systems in investigate their performance in their non-failed state. To assess their initial state performance, flow rates are investigated at selected upstream and downstream links for the existing UDS and for the proposed adaptation strategies. In addition, their global performance is quantified using total flood volume, mean nodal flood duration and number of flooded nodes. This analysis sets the base line performance before the systems are structurally degraded.

7.2.4 Global resilience to cumulative link failure

In this chapter, link failure is modelled in SWMM v5.1 by increasing the Manning's $n$ from its initial (non-failed) state value ($n = 0.020$) to a very high value ($n = 100$). The high value of $n$ was chosen because it significantly curtails the conveyance of flows in each failed link and hence enables modelling of complete failure of each link. To minimise computational complexity inherent in simulating all possible link failure combinations, convergence analysis (Mugume et al., 2015b; Trelea, 2003) is applied to determine the minimum number of random link failure sequences, $rs_x$ that should be simulated to achieve consistent GRA results.

Figure 7.1: Layout of adapted UDS (a) centralised storage strategy (CS) and (b) upstream distributed storage strategy
7.2.4.1 Convergence analysis

The main steps followed in carrying out convergence analysis are:

a) GRA is carried out using 5 random sequences (410 failure scenarios) and the mean values of the total flood volume are determined.

b) The procedure is repeated for 10 (820 failure scenarios), 25 (2050 failure scenarios), 50 (4100 failure scenarios), 100 (8200 failure scenarios), 150 (12,300 failure scenarios) and 200 (16,400 failure scenarios) sequences.

c) The percentage deviation, PD, the between computed mean values is computed for each step-wise increase in $rs_i$, i.e. for $i: i = \{5,10\}; \{10, 25\}; \{25, 50\}, \{50, 100\}, \{100,150\}$ and $\{150,200\}$ (Figure S3)

The results obtained from 5 random sequences indicate the largest variation in the mean values (up to 7.5%) occurs at lower link failure levels (<10% of the failed links), with convergence occurring at higher links failure levels (Figure 7.2).

![Figure 7.2: Convergence of GRA results after 200 random cumulative failure sequences, rs;](image)

The results also indicate that increasing the number of random link failure sequences reduces this variation. A convergence is obtained after 50 random failure sequences with a maximum deviation of 4.5%. The maximum deviation is further reduced to 3.5%, 2.6% and 1.1% by considering 100, 150 and 200 random...
failure sequences respectively. Considering 200 random failure sequences covers all $N-1$ (single link) scenarios and covers a statistically significant proportion of $N-2$ (two link) scenarios (6.2%) and $N-3$ (three link failure) scenarios (0.23%). Consequently, a minimum of 200 random failure sequences is adopted for the GRA.

### 7.2.4.2 GRA implementation

The GRA method is applied to investigate the performance of existing UDS when subject to a total of 16,400 cumulative link failure scenarios involving a wide range (a total of 200) pseudo random and progressive link failure sequences. In addition, the GRA methodology is applied to test each of the proposed UDS adaptation strategies.

For the O&M strategy, opposed to complete failure, partial failure of the links is considered to reflect the effect improved O&M on mitigation of sewer failures in UDSs. Partial failure is modelled using Manning's $n$ of 0.15 which represents an unmaintained channel, with ingrown grass or sediments that significantly reduces the flow conveyance capacity of the whole minor system (Arcement and Schneider, 1989; Rossman, 2010).

The overall performance of the existing system is quantified by carrying out a large number of model simulations at each randomly generated link failure level (number of failed links). At each link failure level, (global) system performance is quantified by computing the average values of the total flood volume and nodal flood duration. For the existing UDS, performance is quantified by simulating a total of 16,400 link failure scenarios generated from 200 random link failure sequences. For each tested strategy (that is, CS, DS and O&M strategies), an additional 16,400 link failure scenarios are simulated. Overall, a total of 65,600 link failure scenarios generated from a total of 800 random link failure sequences are simulated.
7.2.5 Cost-benefit analysis

The discounted total cost of each strategy is evaluated considering a design life of 50 years using the cost equations described in detail in chapter 3. Furthermore, in a recent study in Kampala (NWSC, 2014a), the unit capital cost of large tanks with storage volumes, \( V_{ST} \geq 238 \text{ m}^3 \) is estimated at 238 USD/m\(^3\) (i.e. 144.5 £/m\(^3\) considering an average exchange rate of 1USD = 0.607 GBP for 2014). Based on this, the capital costs of storage tanks in Kampala are computed using Equation 7.1

\[
C_{ST} = \sum_{i=1}^{S} 144.5 V_{ST}
\]  
(7.1)

7.3 Results

7.3.1 Initial system state performance assessment

In order to test the performance of the modelled existing UDS, simulations are carried out and flows are investigated at selected links in the system. The hydraulic data on the selected open channel cross sections is presented in Table A.3. The results of the assessment of the initial system states are presented in Figure 7.3. Considering the existing UDS, relatively lower peak flow rates, are simulated in most upstream links. The flow rates increase along the system leading to very high peaks in downstream links, for example a rather high peak flow rate of 332 m\(^3\)/s is simulated in downstream link C81 after an elapsed time of 80 minutes.

By implementing the CS strategy, a reduction of 29.6% in the peak flow rate is achieved. In addition, by implementing the DS strategy, modest reduction of 10.2% in peak flow rate in link C81 is achieved. On the other hand, implementing the O&M strategy increases the peak flow rate in link C81 by 29.3\% (Figure 7.3a). The performance is different when upstream links are considered. Taking link C40 as an example, the DS strategy reduces the peak flow rate by 27.8% that is; from 69.7 m\(^3\)/s to 50.3 m\(^3\)/s, while the CS strategy has no effect of the simulated flow rates. Similar to the results of the downstream link C81, the O&M strategy leads to an increase of 13.5\% in the peak flow rate (Figure 7.3c).
Considering the global performance of the existing UDS in its initial (non-failed) state, simulation results also indicate the system experiences significant flooding at a total of 57 nodes, representing a flood extent of 70.7%, with a total volume of flooding of 706,045 m$^3$ and mean nodal flood duration of 41 minutes.

Implementing the CS strategy reduces total flood volume by 10.4% and the number of flooded nodes by 17.5%. However, the strategy has a minimal effect of flood duration. On the other hand, the DS strategy leads to considerable reduction in total flood volume, mean flood duration and the number of flooded nodes of 42.6%, 22.1% and 33.3% respectively. Implementing the O&M strategy also considerably reduces the total flood volume by 23%. Also, the O&M strategy slightly lower reduces the mean flood duration (9.5%) and number of flooded nodes (8.8%). The detailed results of the initial state UDS performance are presented in Table 7.1
Table 7.1: Initial system state performance for the existing UDS and for the considered adaptation strategies

<table>
<thead>
<tr>
<th>Adaptation strategy</th>
<th>Flood volume (x $10^3$ m$^3$)</th>
<th>% change</th>
<th>Flood duration (hrs)</th>
<th>% change</th>
<th>Flooded nodes (no.)</th>
<th>% change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing UDS</td>
<td>706.0</td>
<td></td>
<td>0.688</td>
<td></td>
<td>57</td>
<td></td>
</tr>
<tr>
<td>CS strategy</td>
<td>632.3</td>
<td>10.4%</td>
<td>0.701</td>
<td>-1.8%</td>
<td>47</td>
<td>17.5%</td>
</tr>
<tr>
<td>DS strategy</td>
<td>405.0</td>
<td>42.6%</td>
<td>0.536</td>
<td>22.1%</td>
<td>38</td>
<td>33.3%</td>
</tr>
<tr>
<td>O&amp;M strategy</td>
<td>543.9</td>
<td>23.0%</td>
<td>0.625</td>
<td>9.2%</td>
<td>52</td>
<td>8.8%</td>
</tr>
</tbody>
</table>

However, the initial state performance assessment only quantifies its *functional resilience* when the system is subjected to the considered extreme rainstorm. The interesting next logical step of the study is to investigate whether a system’s *initial state* influences its performance when subjected to structural failures i.e. its *structural resilience*. The GRA approach described in detail in chapter 3 is applied in the next subsection characterise the performance of both the existing system and the considered adaptation strategies.

### 7.3.2 GRA of the existing UDS

The overall performance of the system is quantified by simulating total flood volume and mean duration of flooding resulting from 16,400 link failure scenarios generated from 200 random link failure sequences. The average values of the total flood volume and duration of nodal flooding are computed for all the considered link failure scenarios and are presented in Figure 7.4. The GRA results indicate that failure of just 10% of links leads to a disproportionately large increase of 91% in total flood volume (Figure 7.4a). Thereafter, further increase in the percentage of failed links leads to comparatively small increases in the total flood volume.
Figure 7.4: Effect of cumulative pipe failure on (a) total flood volume and (b) mean duration of nodal flooding for the Existing Nakivubo UDS ($ns\ mean$), for the centralised storage strategy ($cs\ mean$), for the distributed storage strategy ($ds\ mean$) and the operation & maintenance strategy ($om\ mean$).

The situation is very different for nodal flood duration, where results show that failure of 10% of links leads to just a 6% increase (Figure 7.4b). Globally, the results indicate that the failure duration increases from 41 minutes to 56 minutes representing an increase of 36.2% when all the links in the system are failed.

7.3.3 Effect of adaptation strategies on system performance

Similarly the argument in Chapter 6 (section 6.3.2), it is noted that the probability of simultaneous failure of a large percentage of links in a given UDS is low. Therefore, in Table 7.2 below, the resilience analysis for pipe failure levels up to 40% are presented.
Table 7.2: Mean values of GRA results for the existing UDS (NS), centralised storage (CS), distributed storage (CS) and improved operations and maintenance (O&M) strategies for link failure levels < 40%.

<table>
<thead>
<tr>
<th>% of pipes failed</th>
<th>Total Flood Volume ($x10^3$ m$^3$)</th>
<th>Mean nodal flood duration (hrs)</th>
<th>% reduction in total FV</th>
<th>% reduction in mean FD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NS</td>
<td>CS</td>
<td>DS</td>
<td>OM</td>
</tr>
<tr>
<td>0.0</td>
<td>706.0</td>
<td>632.3</td>
<td>405.0</td>
<td>543.9</td>
</tr>
<tr>
<td>1.2</td>
<td>901.1</td>
<td>786.7</td>
<td>548.6</td>
<td>615.0</td>
</tr>
<tr>
<td>2.5</td>
<td>1,025.0</td>
<td>890.2</td>
<td>682.9</td>
<td>703.0</td>
</tr>
<tr>
<td>3.7</td>
<td>1,126.5</td>
<td>967.3</td>
<td>772.0</td>
<td>773.8</td>
</tr>
<tr>
<td>4.9</td>
<td>1,197.4</td>
<td>1,021.4</td>
<td>835.8</td>
<td>836.4</td>
</tr>
<tr>
<td>6.2</td>
<td>1,246.5</td>
<td>1,074.8</td>
<td>897.5</td>
<td>894.3</td>
</tr>
<tr>
<td>7.4</td>
<td>1,299.1</td>
<td>1,124.6</td>
<td>931.7</td>
<td>930.0</td>
</tr>
<tr>
<td>8.6</td>
<td>1,329.6</td>
<td>1,156.3</td>
<td>957.6</td>
<td>958.2</td>
</tr>
<tr>
<td>9.9</td>
<td>1,348.7</td>
<td>1,187.0</td>
<td>974.0</td>
<td>988.4</td>
</tr>
<tr>
<td>11.1</td>
<td>1,370.9</td>
<td>1,211.1</td>
<td>988.2</td>
<td>1,012.3</td>
</tr>
<tr>
<td>12.3</td>
<td>1,384.6</td>
<td>1,233.5</td>
<td>997.1</td>
<td>1,026.6</td>
</tr>
<tr>
<td>13.6</td>
<td>1,395.9</td>
<td>1,250.6</td>
<td>1,005.1</td>
<td>1,043.6</td>
</tr>
<tr>
<td>14.8</td>
<td>1,401.3</td>
<td>1,263.4</td>
<td>1,011.2</td>
<td>1,061.9</td>
</tr>
<tr>
<td>16.0</td>
<td>1,407.0</td>
<td>1,281.9</td>
<td>1,017.4</td>
<td>1,073.6</td>
</tr>
<tr>
<td>17.3</td>
<td>1,409.0</td>
<td>1,295.0</td>
<td>1,016.7</td>
<td>1,082.9</td>
</tr>
<tr>
<td>18.5</td>
<td>1,414.7</td>
<td>1,307.7</td>
<td>1,015.6</td>
<td>1,098.6</td>
</tr>
<tr>
<td>19.8</td>
<td>1,418.7</td>
<td>1,323.4</td>
<td>1,015.8</td>
<td>1,112.1</td>
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<td>21.0</td>
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<td>1,333.9</td>
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<td>1,011.2</td>
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<tr>
<td>23.5</td>
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<td>1,352.2</td>
<td>1,009.8</td>
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<td>1,435.4</td>
<td>1,362.4</td>
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<td>1,143.8</td>
</tr>
<tr>
<td>25.9</td>
<td>1,439.0</td>
<td>1,367.9</td>
<td>1,004.1</td>
<td>1,147.0</td>
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<tr>
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<td>1,375.1</td>
<td>1,002.6</td>
<td>1,151.1</td>
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<tr>
<td>28.4</td>
<td>1,445.1</td>
<td>1,379.2</td>
<td>1,001.0</td>
<td>1,161.3</td>
</tr>
<tr>
<td>29.6</td>
<td>1,445.8</td>
<td>1,387.0</td>
<td>1,000.3</td>
<td>1,162.6</td>
</tr>
</tbody>
</table>
The effect of the CS strategy is a slight reduction of flood volume (2.6 – 14.7%) which occurs at lower link failure levels, and very little impact on mean flood duration. On the other hand, the DS strategy results in a significant reduction (27.7 – 31.7%) in total flood volume. At link failure levels greater than 20% any additional increase in link failure levels leads to minimal increase in total flood volume. The strategy also reduces the mean nodal flood duration by 22.1 – 25.4%.

The study results also indicate that implementing the O&M strategy presents a more effective option when compared to the CS strategy. The O&M strategy results in a considerable reduction in total flood volume of 18.5 – 31.7% and a slight reduction (3.3 – 9.6%) in the mean flood duration.

**7.3.4 Link failure envelopes**

The resulting link failure envelopes which represent the range of model solutions from the lowest to the highest flooding impacts computed at each link failure level are presented in Figure 7.5. For the existing UDS and considering the flood volume, a large range of deviation between the computed failure envelopes and the mean values (27 – 87%) is observed at lower link failure levels (<20%). A
convergence of both failure envelopes is observed at higher link failure levels (Figure 7.5a). The results from the nodal flood duration are different, and indicate a narrow range of deviation (< 26.3%) between resulting failure envelopes and the mean values at all link failure levels (Figure 7.5e).

In order to evaluate the effectiveness of the considered adaptation strategies, link failure envelopes for the considered adaptations are determined and plotted together with those of the existing UDS in Figure 7.5. Comparing the results of the CS strategy to those of the existing system, a slight downward shift of both the maximum and minimum flood volume failure envelopes is observed at lower link failure levels (< 40%), which represents the effect of the strategy in minimising the magnitude of flooding (Figure 7.5b). However, there is no significant effect at higher link failure levels (Figure 7.5f). Also, the results suggest that the CS strategy has minimal effect on the flood duration failure envelopes.

For the DS strategy, a significant downward shift in the total flood volume failure envelope (i.e. a reduction in the magnitude of flooding) is observed at all cumulative link failure levels (Figure 7.5c). The strategy also limits the additional increase in flood volume for link failure levels beyond 33% i.e. a flattening of the flood volume failure envelope is observed at higher link failure levels. The strategy also shifts the flood duration failure envelopes downwards (i.e. reduces the failure duration) for all considered link failure levels when compared the existing UDS (Figure 7.5g).

Furthermore, for the O&M strategy, a considerable downward shift of both the total flood volume (Figure 7.5d) and mean flood duration (Figure 7.5h) failure envelopes is observed at all link failure levels. The downward shift of the failure envelopes is higher than that achieved by implementing the CS strategy but less than the corresponding shift resulting from implementing the DS strategy.
Figure 7.5: Results of the generated link failure envelopes for total flood volume (a) – (d) and for mean duration of nodal flooding (e) – (h)
7.3.5 Structural resilience index

The resilience index (Res°) is computed using Equation 3.7 for all simulated link failure scenarios. Based on the computed indices, resilience envelopes which represent the residual functionality of the whole UDS as a function of both the failure magnitude and duration are determined by computing the minimum and maximum values of Res° at each link failure level for the existing system for the tested adaptation strategies (Figure 7.6). To facilitate comparison of the performance of the tested strategies, an assumed acceptable level of resilience threshold of 0.7 is plotted on each of the graphs, as an example of the minimum acceptable flood protection level of service (for example no property flooding) that needs to be achieved by the considered adaptation strategies.

The figure reveals large variations in Res° for the existing system and for the tested strategies at lower link failure levels (< 20%) with a convergence of the results occurring with increasing link failure levels. For the existing UDS, the computed mean values of Res° range from 0.54 to 0.66. When compared to the resilience threshold, the results indicate that the existing system crosses this threshold when link failure levels in system exceed 6.2%.

Considering the CS strategy, a slight improvement in Res° of 1.2 - 2.3% is observed. The results indicate that resilience index falls below the threshold value when link failure levels exceed 8.6%. When the distributed storage strategy is considered, higher mean values of Res° are computed (0.76 – 0.84). The results also indicate that for the DS strategy, the resilience threshold is not crossed at all link failure levels. Overall, the DS strategy leads to significant improvement in the Res° of 27.5 – 41.4%. The O&M strategy on the other hand results in a modest improvement in Res° of 18.4 – 21.7%, which is higher than corresponding results achieved by the implementing the CS strategy but less than those of the DS strategy.
Figure 7.6: Resilience envelopes showing maximum, mean, minimum values of $\text{Res}_o$ computed at each link failure level for (a) existing UDS, (b) CS strategy, (c) DS strategy and (d) O&M strategy. The red dashed horizontal line is an assumed minimum acceptable resilience level of service threshold of 0.7.
7.3.6 Cost benefit analysis

Discounted total costs for the existing UDS and for the tested adaptation strategies are computed by taking into consideration the direct tangible flood damages at various link failure levels (Equation 3.8). The resilience index is plotted against the net benefit in Figure 7.7. The results indicate that net benefit of implementing the CS strategy is minimal ( < 13%), and this is achieved at lower link failure levels (<10%). The results also indicate the O&M strategy performs considerably better than the CS strategy and results in net benefits of 18 – 31% at all considered link failure levels. In contrast, although the DS strategy is associated with higher upfront capital investment costs (due to additional cost of storage tanks), it significantly minimises the discounted total costs, and thus results in higher net benefits (27 – 40%) over the design life of the UDS, whilst maintaining higher levels of resilience at all considered link failure levels.

**Figure 7.7**: Plot of the resilience index against the computed percentage net benefits for the considered adaptation strategies. The values are computed at various link failure levels, ranging from single link (N-1) to complete failure (N-i) of all links, i in the UDS.
7.4 Discussion of results

7.4.1 Existing system

Considering the existing system, random failure of less than 20% of the links leads to disproportionately high degradation of system functionality magnitude (i.e. total flood volume). The disproportionately high loss of system functionality suggests that failure of a small fraction of links rapidly reduces the global hydraulic conveyance capacity of the (minor) system. This result is also confirmed by critical component analysis (CCA) involving targeted failure of single (individual) links in the UDS (Figure 7.8).

![Percentage increase in total flood volume resulting from critical component analysis involving single link failure](image)

**Figure 7.8:** Percentage increase in total flood volume resulting from critical component analysis involving single link failure

This therefore suggests that the existing UDS exhibits low levels of resilience to sewer failures. This could be attributed to the already insufficient hydraulic capacity of the system (due to use of an extreme rainstorm for modelling purposes) but could also be attributed to other key factors such as its dendritic network topology and limitations of using 1D modelling approach which excludes the contribution of
the major system (i.e. effect of additional redundancies) in conveying surface flows to downstream parts of the system during extreme events.

In contrast to the total flood volume, cumulative link failure has a limited effect on mean nodal flood duration. This could be attributed to use of a single short duration rainfall event for the simulations as opposed to using multiple events. Similarly, this could also be attributed to limitations of using a simplified above ground flood model. By using a simplified above-ground flood model, surface flooding which occurs in the major system (i.e. overland flood pathways such as roads, paths or grass ways) during extreme events and which may also cause substantial damage to property and infrastructure is not considered, which could also lead to inaccurate estimation of the mean flood duration (e.g. Digman et al., 2014; Maksimović et al., 2009).

7.4.2 Effect of adaptation strategies

It is argued that an effective adaptation strategy should result in a downward shift (i.e. shift towards the origin) of the failure envelope of the existing system. By doing this, the failure magnitude and duration is minimised across the considered failure scenarios. The derived link failure envelopes suggest that CS strategy has a very limited effect on minimising the total flood volume, with the reduction being achieved at lower link failure levels. More so, no significant effect on flood duration is observed at all considered link failure levels. As a consequence, the CS strategy only minimally improves the residual functionality of the existing system during the considered link failure scenarios. This therefore suggests that sewer failures could significantly limit the effectiveness of adaptation strategies involving enhancement of redundancy at a single location in the UDS.

By implementing the O&M strategy, the system's ability to minimise both the resulting total flood volume and mean flood duration is increased considerably. As consequence, the O&M strategy considerably improves system residual functionality and hence resilience to cumulative link failure. Furthermore, improvements in asset management can minimise failures in UDSs in the long term. This therefore suggests that preventive asset management strategies for
example improved cleaning and maintenance practices may be more effective for resilience enhancement when compared to CS strategies (i.e. capital investment interventions aimed at increasing system redundancy in a single location), because they increase spare capacity in the links themselves and minimise future rate of propagation of structural failures in existing systems (Ten Veldhuis and Clemens, 2011; Ten Veldhuis, 2010).

In contrast to both the CS and O&M strategies, the study results suggest that the DS strategy is more effective in minimising the resulting loss of functionality at all link failure levels. This could be attributed to the effect of increased the spatial distribution of control strategies (i.e. smaller decentralised upstream storage tanks with the same total storage volume as the CS strategy) results in optimal use of the total storage volume for reduction both the storm water volume and the inflow rates before entry into UDS. Reducing the storm water inflows into the system in turn enables the degraded UDS to continue functioning with minimal impacts. It could also be due to a reduction in propagation of hydraulic failures from one part of the UDS to another, which suggests that the DS strategy improves the flexibility properties of the whole (minor) system.

When the costs of failure (direct tangible flooding costs) are taking into consideration, the results suggest that the O&M strategy (which does not require higher upfront capital investment costs) is more cost-effective over the design life of the UDS when compared to both the CS strategy. In contrast, although the DS strategy is associated with higher capital investment costs (due to the additional cost of distributed storage tanks), it is more cost effective when compared to both the CS & O&M strategies over the design life of the system.

Using this argument, it could be suggested that adaptation strategies that increase the spatial distribution of control strategies in upstream parts of the catchment for example implementation of multifunctional (dual-purpose) rainwater harvesting (DeBusk, 2013) at a city district or catchment scale are more cost effective in the long term and could significantly increase the resilience UDSs to sewer failures.
7.5 Conclusions

This research has tested and extended the global resilience analysis (GRA) method to systematically evaluate UDS system resilience to random cumulative link (sewer) failure. The GRA method presents a new and promising approach for performance evaluation of existing and adapted UDSs that shifts emphasis from prediction of the probability of occurrence of key threats that lead to flooding to evaluating the effects of a wide range of possible failure scenarios ranging from normal to unexpected with reduced computational complexity. Furthermore, the research has demonstrated that in addition to functional failures, structural failures such as sewer failure which also significantly contribute to flooding in cities can be effectively considered in resilience-based evaluation of UDSs.

In this chapter, the effect of a wide range of random and progressive sewer (link) failure scenarios on the ability of existing and adapted UDSs to minimise the resulting loss of functionality has been investigated. Link failure envelopes have been determined by computing the minimum and maximum values of the total flood volume and mean nodal flood duration results generated by simulating a large number of pseudo random cumulative link failure scenarios. The structural resilience index has been developed and used to link the resulting loss of functionality to the system’s residual functionality at each link failure level. Based on the results of the study, the following conclusions are drawn.

- The use of convergence analysis enables determination of the minimum number of pseudo random cumulative link failure sequences required to achieve consistent GRA results, which in turn enhances that practicability of resilience assessment by significantly reducing the computational complexity involved in simulating all possible sewer failure combinations.
- Building resilience in UDSs to unexpected failures necessitates explicit consideration of the contribution of different failure modes, effect of interactions between different failures modes for example interdependences between sewer failures and hydraulic overloading in UDS design or performance evaluation of existing systems.
Taking into consideration the cost of failures in resilience-based evaluation confirms that adaptation strategies that enhance system flexibility properties such as distributed storage are more cost-effective over the service life of the UDSs.

Building resilience in UDSs should not only be addressed through capital investments aimed at enhancing inherent UDS properties such as redundancy and flexibility but should also consider investments in asset management strategies such as improved sewer cleaning and maintenance of existing UDSs.

It is therefore concluded that embedding resilience in UDS design or rehabilitation provides a promising and potentially cost-effective approach to maintain acceptable flood protection service levels in cities during unexpected system failures. In addition, the research has demonstrated that increasing that spatial distribution of control strategies (i.e. decentralisation) provides a more effective strategy for enhancement of the global resilience of UDSs to sewer failures. Implementation of multifunctional rainwater harvesting (RWH) systems at a city district or catchment scale provides another promising strategy with a high degree of spatial distribution of storage while simultaneously providing water supply benefits (DeBusk, 2013; Mugume et al., 2015a). In the next chapter, the effectiveness of implementing multifunctional RWH systems at a city district or catchment scale in enhancing UDS global resilience to flooding is investigated.
8. Resilience-based evaluation of multifunctional rainwater harvesting strategies

This chapter focuses on investigating the effect of catchment scale implementation of multifunctional (dual-purpose) rainwater harvesting (RWH) strategies on enhancement of global UDS resilience to flooding and provision of alternative water supply in Nakivubo catchment in Kampala. Section 8.1 provides an introduction to the chapter. Section 8.2 describes the methodology for design and modelling of dual-purpose RWH systems at a catchment scale. The section also describes the adopted methodology for evaluating the effect of implementing RWH strategies on i) enhancing the global resilience of an UDS to cumulative link failure and ii) the improvement in water supply resilience in the case study catchment through harvesting of rainwater as an alternative to centralised (mains) water supply. The section is concluded by describing the adopted cost benefit analysis method that is applied to evaluate the net benefits resulting from implementation of the proposed RWH strategies.

Section 8.3 describes the GRA results for tested RWH strategies with respect to minimisation of the resulting loss of system functionality when subject to cumulative link failure. In sub section 8.3.4, the water supply resilience benefits of the tested RWH strategies are presented. Sub section 8.3.5 describes the cost benefit analysis results. In addition, the net benefits generated by implementing the respective RWH strategies are contrasted with corresponding results obtained using the distributed storage (DS) and improved operation and maintenance (O&M) strategies (investigated in chapter 7). The subsequent sections discuss the results (section 8.4) and present the main chapter conclusions (section 8.5).
8.1 Introduction

The main objective of this chapter is to investigate the effect of wide (catchment) scale implementation of multifunctional (dual-purpose) RWH strategies on: i) enhancement of global UDS resilience to structural failures that may result from unexpected system failures or long term asset degradation and ii) improvement of water supply resilience in the case study catchment through provision of alternative water supplies.

In chapters 6 and 7, emphasis has been placed on evaluating effectiveness of adaptation strategies that improve inbuilt UDS properties or attributes such as redundancy or flexibility during design or rehabilitation on enhancement of global UDS resilience to unexpected system failures. In contrast to a system focused view, recent studies argue that recipients (customers) of urban water services (i.e. individuals or households) can be viewed as agents (actors) in urban water management with the ability to change behavior when impacted upon by threats that lead to disruption of their preferred or acceptable service levels (e.g. Butler et al., 2014; Tyler and Moench, 2012). Consequently, it can be argued that general resilience of urban water systems (UWSs) could also be enhanced through installation of equipment that such as domestic RWH systems that not only improve whole system flexibility properties but also enhance customer preparedness for extreme events or disruptions that lead to unexpected system failures (Butler et al., 2014; Djordjević et al., 2011; Mcbain et al., 2010; Mugume et al., 2015a).

In a number of recent studies, multiple benefits of RWH systems such as reduction of stresses on existing potable water distribution systems (WDSs), provision of backup supplies in cities with insufficient water supply capacity, cost savings to customers or provision of wider water resource conservation benefits have been demonstrated (Aladenola and Adeboye, 2010; Burns et al., 2015a; Campisano and Modica, 2012; Ward et al., 2012).

More recently, a limited number of studies have demonstrated the uniqueness of dual-purpose RWH systems; that is their ability to simultaneously provide
alternative water supply and storm water control benefits in cities (Burns et al., 2015a; DeBusk, 2013; Melville-Shreeve et al., 2014; Taylor, 2013; van der Sterren et al., 2012). With respect to urban drainage, dual-purpose RWH systems can provide spatially distributed detention storage within the catchment that could potentially reduce urban flooding by minimizing storm water volumes and peak flow rates (Burns et al., 2015a; DeBusk, 2013).

Most studies have been focused on evaluating the effect of RWH systems implemented at a plot (site) scale on reduction of storm water peak flow rates and volumes (Burns et al., 2015b; Mahmoud et al., 2014; Van der Sterren et al., 2014; van der Sterren et al., 2012). In more recent studies, the effect of RWH systems on reduction of flooding resulting from functional failures in UDSs has also be investigated (Burns et al., 2015b; Kwak and Han, 2014). However, most of these studies have been carried out at site scale. Consequently, the effect of implementing such innovative strategies in building the global resilience of an UDS to flooding at a city district or catchment scale requires further investigation. Furthermore, most of these studies take a narrow view of functional resilience and do not explicitly consider the effect of potential structural failures that occur in UDSs due to short term unexpected system failures or long term asset degradation (Kellagher et al., 2009; Mugume et al., 2015b). Further research aimed at evaluating the effect of implementing innovative RWH strategies in minimising the negative effects of unexpected sewer failures such as blockages or sewer collapse or long term asset degradation in existing urban water systems is required (e.g. Barton et al., 2007).

In this chapter, the GRA method is applied to investigate the effect of a set of RWH strategies (in which dual-purpose RWH systems are implemented at a catchment scale) on enhancement of global UDS resilience to flooding when subject to a wide range of pseudo random cumulative link failure scenarios. In addition, the resilience benefits of the tested RWH strategies with respect to provision of alternative water supplies are quantified using three indicators: water saving efficiency, \( E_i \), additional volume of required mains water (top-up), \( M_t \) and the
number of days \( f_{D50} \), when \( M_t \) exceeds 50\% of the average daily demand. Finally, cost-benefit analysis of the proposed strategies is carried out and the results are compared with corresponding results obtained using distributed storage (DS) and improved UDS operation and maintenance (O&M) strategies (investigated in Chapter 7).

8.2 Methods

8.2.1 Design of RWH systems

In this research, two key objectives have guided the design of the RWH systems that is: reduction of storm water inflows into the UDS through detention of roof run-off and provision of alternative water supplies to the households. To achieve these objectives, the ‘Intermediate’ approach recommended by British Standard for RWH (BSI, 2013) is used for sizing of the RWH systems (Equation 8.1). In contrast to the ‘Simplified’ approach which uses UK-based rainfall data for RWH tank design, the ‘Intermediate’ approach allows for the use of local design rainstorm events and hence provides a more accurate and flexible methodology for sizing of the storm water control volume of RWH tanks in non UK locations.

\[
V_T = \begin{cases} 
V_{SC} + Y_R, & \text{when } D_N - 3Y_R < 0 \\
R_d \times A, & \text{when } D_N - 3Y_R > 0 
\end{cases} 
\]  

(8.1)

Where;

\[
Y_R = A \times e \times AAR \times h_f \times 0.05 
\]  

(8.2)

\[
D_N = P_d \times n_p \times 365 \times 0.05 
\]  

(8.3)

\[
V_{SC} = R_d \times A - [(D_N - Y_R) \times 0.5] 
\]  

(8.4)

Where \( V_T \) is the total storage for storm water control and alternative water supply (L); \( V_{SC} \) the additional tank volume needed for storm water control (L); \( V_D \) the water demand volume (L); \( Y_R \) 5\% of the annual rainfall yield (L); \( D_N \) 5\% of the annual water demand (L); \( R_d \) the design rainstorm event depth (mm); \( A \) the area of the collection surface \( (m^2) \); \( e \) the yield coefficient (\%); \( AAR \) the annual average
rainfall depth for the location (mm); \( h_f \) the hydraulic filter coefficient; \( P_d \) the daily water requirement per person and \( n_p \) the number of persons per household.

The methodology has been applied to the Nakivubo catchment in Kampala, Uganda, with an estimated population of 376,855 (NWSC, 2014b). The average annual rainfall for Kampala is 1,292 mm (Figure A.1) and this has been used to determine the yield, \( Y_R \) (Equation 8.2). The recently updated standard on RWH systems (BSI, 2013) recommends use of a 100 yr, 6 hr design storm for sizing of dual purpose RWH systems in the UK. However, due to the convective nature of rainfall in Kampala, direct use of a 100 yr, 6 hr design storm may lead to oversized and hence costly RWH systems. In this research, therefore, the storm water control volume, \( V_{Sc} \) of the RWH tank is designed considering a 2 year, 6 hr design storm for Kampala, with a total depth of 56.1 mm (Equation 8.4). Two RWH tank sizing scenarios are used that is (a) **Scenario 1**: Medium demand, 2 bedroomed house and (b) **Scenario 2**: High demand, 3 bedroomed house (Table 8.1).

**Table 8.1**: RWH tank design scenarios for Kampala

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Design parameters</th>
<th>Computed tank sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average household</td>
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</tr>
<tr>
<td></td>
<td>size (no. of</td>
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</tr>
<tr>
<td></td>
<td>persons)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average water</td>
<td></td>
</tr>
<tr>
<td></td>
<td>demand (l/c/d)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contributing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>roof area (m²)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Water demand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>volume, ( V_d )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Storm water control volume, ( V_{Sc} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design tank volume, ( V_T ) (m³)</td>
<td></td>
</tr>
<tr>
<td>Scenario 1</td>
<td>4.02</td>
<td>81.5</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>3.78</td>
<td>63.7</td>
</tr>
</tbody>
</table>

The per capita water demand used in this study is based on analysis of available water demand data for Kampala for the period July 2011 – June 2012 (NWSC, 2014b), in which customer water demand patterns are classified into four demand categories (Table A.5). In this study, the *medium* (63.7 l/c/d) and high (81.5 l/c/d) per capital water demand categories are used. The average household sizes used in the computing of the water demand volume, \( D_N \) range from 3.28 – 4.27 persons per household (UBOS, 2014). In this work, it is assumed that the collected rainwater can be used for potable water uses through treatment using low cost processes (Naddeo et al., 2013; WHO, 2011). It can also be used directly for non-
potable uses such as toilet flushing, washing or urban gardening purposes, without the need for treatment (Aladenola and Adeboye, 2010; Awuah et al., 2014).

### 8.2.2 Modelling of single RWH tank units

The single (unit) RWH tanks designed in 8.2.1 above are modelled in SWMM v5.1, using the ‘rain barrel’ option in the ‘Low Impact Development’ (LID) control editor. The single units are replicated across all the sub-catchments in the case study area effectively displacing an equal amount of non-LID area from each sub-catchment (Rossman, 2010). This modelling approach distributes the total installed RWH storage volume proportionately across the catchment and hence represents a decentralized adaptation option implemented at a large (catchment or city district) scale.

In this research, to simplify the modelling and subsequent GRA simulations, it is assumed that each RWH tank is emptied before the onset of an extreme rainfall event. This could be achieved in practice through implementing passive or active control systems for regulation of the flow of stored rainwater into the urban drainage system. In passive control systems, flow regulation is achieved through temporary capture and release storm water between subsequent rainfall events (DeBusk, 2013; Herrmann and Schmida, 2000; Melville-Shreeve et al., 2014). It could also be achieved through the effect of increased household demand patterns (for example resulting from connecting toilet flushing, laundry and outdoor use devices to the RWH tank) that consequently increase available total storage volume between storm events (DeBusk, 2013).

It is further noted that the assumption that RWH tanks are emptied before the onset of an extreme rainfall event enables simulation of the maximum possible reduction of resulting flooding impacts (i.e. maximum possible resilience enhancement benefits). However, emptying of the entire RWH tank volume conflicts with the second objective of multifunctional RWH systems, that is provision of alternative water supplies in the case study catchment. In subsection 8.3.4, it is demonstrated that a minimum volume of rainwater that can provide at least 35 l/c/d should be maintained (stored) in the RWH tanks to generate
maximum water supply resilience benefits. Figure 8.1 provides an example of multifunctional RWH system.

![Figure 8.1: Example of a multifunctional RWH configuration with an orifice type control for passive release of storm water.](image)

- $A$ is the contributing roof (plan) area, 
- $V_{SC}$ the storm water control volume, 
- $V_D$ the water demand volume, 
- $Q_t$ the collected rainwater runoff during the time interval $t$ and 
- $Y_t$ the tank yield during a time interval $t$, 
- $D_t$, the demand during time interval, 
- $M_t$ the mains supply top-up during time interval, $t$. (Adapted from Fewkes, 2000 and Herrmann and Schmida, 2000)

### 8.2.3 Catchment scale modelling of RWH strategies

The modelled RWH tank units are replicated across all sub catchments in the study area to represent a multifunctional adaptation strategy aimed at enhancing global UDS resilience to flooding while simultaneously providing alternative water supply benefits. For the catchment scale modelling, three RWH adaptation options are proposed for further investigation (Table 8.2).
Table 8.2: RWH strategy options

<table>
<thead>
<tr>
<th>Option</th>
<th>Single RWH tank size (m³)</th>
<th>No. of units</th>
<th>Total storage volume (m³)</th>
<th>Contributing roof area (%)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1 (base case)</td>
<td>11</td>
<td>28,636</td>
<td>315,000</td>
<td>20</td>
<td>Same total storage volume, $V_{TSV}$ as the CS or DS strategies</td>
</tr>
<tr>
<td>Option 2</td>
<td>9</td>
<td>26,250</td>
<td>236,250</td>
<td>15</td>
<td>0.75$V_{TSV}$ (25% reduction)</td>
</tr>
<tr>
<td>Option 3</td>
<td>9</td>
<td>17,500</td>
<td>157,500</td>
<td>10</td>
<td>0.50$V_{TSV}$ (50% reduction)</td>
</tr>
</tbody>
</table>

In Option 1, a total of 28,636 units with a total storage volume, $V_{TSV}$ of 315,000 m³ are represented in the SWMM model. In option 1, the modelled total RWH tank storage volume is the same as that of the CS and DS strategies investigated in chapter 7. The modelled RWH tanks collect runoff from a combined contributing roof area of 228.7 ha which represents 20% of the total roof area in the catchment (i.e. 1 in 5 houses is installed in a RWH tank). To investigate if comparable performance could be achieved with smaller total installed RWH tank volume (with higher levels of spatial distribution of control strategies when compared CS and DS strategies investigated in Chapter 7), RWH Options 2 and 3, in which $V_{TSV}$ is reduced by 25% and 50% respectively are modelled. The details of the catchment scale RWH model parameters are provided in Table A.6.

8.2.4 Global resilience to structural failures

The GRA method is applied to evaluate the effect of the proposed RWH adaptation strategies on enhancing the ability of the UDS to minimise the resulting loss of system functionality when subject to a wide range of random structural failure scenarios involving cumulative link failure. In this study, a combined total of 49,200 cumulative link failure scenarios generated from 600 random link failure sequences is simulated. The minimum, mean and maximum values of all model solutions obtained at each considered link failure level are computed using total flood volume and mean duration of nodal flooding system performance indicators. In addition, resilience envelopes are derived by computing the minimum and maximum values of the resilience index, $Res_o$ at each link failure level for each of the tested RWHs strategies.
8.2.5 Effect on improvement of water supply resilience

Resilience assessment is carried out to evaluate the performance of the proposed RWH strategies (Option 1) with respect to water savings and reduction of the magnitude and duration of centralized (mains) water use in the case study area due to volumetric failure of the proposed RWH strategies. Volumetric failure of RWH systems may be result from inadequate sizing of storage tanks, extended periods of low or no rainfall or high demands connected to the system (Taylor, 2013; Wang and Blackmore, 2012). Water balance modelling is carried out using 18 year (1991-2009) average daily rainfall data set for Makerere University rain gauge station (Figure A.2). The performance of the RWH system is investigated using the ‘Yield After Spill’ (YAS) operating rule (Fewkes, 2000; Jenkins et al., 1978).

The YAS operating rule assigns the RWH tank yield, \( Y_t \) as the minimum value of either the volume of rainwater in storage from the preceding time interval or the demand in the current time interval (Equation 8.5). To obtain the final volume of water stored in the RWH tank, \( V_t \), the rainwater runoff in the current time interval is then added to the volume of rainwater in storage tank from the preceding time interval (with any excess spilling via the overflow) and then subtracting the tank yield (Equation 8.6). The YAS algorithm is preferred because it gives a conservative estimate of system performance irrespective of the model time interval (Fewkes, 2000).

\[
Y_t = \min \left\{ \frac{D_t}{V_{t-1}} \right\}
\]

(8.5)

\[
V_t = \min \left\{ V_{t-1} + Q_t - Y_t, \frac{S - Y_t}{S} \right\}
\]

(8.6)

Where \( S \) is the RWH tank storage capacity and \( V_t \) the volume of collected rainwater during time interval \( t \).

The water saving efficiency \( E_t \), (i.e. volumetric reliability) is computed as the ratio of the demand, \( D_t \) to the yield \( Y_t \) using Equation 8.7 (Ward et al., 2012). To assess
the resilience of the proposed strategies with respect to minimization of volumetric failure magnitude and duration, two performance metrics are investigated at a daily time step for a range of water demand scenarios. The mains top-up, \( M_t \) provides a measure of the failure magnitude over the considered modelling period (Equation 8.8). The number of days in a year when mains top-up, \( M_t \) exceeds 50% of the average daily demand \( f_{D50} \) provides a measure of the RWH system failure duration (Equation 8.9).

\[
E_t = \frac{Y_t}{D_t} \times 100 \quad (8.7)
\]

\[
M_t = D_t - Y_t \quad (8.8)
\]

\[
f_{D50} = \text{no. of days } M_t > 0.5D_t \quad (8.9)
\]

### 8.2.6 Cost benefit analysis

Cost benefit analysis is carried out to investigate the cost effectiveness of the proposed RWH strategies over the design life of the UDS. In recent work, cost estimates for a range of RWH systems in Uganda have been assessed based on available evidence in the Uganda (Parker et al., 2013). The results of the study suggest that average unit tank cost (US$/m\textsuperscript{3}) for tank sizes ranging from 5 – 10 m\textsuperscript{3} is 150 - 250 US$/m\textsuperscript{3} (96 – 160 £/m\textsuperscript{3} considering the 2013 average exchange rate of 1GBP = 0.64 USD). Based on this cost data, an average unit RWH cost of £128/m\textsuperscript{3} is used for the cost assessment. O & M costs for domestic RWH tanks (without pumping costs) are considerable lower when considered to other Sustainable Drainage System (SUDs) options (Environmental Agency, 2015; Parker et al., 2013). In a recent study, O&M costs for domestic RWH systems that range from 0.1 – 0.6 £/m\textsuperscript{2} of contributing roof area are reported (Environmental Agency, 2015). In this study, an average annual O&M cost estimate of 0.35 £/m\textsuperscript{2} is applied (Table 8.3).
### Table 8.3: Unit capital and O&M costs for RWH tanks

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Contributing roof area (m²)</th>
<th>Capital costs</th>
<th>Annual O&amp;M costs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Unit cost (£/m³)</td>
<td>Capital Cost (£/tank)</td>
</tr>
<tr>
<td>Scenario 1 (11 m³ tank)</td>
<td>99.8</td>
<td>128</td>
<td>1,408</td>
</tr>
<tr>
<td>Scenario 2 (9 m³ tank)</td>
<td>79.8</td>
<td>128</td>
<td>1,152</td>
</tr>
</tbody>
</table>

In addition, it is noted that RWH systems have relatively shorter operational life spans of between (15 – 25 years) when compared to the life span of conventional UDSs. In this work, it is assumed that RWH tanks are renewed (capital re-investment) after 25 years. To aid comparison with the DS and O&M strategies, discounted total cost analysis is carried out for the proposed RWH strategies considering a service life of 50 years.

### 8.3 Results

#### 8.3.1 Initial state system performance assessment

Model simulations are carried out for each of the proposed RWH adaptation options in their respective initial states (i.e. before the UDS is subjected to sewer failure). The global performance of the whole (minor) system is quantified (Table 8.4). Implementing Option 1 reduces the total flood volume, mean flood duration and the number of flooded nodes by 27.1%, 17.7% and 14% respectively. Implementing option 2 leads to a modest reduction in the total flood volume and mean flood duration of 17.7% and 13.9%. However, the strategy has a minimal effect on the number of flooded nodes (7%). Implementing option 3 results in minimal reduction of the total flood volume and mean flood duration of 9.5% and 9.1% respectively.
Table 8.4: Initial system state performance for the existing UDS and for the tested RWH adaptation strategies

<table>
<thead>
<tr>
<th>Adaptation strategy</th>
<th>Flood volume (Value x 10^3 m^3)</th>
<th>% change</th>
<th>Flood duration (Value hrs)</th>
<th>% change</th>
<th>Flooded nodes (Value no.)</th>
<th>% change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing UDS</td>
<td>706.0</td>
<td></td>
<td>0.69</td>
<td></td>
<td>57</td>
<td></td>
</tr>
<tr>
<td>RWH Option 1</td>
<td>514.6</td>
<td>27.1%</td>
<td>0.57</td>
<td>17.0%</td>
<td>49</td>
<td>14.0%</td>
</tr>
<tr>
<td>RWH Option 2</td>
<td>580.7</td>
<td>17.7%</td>
<td>0.59</td>
<td>13.9%</td>
<td>53</td>
<td>7.0%</td>
</tr>
<tr>
<td>RWH Option 3</td>
<td>639.0</td>
<td>9.5%</td>
<td>0.63</td>
<td>9.1%</td>
<td>56</td>
<td>1.8%</td>
</tr>
</tbody>
</table>

8.3.2 GRA results

GRA results for option 1 are compared with those obtained by simulating the performance of the existing UDS in which no RWH adaptation strategy has been implemented. The effect of implementing option 1 is a considerable reduction in the total flood volume that ranges for 19.7 – 20.5%, and which at all considered link failure levels (Figure 8.2a). The strategy also leads to a modest reduction in the mean nodal flooding (11.6 -16.6%), with the reduction being achieved at all link failure levels (Figure 8.2b).

Implementing option 2, results in slightly lower reduction in the total flood volume (14.6 – 17.7%) and the mean flood duration (11.7 -13.8%), which is occurs at all link failure levels. In contrast, implementing option 3, leads to the lowest reduction in both the total flood volume and mean nodal flood duration of 9.4 – 10.0% and 8.7 – 10.2% respectively. The results also indicate that for all tested RWH strategies, any progressive increases in link failure levels beyond 20% lead to minimal increase in the total flood volume.
Figure 8.2: Results of GRA for the existing UDS and for RWH adaptation Options 1, 2 and 3 showing the effect of cumulative link failure on (a) total flood volume and (b) mean nodal flood duration

8.3.3 Resilience envelopes

The resulting resilience envelopes are presented in Figure 8.3 for the existing UDS and for all the tested RWH adaptation strategies. To facilitate comparison of the performance of the tested strategies, a resilience threshold of 0.7 is plotted on each of the graphs. The figure shows that implementing Option 1 leads to a considerable improvement in computed $Res_o$ values that ranges from 17.3 – 24.9% when compared to the existing system. In addition, the computed mean $Res_o$ values are higher than the threshold until 63% of the links have failed. Options 2 results in a slightly lower increase in $Res_o$ (17.7 – 20.0%) when compared to Option 1. In addition, the computed $Res_o$ values for Option 2 fall below rapidly fall below the resilience threshold after random failure of 53% of the links in the UDS. In contrast, Option 3 leads a slight increase in $Res_o$ that ranges from 9.7 – 14.9%. Similar to Option 2, the computed $Res_o$ values rapidly fall below the threshold value with increasing link failure levels.
Figure 8.3: Resilience envelopes showing maximum (upper dotted line), mean (solid line), minimum (lower dashed line) of the computed values of $Res_o$ for the existing UDS (a) and for the tested RWH strategies i.e. option 1 (base case with same total storage volume, $V_T$ as the CS and DS strategies (b), option 2 ($0.75V_T$) and (c), option 3 ($0.50V_T$). The plots also show the $Res_o$ values relative to an assumed threshold of 0.7 (dashed horizontal line).
8.3.4 Water supply resilience enhancement benefits

The study results indicate that RWH Option 1 results in a considerable water saving efficiency, \( E_t \) of 29.3%. RWH Options 2 and 3 result in slightly lower but significant \( E_t \) values of 22.7 and 15.1% respectively.

Table 8.5: Water saving efficiency considering connected RWH water demand of 63.7 l/c/d

<table>
<thead>
<tr>
<th>RWH Option</th>
<th>Water demand, ( D_t ) (m(^3)/d)</th>
<th>Tank yield, ( Y_t ) (m(^3)/d)</th>
<th>Water saving efficiency, ( E_t )</th>
<th>Mains top-up ratio (-)</th>
<th>No of days ( M_t &gt; 0.5D_n ) ( f_{D50} ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>30,714</td>
<td>8,990</td>
<td>29.3</td>
<td>70.3</td>
<td>90.7</td>
</tr>
<tr>
<td>Option 2</td>
<td>24,004</td>
<td>5,444</td>
<td>22.7</td>
<td>77.3</td>
<td>92.9</td>
</tr>
<tr>
<td>Option 3</td>
<td>24,004</td>
<td>3,629</td>
<td>15.1</td>
<td>84.9</td>
<td>98.6</td>
</tr>
</tbody>
</table>

The study results also indicate that for a constant total installed RWH tank capacity and contributing roof area, reducing the water demand volumes connected to the RWH tanks to less than 35 l/c/d significantly increases \( E_t \) and consequently reduces the required volume of mains water use, \( M_t \) and the number of days when the required mains water use is greater than 50% of the daily demand, \( f_{D50} \) (Figure 8.4).

Figure 8.4: Water supply resilience analysis results for RWH Option 1 showing the effect of changes in connected (RWH tank) water demand levels on required mains top up volume, \( M_t \) and number of days when the mains top up exceeds 50% of the daily water demand, \( f_{D50} \).
8.3.5 Cost benefit analysis results

Figure 8.5 shows the results of the discounted cost calculations for the capital and O&M costs over a 50 year service life the UDS. For comparison purposes, the results of the DS and O&M strategies (investigated in chapter 7) are also plotted in Figure 8.5. The results indicate that RWH option 1 is associated higher discounted capital (23%) and O&M costs (106%) when compared the DS strategy. RWH option 2 has lower discounted capital (5%) but is associated with higher O&M costs (54%) when compared to the DS strategy. RWH option 3 is associated significantly lower discounted capital costs (33%). However, the strategy leads a slight increase in the O&M costs (7%), when compared to the DS strategy.

![Discounted capital and O&M costs for tested RWH adaptation options and for the DS and O&M strategies](image)

**Figure 8.5**: Discounted capital and O&M costs for tested RWH adaptation options and for the DS and O&M strategies

In order to evaluate the resilience benefits of each tested strategy, discounted total costs are computed using Equation 3.8. The net benefit of each strategy over the service life of the UDS is computed using 3.13. The computed mean values of the resilience index, $\text{Res}_o$ are plotted against the net benefits (Figure 8.6). Similarly, for comparison purposes, the results are plotted together with those obtained for the CS, DS and O&M strategies in chapter 7.

The results indicate that RWH Option 3, results in net benefits that range from 18.3 – 24.3%. The results are comparable to those obtained by the O&M and DS
strategies at link failure levels less than 60%. In contrast, RWH Option 2 and 3 results in net benefits that range from 13.7 – 15.7% and 9.1 – 11.3% respectively.

![Figure 8.6: Plot of the resilience index against the computed percentage net benefits for all tested RWH adaptation options including the CS, DS and O&M adaptation strategies. The values are computed at various link failure levels, ranging from random single link \((N-1)\) to random failure of multiple links, \(N - i: i = 2, 3, 4, 8, 16, 24, 32, 40, 48, 57, 65, 73 & 81\)](image)

### 8.4 Discussion of results

#### 8.4.1 Effect on global UDS resilience to flooding

In RWH Option 1, RWH tanks are installed for every 1 in 5 properties in the catchment area. The strategy leads to a significant reduction of the total flood volume during the considered link failure scenarios when compared to the existing UDS. However, the strategy is slightly less effective when compared to both the DS and O&M strategies when link failure levels exceed 60%. This could be attributed to the fact the RWH tanks may over flow during extreme events. The results could also suggest that there is an optimum storage tank capacity and
distribution of storage controls that result in the highest reduction of the flooding magnitude and duration during extreme events.

Considering RWH Option 2, the total storage volume is reduced by 25% (approx. 1 in 7 properties is installed with a 9m$^3$ RWH tank). Despite the reduction in the total storage volume, the strategy is still more effective in maintaining higher global system residual functionality and also results in higher net benefits when compared to the CS strategy. Implementing Option 3 reduces the total storage volume by 50% (1 in every 10 properties is installed with a RWH tank). However, the strategy is relatively less effective in maintaining higher global system residual functionality when link failure levels exceed 20%.

The study results suggest that use of household RWH systems solely for enhancement of flood resilience may be less cost effective when compared to use of relatively larger sub-catchment scale distributed storage tanks or investments in improved sewer asset management when the UDS structure is severely degraded (e.g. Kwak and Han, 2014; Mugume et al., 2015b; Ten Veldhuis and Clemens, 2011). These results suggest that the spatial scale of control strategies may be crucial for achieving the most optimal improvements in global resilience to flooding. Based on the study, it is suggested that there exists an optimal spatial scale (i.e. distribution of RWH units) and size of storage tanks that maximises the resulting improvement in global UDS resilience to unexpected sewer failures while minimising upfront capital costs.

8.4.2 Effect on enhancement of water supply resilience

It should also be noted that although all the tested RWH strategies are relatively less effective when compared to both DS and improved O&M strategies for a flood resilience perspective, their ability to provide alternative and renewable water supply in cities is very important particularly in most developing country cities where centralised water supply and distribution system resilience to unexpected system failures such as pipe, pump or interconnected electrical power system failures is considerably low (e.g. Aladenola and Adeboye, 2010; Yazdani et al., 2011).
Based on this premise, it is argued that large scale implementation of RWH systems improves the resilience of water service provision to individual households through enhancing user flexibility (ability to switch from mains water to rainwater for example unexpected WDS failures) and preparedness for exceptional failures by providing back-up water storage to augment water supplies until centralised mains water services are restored. It is recommended that for future work, the presented approach could applied in integrated modelling approaches (e.g. Urich and Rauch, 2014) to investigate the effect of catchment scale RWH strategies on enhancement of global resilience to unexpected system failures in both UDSs (i.e. flooding) and WDSs (e.g. electric power failure or leakages in WDSs).

8.4.3 Cost effectiveness of RWH strategies

The results of the study suggest that all the tested RWH strategies provides are more cost effective when compared to use of large centralised storage tanks (CS strategy). However, the results also suggest that RWH strategies may be less cost effective from a flood resilience perspective when compared to DS and improved O&M strategies when the UDS’s structure is severely degraded (i.e. link failure levels > 60%). This could be explained by the emphasis placed on quantifying the net benefits resulting from the tested RWH strategies from the perspective of minimising the magnitude and duration of flooding so as to aid comparison with the other adaptation strategies i.e. distributed storage and improved O&M strategies.

Although outside the scope of this study, integrated analysis of RWH strategies that includes other resilience benefits of RWH systems such as provision of emergency water supplies during unexpected WDS failures, reduction of WDS pumping costs and enhancement of global WDS resilience could further improve cost effectiveness of urban catchment scale RWH strategies and thus contribute to more sustainable water management in cities. This could be operationalised in practice by developing new policies mandating installation of RWH systems on all new builds or during retrofit of large scale commercial or institutional buildings (e.g. Mahmoud et al., 2014; Ward et al., 2012).
8.5 Conclusions
In this study, the effect of implementing multifunctional (dual-purpose) RWH adaptation strategies on enhancement of structural resilience of an existing UDS with respect to flooding and improvement of water supply resilience in the Nakivubo catchment in Kampala has been investigated. Based on the results of the study, the following conclusions are drawn:

- Wide scale implementation of dual-purpose RWH systems in the case study catchment enhances the global resilience of the UDS to cumulative link failure by up to 25%. This is attributed to the increased spatial distribution of storage volumes within the catchment and the use of rainwater (which would have otherwise been directly discharged to storm water systems) for household purposes.

- Implementing RWH systems is slightly less effective for enhancing the global resilience of UDSs to structural failures when compared use of larger distributed storage (DS) strategies or investments in improved sewer asset management, when the UDS structure is severely degraded.

- Although relatively less effective when compared to DS and improved asset management strategies from a flood resilience perspective, catchment scale implementation of RWH systems provide a more cost effective strategy in the long term in respect to enhancement of resilience to both flooding and unexpected water supply system failures particularly in tropical developing cities where annual rainfall is relatively evenly distributed and where water supply system resilience to unexpected failures is considerably low.

- Taking a holistic and integrated view of the whole urban water cycle in new resilience based evaluation approaches provides more complete view of resilience and long term sustainability benefits resulting from large scale implementation of multifunctional RWH systems in future cities.
Chapter Nine

9. Conclusions and recommendations for future work

This chapter presents the thesis conclusions, research contribution to the field and recommendations for future work. Section 9.1 presents a concise summary of the thesis. Section 9.2 presents the main conclusions drawn from the undertaken research, while section 9.3 discusses and synthesizes the main research contributions to the field. In section 9.4, the main recommendations for practice and future research are presented.

9.1 Thesis summary

The need to develop more resilient urban drainage and flood management systems in cities is now widely recognised as key to maintaining acceptable flood protection service levels during not only normal operating conditions but also in the event of unexpected (exceptional) loading conditions that lead to system failure. Conventional reliability-based design and rehabilitation approaches tend to focus on accurate quantification of the probability of occurrence of key threats such as occurrence of an extreme rainfall that may result from climate change or variability and minimising the probability of occurrence of resulting hydraulic failures. However, urban flooding incidences may result from other causes that include episodic system failures such as sewer collapse, blockage, pump or sensor failure and long term asset degradation. Consequently, new evaluation approaches that enable: (i) consideration of ‘all possible threats’ or ‘combinations of threats’ including existing network capacity and system failures and (ii) explicit consideration of interactions between threats, system performance (structure and function) and resulting failure impact magnitude and duration are required. This research therefore set out to address the following aim and objectives:
9.1.1 Research aim and objectives

Research aim:
To investigate, develop and apply the Global Resilience Analysis (GRA) approach to systematically evaluate the resilience of urban drainage systems to exceptional (unexpected) threats.

Specific objectives:

- To investigate and characterise potential failure modes that lead to pluvial or urban drainage system flooding
- To evaluate the effect of a large number and range of both functional and structural failure scenarios on UDS performance
- To develop a new resilience index that quantifies system residual functionality as a function of failure magnitude and duration
- To model and evaluate the effect of implementing potential adaptation strategies on enhancement of resilience in UDSs.
- To develop a methodology that embeds the cost of failure in cost-benefit analysis of resilience enhancement (adaptation) strategies

The key research questions that formed the basis for the investigation carried out to address the aim and objectives of the research (and the chapters where the questions are investigated) include:

a) How can the concept of resilience be defined in a clear, consistent and meaningful way? (Chapter 3)
b) What is the scope of resilience assessment? (Chapters 3 & 4)
c) Which performance indicators or metrics are most suitable for quantifying global UDS resilience to flooding? (Chapter 3)
d) How can functional and structural failures in UDSs be effectively characterized and modelled? (Chapters 3, 5, 6 & 7)
e) What is the effect of improving redundancy and flexibility properties of a given UDS (achieved through implementing various adaptation strategies)
on enhancement of its global resilience to unexpected system failures? *(Chapters 6, 7 & 8)*

f) When the cost of failure is included in the analysis, how cost-effective are the proposed adaptation strategies over the system’s service life? *(Chapters 3, 6, 7 & 8).*

The research, that was carried out to address these underpinning research questions is summarised under three sub sections.

### 9.1.2 Key definitions and resilience enhancement strategies

In this research, resilience has been interpreted as an emergent property of a system that enables it to withstand service failure or recover from failure once it occurs. Resilience was formally defined in this work as *“the degree to which the system minimises level of service failure magnitude and duration over its design life when subject to exceptional conditions”* (Butler et al., 2014). The term *“exceptional conditions”* was used to refer to uncertain threats or disturbances that lead to system failure for example occurrence of an extreme rainfall event, sewer collapse or blockage. Resilience was further classified into two categories that is *general* and *specified resilience*. *General resilience* refers to the state of the system that enables it to limit failure duration and magnitude to *any threat* while *specified resilience* refers to the agreed performance of the system in limiting failure magnitude and duration to a *given threat* (Butler et al., 2014; Hassler and Kohler, 2014; Scholz et al., 2011).

It was postulated that a given system exhibits inherent properties or attributes such as *flexibility* and *redundancy* which can be altered/influenced in order to enhance its behaviour or response to a given threat or failure scenario (Hassler and Kohler, 2014; Mugume et al., 2015a). Flexibility can be improved through implementing intentional one-off or phased interventions (i.e. adaptation strategies) that enhance vital system properties such as buffer capacity (head room) or flatness (reduced system hierarchy) or through ensuring that more resources (e.g. trained repair crews, emergency supplies) are readily available at any given time to facilitate rapid response to unexpected failure events (Butler et al., 2014; Hassler and
Kohler, 2014; Lansey, 2012; Mugume et al., 2015a; Watt and Craig, 1986; Wildavsky, 1988). In contrast, redundancy can be enhanced by introducing multiple components providing similar functions for example storage tanks or parallel pipes, in order to minimize failure propagation through the system or to enable operations to be diverted to alternative parts of the system during exceptional loading conditions (Ahern, 2011; Cabinet Office, 2011; Mugume et al., 2015a; NIAC, 2009).

It was further argued that reliability (which focuses on prevention of failure) and resilience are interrelated with the latter building on the former. It was also postulated that resilience contributes to the long term sustainability of a given system through enhancing vital system properties such as recovery, renewability and innovation (Park et al., 2013; Seager, 2008).

However, it was identified that the effect of implementing adaptation strategies aimed at improving a given system’s flexibility or redundancy properties on improvement of UDS resilience to unexpected system failures that lead to flooding was largely unknown and thus provided a sound justification for this research.

9.1.3 Scope of resilience assessment

In this research, it was identified that conventional hydraulic reliability-based approaches which focus on investigating UDS when subject to functional failures that result from external threats such as extreme rainfall or increased dry weather flows fail to explore the full scenario space of potential failures that lead to the same failed state (i.e. urban flooding). Based on this argument, the scope of resilience assessment was extended to cover internal threats that is; structural failures that include sewer failure (collapse, blockages or bed load sediment deposition) or equipment malfunction (Mugume et al., 2015b).

Furthermore, emphasis was placed on investigating global UDS resilience to a wide range of both functional (extreme rainfall) and structural (sewer) failure scenarios. Two key failure modes were investigated that is: non-failure (initial state), and complete failure. In addition, a third failure mode that is partial failure was applied to represent the effect of improved operations and maintenance in
chapter 7. In respect to evaluation of system resilience to sewer failures, the developed GRA method was initially tested using a small synthetic UDS draining a small catchment area (22.5 ha). It was thereafter extended to investigate the global resilience of the Nakivubo UDS, that drains a large urbanised catchment (2,793 ha) in Kampala, Uganda.

9.1.4 Middle State-based Global Resilience Analysis

A new and computationally efficient GRA approach was developed and applied to that systematically evaluate ability of existing UDSs to minimise the magnitude and duration of flooding when subject to a wide range of both functional and structural failure scenarios.

The developed GRA method was applied to investigate the effect of implementing a set of adaptation strategies on improvement of global UDS resilience to a wide range of random structural failures (i.e. structural resilience). The set of strategies that were investigated included:

- **CS strategy:** Introduction of a large centralised detention pond;
- **DS strategy:** Use of spatially distributed storage tanks;
- **O&M strategy:** Improved system operation and maintenance and
- **RWH strategy:** Catchment scale implementation of dual-purpose rainwater harvesting systems
9.2 Conclusions

9.2.1 Main conclusions

The main conclusions drawn from the research include:

- The developed GRA method provides a systematic approach that has enabled evaluation of whole system resilience, where resilience concerns ‘beyond failure’ magnitude and duration.
- System resilience can be assessed without needing to know (or quantify) the probability of the cause (threat) of the impact.
- The developed GRA method allows specified resilience to be derived for various failure states (functional and structural) potentially allowing a picture of general resilience to be built up.
- The developed resilience index which is used to link the resulting loss of functionality magnitude and duration to residual functionality effectively estimates system ‘headroom’.
- The developed methodology provides a way of quantifying the impact of interventions (adaptation or resilience enhancement strategies) on system resilience either absolutely, relatively or against agreed standards.
- Embedding the cost of failure in resilience-based evaluation confirms that adaptation strategies which enhance system flexibility properties such as distributed storage are more cost-effective over the service life of the UDSs.

9.2.2 Conclusions specific to the Kampala case study

9.2.2.1 Global resilience to functional failures

- Occurrence of short duration high intensity rainfall events with higher return periods significantly reduces the residual functionality (available hydraulic capacity) of the existing UDS when compared to corresponding lower intensity events with similar magnitudes.
- Globally, the existing UDS exhibits low levels of resilience to extreme rainfall that may result from climate change or variability.
• The developed GRA method has enabled the functional resilience of the existing UDS to be more realistically characterised through evaluation of its performance when subject to wide range of spatially distributed extreme rainfall inputs. It can therefore minimise potentially erroneous and costly adaptation decisions by ensuring more accurate design (sizing) of resilience enhancement strategies such as distributed storage or dual-purpose RWH systems.

• Because the short duration events lead to higher loss of functionality magnitude but less effect of duration, it is suggested that implementation of multifunctional infrastructure for example intentional design of specific road network sections (major system) to enable safe conveyance of exceedance flows during extreme rainfall events could significantly enhance system resilience to functional failures in Kampala, while minimising negative consequences such as property damage, urban pollution incidents and potential disruptions to traffic in the city.

9.2.2.2 Global resilience to structural failures

• For the existing Nakivubo UDS:
  o Random failure of 10% of links leads to disproportionately high increase in flood volume (91%) and a minimal effect on flood duration (6% increase).
  o Globally the system exhibits low levels of resilience to cumulative link failure, which is attributed to insufficient existing hydraulic capacity and the effect of dendritic urban drainage network topology
  o Occurrence of sewer failures leads to more significant reduction of the global resilience of the UDS when compared to occurrence of extreme rainfall.

• For the adapted UDSs, it is concluded that:
  o Introduction of large centralised storage tanks (CS strategy) is ineffective in minimising the resulting flood magnitude and duration.
Globally, the CS Strategy minimally improves global resilience to cumulative link failure (i.e. by 1.2 – 2.3%).

- Improved system operation and maintenance (O&M strategy) results in a considerable reduction in resulting flood volume and duration of 21% and 12% respectively with the reductions being achieved at all link failure levels. Globally, the O&M strategy is more effective when compared to the CS strategy and results in a considerable improvement of resilience to cumulative link failure of 18.4 – 21.7%.

- Increasing the spatial distribution of storage controls (DS strategy) reduces the resulting flood volume and duration by 32% and 27% respectively. Globally, the DS strategy is the most effective in enhancing global resilience to cumulative link failure (27.5 – 41.4%) when compared both CS and O&M strategies.

- Wide scale implementation of dual-purpose RWH systems enhances the global resilience of the UDS to cumulative link failure by up to 25%, suggesting that the strategy is less effective when compared to use of larger distributed storage tanks (DS strategy) or investments in improved sewer asset management (O&M strategy) from a flood management perspective.

### 9.2.2.3 Cost effectiveness of adaptation strategies

- Investments in asset management strategies for example through improved sewer cleaning and maintenance provide a more cost effective strategy over the service life of the UDS when compared to both CS and RWH strategies which require higher upfront capital investment costs.

- Catchment scale implementation of household RWH systems solely for enhancement of flood resilience is slightly less cost effective when compared both the DS and O&M strategies, especially when the UDS structure is severely degraded (for link failure levels > 60%). Therefore, it is concluded that in future cities, implementing RWH strategies in combination with improved asset management could provide a more cost effective
strategy for enhancing the resilience of both UDSs and water supply systems to unexpected failures.

- Taking a holistic and integrated view of the whole urban water cycle during evaluation of cost-effectiveness of multifunctional strategies such as dual-purpose RWH systems provides a more complete view of their resilience and long term sustainability benefits.

### 9.3 Research contributions

The undertaken research has contributed to the development of new quantitative resilience-based evaluation frameworks and techniques that are required to operationalise resilience concepts in urban drainage and flood management in future cities. The developed GRA approach can be used by regulatory authorities, water utilities and local councils during medium to long term planning and prioritisation of resilience investments in capital projects or improvements in asset management. In summary, the main research contributions include:

- Development of the GRA method which has enabled the contribution of a wide range of threats that not only include functional failures but also structural failures such as sewer collapse, blockage or equipment malfunction to be explicitly considered in resilience-based evaluation of UDSs.

- Development of a convergence analysis technique which has enhanced practicability of resilience evaluation of city scale drainage networks consisting of a large number of components by significantly reducing the computational complexity involved in simulating ‘all possible’ failure combinations. The developed approach can also be extended to reduce computational complexity in resilience-based evaluation of other critical infrastructure such as water distribution, transportation or electrical power systems.

- Demonstration that by using physically based modelling approaches (as opposed to the use of generic methods such as surrogate models or graph theory for modelling of technical infrastructure), interactions between system
Conclusions and Recommendations

structure and function during failure can be realistically and effectively modelled, thus enabling more reliable (accurate) characterisation of engineering system resilience

- Development of an approach that enables inclusion of the cost of failure (penalty costs) in evaluation of cost-effectiveness of resilience enhancement strategies

9.4 Recommendations

The research undertaken has opened up new opportunities for holistic and systematic evaluation of the effect of a wide range of threats that have not been considered in conventional hydraulic reliability based urban drainage design and rehabilitation approaches. The following recommendations are put forward for operationalising resilience in urban drainage and flood management.

9.4.1 Recommendations for practice

- Rapid diagnosis of UDS resilience to extreme rainfall should be undertaken using block rainfall events derived from existing IDF curves or observed extreme rainfall events (that have previously caused significant flooding in a given city), particularly in cities which lack high spatial-temporal resolution extreme rainfall data sets.
- Development of new resilience-based standards should be pursued by water utility regulators at all levels (national, regional or local). The new resilience guidelines should specify that resilience to both functional and structural failures should be tested against agreed resilience standards before approval of new urban drainage infrastructure.
- To mandate testing of global UDS resilience to single link failures ($N-1$ resilience analysis) during planning and design of new UDSs or during rehabilitation and expansion of existing ones.
- In so far as rainfall and local conditions permit, resilience should be straightforwardly operationalised in practice by developing new policies mandating installation of multifunctional strategies such as dual purpose RWH systems.
or green infrastructure on all new builds or during retrofit of large scale commercial or institutional buildings and parking lots in cities.

- To achieve more accurate assessment of trade-offs between costs and resulting improvements in system resilience, the cost of failure to achieve the minimum acceptable customer service levels (i.e. penalty costs) over the service life of a given system should be embedded in new cost-benefit analysis techniques.

### 9.4.2 Recommendations for further research

The completion of a time bound research project always exposes new problems and potential areas for further research. Indeed, a number of areas that require further research were identified both within the field of urban drainage and flood management but also other related fields such as water supply and distribution systems and critical infrastructure resilience in general. The following specific areas are recommended for further research.

- Investigation of the influence of inherent/inbuilt UDS characteristics for example network structure, network size (number of links), pipe diameters, pipe gradients on general resilience in UDSs.
- Investigation of interactions and influence of prevailing solid waste management practices in Kampala on urban flood resilience.
- Investigation of the effect of other types of component failures (e.g. pump or power failures) on global resilience in UDSs.
- Investigation of the linkages and interdependences between UDS failure (flooding) and unexpected failures in interconnected systems such as electrical power systems and water distribution systems.
  - Modelling of interdependences between UDS failures (surface flooding) and water quality failures in water distribution systems.
  - Investigation of resilience of critical electrical power, water distribution or transport hubs to extreme flooding events
- Further investigation aimed at linking the computed resilience indices to new resilience-based flood protection level of service standards that are based
Conclusions and Recommendations

• on minimisation of the magnitude and duration flooding as opposed to use of design return periods.

• Comparison of the results obtained using the presented GRA method in which a simplified 1D modelling of surface flooding is applied with those obtained by using dual-drainage (1D-1D) or 2D rapid flood spreading models in GRA to account for the effect of the major system in providing additional system redundancies during flooding conditions is recommended.

• Evaluation of the effect of other multifunctional infrastructure such as intentional design of specific road networks in Kampala for safe conveyance of exceedance flows resulting from extreme rainfall on enhancement of global UDS resilience.

• Further development (coupling) of GRA with integrated modelling approaches to investigate the effect of catchment scale RWH strategies on enhancement of global resilience to unexpected system failures in both urban drainage and water distribution systems.
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References


## Appendix

### Table A.1 Synthetic UDS hydraulic data

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Table A.3: Hydraulic data of selected trapezoidal open channel sections in the Nakivubo UDS. The slope values represent ratios of horizontal to vertical distance.

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<th>bottom width, $b$ (m)</th>
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### Table A.5: Water demand categories for Kampala City (based on NWSC, 2014)

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<th>average water demand (l/c/d)</th>
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Table A.6: Catchment scale RWH tank model parameters

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<th>Sub catchment area (ha)</th>
<th>% Imperviousness</th>
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<th>Connected roofs (%)</th>
<th>No. of tanks</th>
<th>Connected roofs (%)</th>
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<td>72.0</td>
<td>74.2</td>
<td>1,000</td>
<td>13</td>
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<td>11</td>
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<td>76.1</td>
<td>71.5</td>
<td>45.2</td>
<td>1,000</td>
<td>22</td>
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<td>18</td>
<td>700</td>
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<tr>
<td>S12</td>
<td>81.4</td>
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<td>43.0</td>
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<td>23</td>
<td>1,000</td>
<td>19</td>
<td>650</td>
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<tr>
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<td>50.0</td>
<td>79.6</td>
<td>30.2</td>
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<tr>
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<tr>
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<td>52.9</td>
<td>56.6</td>
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<td>66.7</td>
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<td>38.8</td>
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<td>650</td>
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<tr>
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<tr>
<td>S27</td>
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<td>S31</td>
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<td>5</td>
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<tr>
<td><strong>Total</strong></td>
<td><strong>2,793.2</strong></td>
<td></td>
<td><strong>1,442</strong></td>
<td><strong>28,636</strong></td>
<td><strong>20</strong></td>
<td><strong>26,250</strong></td>
<td><strong>15</strong></td>
<td><strong>17,500</strong></td>
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</tbody>
</table>
Figure A.1: Average monthly rainfall for Kampala city. The observed rainfall data was obtained from three rain gauge stations (a) Makerere (1991-2009) (b) Municipality (1942 – 1993) and (c) City hall (1963 – 1992).

Figure A.2: Average daily rainfall for Makerere University rain gauge station for the period 1991-2009 showing a typical bi-modal peak rainfall seasons during March – May and October to December periods.
Publications
A global analysis approach for investigating structural resilience in urban drainage systems

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A B S T R A C T

Building resilience in urban drainage systems requires consideration of a wide range of threats that contribute to urban flooding. Existing hydraulic reliability based approaches have focused on quantifying functional failure caused by extreme rainfall or increase in dry weather flows that lead to hydraulic overloading of the system. Such approaches however, do not fully explore the full system failure scenario space due to exclusion of crucial threats such as equipment malfunction, pipe collapse and blockage that can also lead to urban flooding. In this research, a new analytical approach based on global resilience analysis is investigated and applied to systematically evaluate the performance of an urban drainage system when subjected to a wide range of structural failure scenarios resulting from random cumulative link failure. Link failure envelopes, which represent the resulting loss of system functionality (impacts) are determined by computing the upper and lower limits of the simulation results for total flood volume (failure magnitude) and average flood duration (failure duration) at each link failure level. A new resilience index that combines the failure magnitude and duration into a single metric is applied to quantify system residual functionality at each considered link failure level. With this approach, resilience has been tested and characterised for an existing urban drainage system in Kampala city, Uganda. In addition, the effectiveness of potential adaptation strategies in enhancing its resilience to cumulative link failure has been tested.

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1. Introduction

Recent natural and manmade catastrophic events that have led to extreme flooding in various cities worldwide have underscored the need to build resilience into existing urban drainage and flood management systems as a key strategy to minimise the resulting flooding impacts and consequences (Djordjević et al., 2011; Park et al., 2013). Urban drainage system flooding is not only caused by external climate-related and urbanisation threats such as extreme rainfall and increasing urbanisation but also internal system threats for example equipment malfunction, sewer collapse and blockages (Kellagher et al., 2009; Mugume et al., 2014; Ryu and Butler, 2008; Ten Veldhuis, 2010). System or component failures can either be abrupt (unexpected) shocks for example pump or sensor failure or chronic pressures such as asset aging and long term asset decay or sewer sedimentation. The impact of such failures, either singly or in combination on existing urban drainage infrastructure could significantly reduce the expected flood protection service levels in cities and lead to negative consequences such as loss of lives, damage to properties and critical infrastructure (Djordjević et al., 2011; IPCC, 2014; Ryu and Butler, 2008; Ten Veldhuis, 2010).

Consequently, the need to build resilience in urban drainage systems (UDSs) is increasingly recognised as vital to enhance their ability to maintain acceptable flood protection service levels in cities that they serve and to minimise the resulting flooding consequences during unexpected or exceptional loading conditions that lead to system failure (Butler et al., 2014; Djordjević et al., 2011). Although the application of concept of resilience to infrastructure systems is a recent development, there is an extensive literature on definitions and interpretation of resilience, much of which has come from the ecological systems academic community (Butler et al., 2014; Park et al., 2013). Ecological system resilience is interpreted as a measure of system integrity and is defined as a system's ability to maintain its basic structure and patterns of behaviour (i.e. to persist) through absorbing shocks or disturbances under dynamic (non-equilibrium) conditions (Holling, 1996). In
contrast to ecological systems, engineering systems are product of intentional human invention and are designed to provide continued (uninterrupted) services to society in an efficient manner (Blackmore and Plant, 2008; Holling, 1996; Park et al., 2013). Engineering system resilience is therefore interpreted differently from ecological resilience and focuses on ensuring continuity and efficiency of system function during and after failure (Butler et al., 2014; Lansey, 2012).

In the context of urban drainage, current hydraulic reliability-based design and rehabilitation approaches tend to focus on prevention of hydraulic (functional) failures resulting from a specified design storm of a given frequency (i.e. return period). The design storm return period determines the flood protection level provided by the system (Butler and Davies, 2011). Hydraulic reliability-based approaches place significant emphasis on identifying and quantifying the probability of occurrence of extreme rainfall and minimizing the probability of the resulting hydraulic failures i.e. the fail-safe approach (Ryu and Butler, 2008; Thordahl and Willems, 2008). However, such approaches fail to consider other causes of failure for example structural or component failures (Table 1) which also lead to flooding (e.g. Kellagher et al., 2009; Mugume et al., 2014; Ten Veldhuis, 2010).

Furthermore, it is argued that the direct application of reliability-based approaches for evaluation of structural failures in UDSs could be insufficient mainly because causes and mechanisms of failure are largely unknown and difficult to quantify (Ana and Bauwens, 2010; Kellagher et al., 2009; Park et al., 2013; Ten Veldhuis, 2010). It is therefore important to develop new approaches that seek to ensure that UDSs are designed to not only be reliable during normal (standard) loading conditions but also to be resilient to unexpected (exceptional) conditions i.e. the safe-fail approach (Butler et al., 2014; Mugume et al., 2014). In this study, the definition and interpretation of resilience in engineering systems is pursued. Resilience is formally defined based on recent work on ‘Safe and SuRe’ Water Management as the “the degree to which the system minimises level of service failure magnitude and duration over its design life when subject to exceptional conditions” (Butler et al., 2014). Exceptional conditions refer to uncertain threats or disturbances that lead to system failure for example climate change induced extreme rainfall events, sewer collapse or blockage. Based on this definition, the goal of resilience is therefore to maintain acceptable functionality levels (by withstanding service failure) and to rapidly recover from failure once it occurs (Butler et al., 2014; Lansey, 2012; Park et al., 2013).

Resilience is further classified into two broad categories: a) general (attribute-based) resilience which refers to the state of the system that enables it to limit failure duration and magnitude to any threat (i.e. all hazards including unknowns) and b) specified (performance-based) resilience which refers to the agreed performance of the system in limiting failure magnitude and duration to a given (known) threat (Butler et al., 2014; Scholz et al., 2011). Reliability on the other hand is defined as the degree to which the system minimises the level of service failure frequency over its design life when subject to standard loading (Butler et al., 2014). Intuitively, it is argued that reliability and resilience are related with the latter extending and building on the former. It is consequently postulated that if resilience builds on reliability, by improving the former, the latter can also be improved (Butler et al., 2014).

Taking the UK water sector as an example, recent studies have proposed range of strategies or options for building resilience in UDSs (Cabinet Office, 2011; CIRIA, 2014; Mcibain et al., 2010). These strategies generally seek to enhance inbuilt system properties or attributes such as redundancy and flexibility during design, retrofit or rehabilitation so as to influence the ability of the system to withstand the level of service failure and to rapidly recover from failure once it occurs (Hassler and Kohler, 2014; Vugrin et al., 2011). Redundancy is defined as the degree of overlapping function in a system that permits the system to change in order to allow vital functions to continue while formerly redundant elements take on new functions (Hassler and Kohler, 2014). In UDSs, redundancy is enhanced by introducing multiple elements (components) providing similar functions for example storage tanks or parallel pipes, in order to minimise failure propagation through the system or to enable operations to be diverted to alternative parts of the system during exceptional loading conditions (Cabinet Office, 2011; Mugume et al., 2014). Flexibility on the other hand is defined as the inbuilt system capability to adjust or reconfigure so as to maintain acceptable performance levels when subject to multiple (varying) loading conditions (Gersonius et al., 2013; Vugrin et al., 2011). It can be achieved in UDSs, for example, by designing in future proofing options (Gersonius et al., 2013), use of distributed (decentralised) or modular elements for example distributed storage tanks, rainwater harvesting systems, roof disconnection and use of

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Failure modes in urban drainage systems.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure mode</td>
<td>Description</td>
</tr>
<tr>
<td>Functional failure</td>
<td>Hydraulic overloading due to changes in inflows leading to failure e.g. overflow operation, surcharging and surface flooding</td>
</tr>
<tr>
<td>Structural failure</td>
<td>Malfunctioning of single or multiple components in the system such as pumps, tanks or pipes leading to the inability of the failed component to deliver its desired function in full or in part</td>
</tr>
</tbody>
</table>
designed multifunctional urban spaces such as car parks, play-grounds or roads (Mugume et al., 2014).

However, the operationalisation of resilience in urban drainage and flood management is still constrained by lack of guidelines, standards, and suitable quantitative evaluation methods (Butler et al., 2014; Ofwat, 2012; Park et al., 2013). In water distribution systems, a number of recent studies have investigated both component (structural) and hydraulic reliability when subject to stresses such as demand variations, single pipe failure and changes in pipe roughness (Atkinson et al., 2014; Trifunovic, 2012). In urban drainage systems however, most quantitative studies tend to focus on investigating hydraulic reliability which only considers functional failures such as occurrence of extreme rainfall or increasing dry weather flows (Sun et al., 2011; Thordahl and Willems, 2008). The main short coming of such approaches is that the full system failure scenario space that includes other causes of surface flooding such as equipment failure, sewer collapse and blockage is not explored.

It is recognised that different threats or combinations of threats such as extreme rainfall or sewer failure could lead to the same failed state (i.e. surface flooding). Therefore, by only considering a narrow range of hydraulic failures, current approaches take a limited view of functional resilience with no due consideration given to structural resilience. Further research is needed to develop new quantitative approaches that explicitly consider all possible failure scenarios in order to holistically evaluate resilience in UDSs (Butler et al., 2014; Kellagher et al., 2009; Ofwat, 2012; Ten Veldhuis, 2010).

In this study, a new Global Resilience Analysis (GRA) approach is developed, that shifts the object of analysis from the threat themselves to explicit consideration of system performance (i.e. failed states) when subject to large number of failure scenarios (Johansson, 2010). Global Resilience Analysis has been carried out by evaluating the effect of a wide range of progressive structural failure scenarios in various systems such as water distribution systems and electrical power systems (Johansson, 2010). The GRA methodology is extended to investigate the effect of random cumulative link (sewer) failure scenarios on the performance of an UDS. The methodology is then applied to test the effect of random cumulative link failure level are determined based on the hydraulic simulation results from 49,200 scenarios. The failure envelopes reflect vital system resilience properties that determine the resulting loss of functionality when the system is subjected to increasing failure levels. Finally, a new resilience index, ReN, that quantifies system residual functionality as a function of failure magnitude and duration is computed at each failure level for both the existing system and for the tested adaptation strategies.

2. Methods

2.1. Global Resilience Analysis (GRA) approach

Global Resilience Analysis is applied to characterise the performance of an existing UDS when subject to a wide range of structural failure scenarios involving random cumulative link failure. Structural failure in an UDS can be modelled by removal of components for example sewers (links), storage tanks or pumps in the system to represent the inability of the removed component to deliver its prescribed function. In this study, links in an UDS are randomly and cumulatively failed and the resulting impacts on the global performance of the system are investigated at each failure level until all the links in the system have been failed. This process of cumulative link failure is used to represent structural failure modes such as sewer collapse, blockages and sediment deposition in closed systems and blockage resulting from deposition of solid waste and washed-in sediments in open channel systems. The approach of failing links randomly ensures that all links, N in the system have an equal probability of being removed (Johansson and Hassel, 2012). In addition, a step by step increase in sewer failure levels enables the exploration of the full sewer failure scenario space that ranges from predictable or commonly occurring failure scenarios such as single component (N – 1), or two component (N – 2) failure modes but also other unexpected scenarios involving simultaneous failure of a large number of components (e.g. Johansson, 2010; Park et al., 2013).

To fully explore the extent of the failure scenario space in global resilience analysis, a very large number of model of simulations involving different failure scenarios would be required to capture the resulting flooding impacts (e.g. Kellagher et al., 2009). In addition, different possible sewer (link) states for example non-failed (good condition), partial or complete failure need to be evaluated (Ana and Bauwens, 2010; Kellagher et al., 2009). Taking an UDS with 81 links as an example, and assuming only two link states (non-failed or completely failed), the total number of link failure scenarios within the full failure scenario space would be $2.4 \times 10^{24}$. To reduce the computational time, a convergence analysis (Trelea, 2003) is carried out to determine the minimum number of random cumulative link failure sequences, rs, that are required to achieve consistent results (refer to Supplementary information Section 1.1). Given the significant computational burden of GRA, a simple 1D approach to modelling of surface flooding (of the minor system) is proposed rather than using more complex 2D overland flow models (Digman et al., 2014; Maksimović et al., 2009).

2.2. GRA implementation

The GRA method is implemented in the MATLAB environment linked to the Storm Water Management Model, SWMMv5.1; a physically based discrete time hydrological and hydraulic model that can be used for single event and continuous simulation of run-off quantity and quality, primarily built for urban areas (Rossman, 2010). Link failure can be modelled in SWMMv5.1 by either significantly reducing pipe diameters in the model (e.g. Mugume et al., 2014) or increasing the Manning's roughness coefficient, n to a very high value. In this study, link failure is modelled by increasing the Manning’s n from its initial (non-failed) state value ($n = 0.020$) to a very high value ($n = 100$). The high value of n was chosen because it significantly curtails the conveyance of flows in each failed link and hence enables modelling of complete failure of each link.

Model simulations are carried out at each randomly generated link failure level and system performance is quantified using the total flood volume and mean duration of nodal flooding as key performance indicators. Surface flooding is simply modelled using the ponding option inbuilt in SWMM which allows exceedance flows to be stored atop of the nodes and to subsequently re-enter the UDS when the capacity allows (Rossman, 2010). The flooding extent at each node is modelled using an assumed ponded area of
7500 m². Fig. 1 further illustrates the adopted modelling framework. The main steps in implementing the GRA include:

a) A simulation is run to assess UDS performance in its initial (non-failed) state using the considered extreme rainfall loading

b) A randomly selected single link \(c_1\), \(c_2\), \(c_3\), ..., \(c_N\), in the UDS is failed and a simulation is run using the same extreme rainfall loading. This step represents single link failure mode and is denoted as \(N/1\).

c) Two randomly selected links, in the UDS are failed (denoted as \(N/2\) failure mode) and the simulation is repeated

d) The procedure is repeated for all \(N-i\) \(i = 1, 2, 3, \ldots N\) failure modes until all the links in the system have been failed.

e) The procedure in (a)–(d) is repeated to determine the minimum number of random failure sequences \(rs_n\) that ensures convergence of results. A detailed description of convergence analysis in GRA is presented in the Supplementary information Section 1.1).

f) Using the determined \(rs_n\), the procedure in (a)–(d) above is repeated to investigate the effect of the proposed adaptation strategies on minimising the loss of system functionality resulting from the considered cumulative link failure scenarios.

2.3. Determination of link failure envelopes

The use of average values in reliability and resilience analysis simplifies results interpretation but can potentially hide key information about the range of possible failure impacts and consequences (Trifunovic, 2012). The process of determining failure envelopes provides a means of graphically illustrating the range of failure impacts at each considered failure level (e.g. Church and Scaparra, 2007). In this study, link failure envelopes are determined by computing the minimum and maximum values of all model solutions (total flood volume and mean duration of nodal flooding) obtained at each considered link failure level for the existing UDS and for the considered adaptation strategies. The resulting envelopes represent the upper and lower limits of the resulting loss of system functionality (impacts) that therefore provide vital information about the resilience properties of the system being tested. If the resulting envelope covers solutions with lower impacts at all link failure levels, then the resulting loss of system functionality is minimised during the considered failure scenarios. If the resulting envelope covers solutions with higher impacts and with a larger range between the minimum and maximum values, the tested system exhibits higher loss of system functionality during the considered failure scenarios (e.g. O’Kelly and Kim, 2007).

2.4. Computation of the flood resilience index

The resilience index, \(Res_0\), is used to link the resulting loss of functionality to the system’s residual functionality and hence the level of resilience at each link failure level. The resulting loss of

![Fig. 1. Modelling framework for random cumulative link failure in a simplified urban drainage system with 8 links, 8 nodes and 1 outfall illustrating (a) random and increasing link failure levels \(c_1, c_2, c_3, \ldots c_N\) and (b) three potential random failure sequences \(rs_1, rs_2, rs_3\).

![Fig. 2. Theoretical system performance curve for an urban drainage system. The black solid horizontal line, \(P_o\) represents the original (design) performance level of service. The blue dotted line, \(P_s\) represents a lower but acceptable level of service. \(P_f\) represents the maximum system failure level resulting from the considered threat. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)](image-url)
system functionality is estimated using the concept of severity, \( Sev_i \) (Hwang et al., 2015; Lansey, 2012). Severity is interpreted as a function of maximum failure magnitude (peak severity) and failure duration (Fig. 2). Fig. 2 illustrates the theoretical response of an UDS (in which one or more links have been failed) to a single extreme rainfall loading scenario. In Fig. 2, severity can be estimated as the shaded area between the original system performance level, \( P_0 \), and the actual system performance curve, \( P_i(t) \), at any time \( t \) after occurrence of a given threat that lead to system failure (Equation (1)).

\[
Sev_i = f[Sev_p, tf] = \frac{1}{P_0} \int_{t_f}^{t_0} (P_0 - P_i(t)) dt
\]  

Where \( tf \) is the failure duration, \( t_0 \) the time of occurrence of the threat, and \( t_e \) the total elapse time. Equation (1) above is further simplified by assuming that the system failure and recovery curve is rectangular (Equation (2))

\[
Sev_i = \frac{V_{TT}}{V_{HI}} \times \frac{t_f - t_0}{t_e - t_0} = \frac{V_{TT}}{V_{HI}} \times \frac{t_f}{t_e}
\]  

The resilience index, \( Res_o \), which is a measure of system residual functionality, is estimated as one minus the computed volumetric severity and is computed at each link failure level (Equation (3)).

\[
Res_o = 1 - Sev_i = 1 - \frac{V_{TT}}{V_{HI}} \times \frac{t_f}{t_e}
\]  

Where \( V_{TT} \) is the total flood volume, \( V_{HI} \) the total inflow into the system, \( t_f \) the mean duration of nodal flooding and \( t_e \) the total elapsed (simulation) time.

For a given threat (i.e. percentage of failed links), the proposed index quantifies the residual functionality of the UDS as function of both the failure magnitude (total flood volume) and duration (mean nodal flood duration). \( Res_o \) ranges from 0 to 1; with 0 indicating the lowest level of resilience and 1 the highest level resilience to the considered link failure scenarios. Resilience envelopes are then derived by plotting the minimum and maximum values of \( Res_o \) computed at each failure against the percentage of failed links. The resulting envelopes graphically illustrate the system residual functionality at each considered link failure level. A detailed description the theoretical behaviour of an UDS during failure conditions and the derivation of the \( Res_o \) is provided in Supplementary information Section 1.3.

### 3. Urban drainage system description and modelling results

#### 3.1. Case study UDS

A case study of the existing urban drainage system in the Nakivubo catchment, a highly urbanised part of Kampala city, Uganda is used in this work. The system requires rehabilitation to minimise the frequency, magnitude and duration of flooding during extreme convective rainfall events (Sliuzas et al., 2013). A model of the existing system is built in SWMMv5.1. The full dynamic wave model in SWMM is used to route flows through the modelled UDS. The data needed to build the model has been obtained from a Digital Elevation Model (DEM) for Kampala (2 m horizontal resolution), a 2011 satellite image for Kampala (0.5 m horizontal resolution), as-built drawings and from existing reports (e.g. KCC, 2002). A single, non-areally adjusted extreme event was used to represent a worst functional loading case in the GRA. This event used was recorded on 25th June 2012 at 10 min resolution with a 100 min duration and depth of 66.2 mm (Sliuzas et al., 2013).

The existing primary and secondary conveyance system consists of trapezoidal open channel sections constructed using reinforced concrete in upstream sections and gabion walls in the downstream sections. The resulting hydraulic model of the system consists of 81 links, 81 nodes and 1 outfall, and with a total conduit length of 22,782 m. The system drains into the Nakivubo wetland and finally into Lake Victoria. The gradients of the open channel sections range from 0.001 to 0.0124. The modelled system drains a total area of 2793 ha delineated into 31 sub-catchments (Fig. 3). The computed average subcatchment slopes and percentage imperviousness values range from 0.034 to 0.172 (Fig. A.1) and 52.3–85.7 (Table A.1) respectively. The existing system is not always clean in a ‘business as usual’ case. This was reflected in the SWMM model by taking the initial value of Manning’s n as 0.020 which is the upper limit of the recommended range (i.e. 0.010–0.020) for concrete lined channels.

#### 3.2. Modelling the effect of adaptation strategies on UDS performance

Enhancing the resilience of an UDS during design or retrofit can be achieved by altering its configuration in order to enhance its redundancy and flexibility. Redundancy could be increased by introducing extra elements such additional storage tanks, temporary storage areas or increasing spare capacity in critical links (Butler and Davies, 2011; Cabinet Of, 2011; CIRIA, 2014). Flexibility on the other hand can be increased, for example, by designing in future proofing options, use of distributed elements and provision of back-up capacity (e.g. Gersonius et al., 2013). In this study, two adaptation strategies are modelled and tested using the GRA methodology namely, addition of one large centralised detention pond (centralised storage strategy) and several, spatially distributed storage tanks (distributed storage strategy) respectively (Fig. A.2).

In the centralised storage (CS) strategy, a large centralised detention pond with a total storage volume of 3.15 × 10³ m³ is introduced upstream of link C47 (Fig. A.2a) to enhance system redundancy. In choosing the possible location of the centralised storage tank, two main criteria were used; land availability and flow rates in the downstream links in the primary Nakivubo channel. In the distributed storage (DS) strategy, 28 spatially distributed upstream storage tanks with a combined total storage volume of 3.15 × 10³ m³ are introduced at the outlets of the sub-catchments.
catchments to enhance flexibility in crucial points in the network (Fig. A.2b). The DS strategy models upstream distributed source control.

3.3. Simulation and performance assessment of the existing UDS

In order to test the performance of the modelled existing UDS, simulations were carried out and flows were investigated at selected links in the system (Fig. 4). The hydraulic data on the selected open channel cross sections is presented in Table A.2.

Lower peak flow rates, are simulated in most upstream links. The flow rates increase along the system leading to very high peaks in downstream links, for example flows of 297.4 m³/s and 318.2 m³/s are simulated at downstream links C76 and C81 respectively after an elapsed time of 75 min (Fig. A.3). Globally, 57 links (70.4%) in the system experience hydraulic overloading that consequently leads to surface flooding. Hydraulic overloading in links occurs when: (i) the upstream ends of the link run at full capacity and (ii) when the slope of the hydraulic grade line exceeds the slope of the link (Butler and Davies, 2011). The most severe hydraulic overloading is simulated in 26 links (32%), with the duration of hydraulic overloading ranging from 13 to 54 min.

The results of the simulation also indicate the system experiences flooding at a total of 57 nodes, representing a flood extent of 70.7%, with a total volume of flooding of 706,045 m³ and mean nodal flood duration of 48 ± 4 min.

3.4. Global Resilience Analysis of the existing UDS

The proposed GRA methodology described in section 2 is applied to characterise the performance of existing UDS. The overall performance of the system is quantified by simulating total flood volume and mean duration of flooding resulting from 16,400 link failure scenarios generated from 200 random link failure sequences (Fig. A.4). The average values of the total flood volume and duration of nodal flooding are computed for all the considered link failure scenarios and are presented in Fig. 5. The GRA results indicate that failure of just 10% of links leads to a disproportionately large increase of 91% in total flood volume (Fig. 5a). Thereafter, further increase in the percentage of failed links leads to comparatively small increases in the total flood volume.

![Fig. 4. Layout of the modelled Nakivubo urban drainage network.](image-url)

The situation is very different for nodal flood duration, where results show failure of 10% of links leads to just a 6% increase (Fig. 5b). Globally, the results indicate that the failure duration increases from 41 min to 56 min representing an increase of 36.2% when all the links in the system are failed.

3.5. Effect of adaptation strategies on system performance

The GRA methodology is applied to test each of the proposed UDS adaptation strategies. An additional 16,400 link failure scenarios are simulated for the CS and DS strategies respectively that is, a total of 32,800 generated from a total of 400 random link failure sequences (Fig. A.4). The effect of the CS strategy is a slight reduction of flood volume which occurs at lower link failure levels (less than 60%) with very little impact on flood duration at all failure levels. Globally, it results in a 3.4% reduction of total flood volume and a 1.1% increase in mean duration of flooding (Fig. 5).

On the other hand, the DS strategy results in a significant reduction of the total flood volume (32%) at all considered link failure levels. At link failure levels greater than 20% any additional increase in link failure levels leads to minimal increase in total flood volume. The strategy also reduces the mean nodal flooding duration from 48 min to 35 min giving a reduction of 27% for all considered link failure scenarios. Table 2 details the key statistics of the GRA results for the existing system and for the considered resilience strategies.

3.6. Link failure envelopes

The resulting link failure envelopes which represent the range of model solutions from the lowest to the highest flooding impacts computed at each link failure level are presented in Fig. 6. For the existing UDS and considering the flood volume, a large range of deviation between the computed failure envelopes and the mean values (27–87%) is observed at lower link failure levels (<20%). A convergence of both failure envelopes is observed at higher link failure levels. The results from the nodal flood duration are different, and indicate a narrow range of deviation (<26.3%) between resulting failure envelopes and the mean values at all link failure levels. Rather similar ranges of deviation between the resulting flood volume and flood duration failure envelopes and the respective mean values are observed for the CS and DS strategies respectively.

In order to evaluate the effectiveness of the considered adaptation strategies, the generated link failure envelopes are plotted into one graph to map out the failure space common to all (Fig. 7). Comparing the results of the CS strategy to those of the existing system, a slight downward shift of both the maximum and minimum flood volume failure envelopes is observed at lower link failure levels (<40%), which represents the effect of the strategy in minimising the magnitude of flooding. However, there is no significant effect at higher link failure levels. Also, the results suggest that the CS strategy has minimal effect on the flood duration failure envelopes.

For the DS strategy, a significant downward shift in the flood volume failure envelope (i.e. a reduction in the magnitude of flooding) is observed at all cumulative link failure levels. The strategy limits further increase in flood volume when link failure levels exceed 33% (i.e. a flattening of the flood volume failure envelope is observed at higher link failure levels). The strategy also shifts the flood duration failure envelopes downwards (i.e. reduces the failure duration) for all considered link failure levels when compared the existing UDS.
Fig. 5. Effect of cumulative link failure on (a) total flood volume (b) mean duration of nodal flooding for the existing UDS (ns mean), for the centralised storage strategy (cs mean) and for the distributed storage strategy (ds mean).

Table 2
Mean values of GRA results for all considered link failure scenarios. The values in the square brackets indicate the reduction range computed by considering 1 standard deviation of the mean.

<table>
<thead>
<tr>
<th>Strategy</th>
<th>Flood volume ($\times 10^3$ m$^3$)</th>
<th>Mean, $\mu$</th>
<th>Standard deviation, $\sigma$</th>
<th>% Reduction</th>
<th>Mean nodal flood duration (hrs)</th>
<th>Mean, $\mu$</th>
<th>Standard deviation, $\sigma$</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing system</td>
<td>1457.5</td>
<td>143.6</td>
<td>0.80</td>
<td>3.3 [1.0–5.1]</td>
<td>32.3 [29.9–34.1]</td>
<td>0.80</td>
<td>0.07</td>
<td>26.8 [25.6–28.4]</td>
</tr>
<tr>
<td>Centralised storage</td>
<td>1408.8</td>
<td>183.4</td>
<td>3.3 [1.0–5.1]</td>
<td>0.81</td>
<td>0.07</td>
<td>26.8 [25.6–28.4]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Distributed storage</td>
<td>986.1</td>
<td>96.3</td>
<td>32.3 [29.9–34.1]</td>
<td>0.59</td>
<td>0.03</td>
<td>26.8 [25.6–28.4]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 6. Results of generated link failure envelopes for total flood volume (a)–(c) and for mean duration of nodal flooding (d)–(f) for the existing UDS and for the CS and DS strategies.
3.7. Resilience index

The resilience index ($Res_o$) is computed using Equation (3). Based on the computed indices, resilience envelopes which represent the residual functionality of the whole UDS as a function of both the failure magnitude and duration are determined by computing the minimum and maximum values of $Res_o$ at each link failure level for the existing system for the tested adaptation strategies (Fig. 8). To facilitate comparison of the performance of the tested strategies, an assumed acceptable level of resilience threshold of 0.7 is plotted on each of the graphs, as an example of the minimum acceptable flood protection level of service (for example no property flooding) that needs to be achieved by the considered adaptation strategies.

The figure reveals large variations in $Res_o$ for the existing system and for the tested strategies at lower link failure levels (<20%) with a convergence of the results occurring with increasing link failure levels. For the existing UDS, the computed mean values of $Res_o$ range from 0.54 to 0.66. When compared to the resilience threshold, the results indicate that the existing system crosses this threshold when link failure levels in system exceed 6.2%.

Considering the CS strategy, a slight improvement in $Res_o$ of 1.2–2.3% is observed. The results indicate that resilience index falls below the threshold value when link failure levels exceed 8.6%. When the distributed storage strategy is considered, higher mean values of $Res_o$ are computed (0.76–0.84). The results also indicate that for the DS strategy, the resilience threshold is not crossed at all link failure levels. Overall, the DS strategy leads to significant improvement in the $Res_o$ of 27.5–41.4%.

4. Discussion of results

4.1. Existing system

Considering the existing system, random failure of less than 20% of the links leads to disproportionately high degradation of system functionality magnitude (i.e. total flood volume). The disproportionately high loss of system functionality suggests that failure of a small fraction of links rapidly reduces the global hydraulic conveyance capacity of the (minor) system. This result is also confirmed by critical component analysis (Johansson and Hassel, 2012) involving targeted failure of single (individual) links in the UDS (Refer to Supplementary information Section 1.1, Fig. S2). This therefore suggests that the existing UDS exhibits low levels of resilience to sewer failures. This could be attributed to the already insufficient hydraulic capacity of the system (due to use of an extreme rainstorm for modelling purposes) but could also be attributed to other key factors such as its dendritic network topology and limitations of using 1D modelling approach which excludes the contribution of the major system (i.e. effect of additional redundancies) in conveying surface flows to downstream parts of the system during extreme events.
In contrast to the total flood volume, random cumulative link failure has a limited effect on mean nodal flood duration. This could be attributed to use of a single short duration rainfall event for the simulations as opposed to using multiple events. Similarly, this could also be attributed limitations of using a simplified above ground flood model. By using a simplified above-ground flood model, surface flooding which occurs in the major system (i.e. overland flood pathways such as roads, paths or grass ways) during extreme events and which may also cause substantial damage to property and infrastructure is not considered, which could also lead to an inaccurate estimation of the mean flood duration (e.g. Digman et al., 2014; Maksimović et al., 2009).

4.2. Effect of adaptation strategies

It is argued that an effective adaptation strategy should result in a downward shift (i.e. towards the origin) of the failure envelope of the existing system. By doing this, the failure magnitude and duration is minimised across the considered failure scenarios. The derived link failure envelopes suggest that CS strategy has a very limited effect on minimising the total flood volume, with the reduction being achieved at lower link failure levels. More so, no significant effect on flood duration is observed at all considered link failure levels. As a consequence, the CS strategy only minimally improves the residual functionality of the existing system during the considered link failure scenarios. This therefore suggests that sewer failures could significantly limit the effectiveness of adaptation strategies involving enhancement of redundancy at a single location in the UDS. This also suggests that other preventive asset management strategies for example improved cleaning and maintenance practices may be more effective for resilience enhancement, because they increase spare capacity in the links themselves and minimise structural failure in existing systems (e.g. Ten Veldhuis, 2010).

In contrast to the CS strategy, the study results suggest that the DS strategy is more effective in minimising the resulting loss of functionality at all link failure levels. This could be attributed to the effect of increased the spatial distribution of control strategies (i.e. smaller decentralised upstream storage tanks with the same total storage volume as the CS strategy) results in optimal use of the total storage volume for reduction both the storm water volume and the inflow rates before entry into UDS. Reducing the storm water inflows into the system in turn enables the degraded UDS to continue functioning with minimal impacts. It could also be due to a reduction in propagation of hydraulic failures from one part of the UDS to another, which suggests that the DS strategy improves the flexibility properties of the whole (minor) system. Using this argument, it could be suggested that adaptation strategies that increase the spatial distribution of control strategies in upstream parts of the catchment for example implementation of multifunctional (dual-purpose) rainwater harvesting (DeBusk, 2013) at a city district or catchment scale could significantly increase the resilience UDSs to sewer failures.

4.3. Outlook

The developed global resilience analysis approach presents a promising quantitative tool which opens up new opportunities for holistic and systematic evaluation of the effect of a wide range of threats that have not been considered in conventional hydraulic reliability based urban drainage design and rehabilitation approaches. Future research will compare the results obtained by the presented GRA method with those obtained by using dual-drainage (1D–1D) or 2D rapid flood spreading models (e.g. Blanc et al., 2012; Maksimović et al., 2009) in GRA to account for the effect of the major system in providing additional system redundancies during flooding conditions.

Additionally, the following areas are recommended for further research.

- Investigation of the influence of inherent/inbuilt UDS characteristics for example network structure, network size (number of links), pipe diameters, pipe gradients on resilience to structural failures.
- Investigation of the effect of other types of component failures (e.g. pump failures) on global resilience in UDSs.
- Investigation of the linkages and interdependencies between UDS failure (flooding) and unexpected failures in interconnected systems such as electrical power systems.
- Further investigation aimed at linking the computed resilience indices to new resilience-based flood protection level of service standards that are based on minimisation of the magnitude and duration flooding as opposed to use of design return periods.

5. Conclusions

This research has tested and extended the global resilience analysis (GRA) methodology to systematically evaluate UDS system resilience to random cumulative link (sewer) failure. The GRA method presents a new and promising approach for performance evaluation of UDSs that shifts emphasis from prediction of the probability of occurrence of key threats that lead to flooding (the fail-safe approach) to evaluating the effects of a wide range of failure scenarios that not only includes functional failures but also structural or component failures which also contribute to flooding in cities (the safe-fail approach).

In this study, the effect of a wide range of random and progressive link failure scenarios on the ability of existing and adapted UDSs to minimise the resulting loss of functionality has been investigated. Link failure envelopes have been determined by computing the minimum and maximum values of the total flood volume and mean nodal flood duration results generated by simulating a large number of random cumulative link failure scenarios. A new resilience index has been developed and used to link the resulting loss of functionality to the system’s residual functionality at each link failure level. Based on the results of the study, the following conclusions are drawn.

- The presented global resilience analysis approach provides a promising quantitative evaluation tool that enables consideration of wide range of possible sewer failure scenarios ranging from normal to unexpected with reduced computational complexity.
- The use of convergence analysis enables determination of the minimum number of random cumulative link failure sequences required to achieve consistent GRA results, which in turn enhances that practicability of resilience assessment by significantly reducing the computational complexity involved in simulating all possible sewer failure combinations.
- Building resilience in UDSs to unexpected failures necessitates explicit consideration of the contribution of different failure modes, effect of interactions between different failures modes for example interdependencies between sewer failures and hydraulic overloading in UDS design or performance evaluation of existing systems.
- Building resilience in UDSs should not only be addressed through capital investments aimed at enhancing inherent UDS properties such as redundancy and flexibility but should also consider investments in asset management strategies such as improved cleaning and maintenance of existing UDSs.
Acknowledgement

This research is financially supported through a UK Commonwealth PhD scholarship awarded to the first author. The work is also supported through the UK Engineering & Physical Sciences Research Council (EPSRC) funded Safe & SuRe research fellowship (EP/K006924/1) awarded to last author. Acknowledgement is given to the National Water and Sewerage Corporation (NWSC) and the Kampala Capital City Authority (KCCA), Uganda for providing datasets used in SWMM model development. Thanks are given to Dr. Richard Sliuzas (University of Twente, Netherlands) for provision of high resolution rainfall data for Kampala. The insights of the three anonymous reviewers are also gratefully acknowledged.

Appendix A. Supplementary data

Supplementary data related to this article can be found at http://dx.doi.org/10.1016/j.watres.2015.05.030.

Appendix

Table A.1
Sub catchment area and computed percentage imperviousness.

<table>
<thead>
<tr>
<th>Sub catchment ID</th>
<th>Sub catchment area (ha)</th>
<th>Imperviousness (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>83.6</td>
<td>69.9</td>
</tr>
<tr>
<td>S2</td>
<td>59.5</td>
<td>71.3</td>
</tr>
<tr>
<td>S3</td>
<td>69.0</td>
<td>67.2</td>
</tr>
<tr>
<td>S4</td>
<td>97.2</td>
<td>84.1</td>
</tr>
<tr>
<td>S5</td>
<td>52.0</td>
<td>81.1</td>
</tr>
<tr>
<td>S6</td>
<td>46.1</td>
<td>76.6</td>
</tr>
<tr>
<td>S7</td>
<td>23.8</td>
<td>82.7</td>
</tr>
<tr>
<td>S8</td>
<td>10.2</td>
<td>66.2</td>
</tr>
<tr>
<td>S9</td>
<td>60.0</td>
<td>72.4</td>
</tr>
<tr>
<td>S10</td>
<td>144.4</td>
<td>72.0</td>
</tr>
<tr>
<td>S11</td>
<td>76.1</td>
<td>71.3</td>
</tr>
<tr>
<td>S12</td>
<td>81.4</td>
<td>71.1</td>
</tr>
<tr>
<td>S13</td>
<td>50.0</td>
<td>79.6</td>
</tr>
<tr>
<td>S14</td>
<td>67.3</td>
<td>75.3</td>
</tr>
<tr>
<td>S15</td>
<td>57.4</td>
<td>70.7</td>
</tr>
<tr>
<td>S16</td>
<td>55.4</td>
<td>52.3</td>
</tr>
<tr>
<td>S17</td>
<td>67.9</td>
<td>61.5</td>
</tr>
<tr>
<td>S18</td>
<td>52.9</td>
<td>56.6</td>
</tr>
<tr>
<td>S19</td>
<td>52.3</td>
<td>66.7</td>
</tr>
<tr>
<td>S20</td>
<td>158.8</td>
<td>61.3</td>
</tr>
<tr>
<td>S21</td>
<td>108.5</td>
<td>71.0</td>
</tr>
<tr>
<td>S22</td>
<td>71.0</td>
<td>78.2</td>
</tr>
<tr>
<td>S23</td>
<td>89.1</td>
<td>82.1</td>
</tr>
<tr>
<td>S24</td>
<td>25.4</td>
<td>85.7</td>
</tr>
<tr>
<td>S25</td>
<td>199.9</td>
<td>68.1</td>
</tr>
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<td>S26</td>
<td>115.7</td>
<td>62.7</td>
</tr>
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<td>S27</td>
<td>147.5</td>
<td>80.7</td>
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<td>S28</td>
<td>134.4</td>
<td>75.8</td>
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<td>S29</td>
<td>23.1</td>
<td>81.1</td>
</tr>
<tr>
<td>S30</td>
<td>88.7</td>
<td>69.1</td>
</tr>
<tr>
<td>S31</td>
<td>424.4</td>
<td>73.0</td>
</tr>
<tr>
<td>Total Area</td>
<td>2,793.2</td>
<td></td>
</tr>
</tbody>
</table>

Table A.2
Hydraulic data of selected trapezoidal open channel sections in the Nakivubo UDS. The slope values represent ratios of horizontal to vertical distance.

<table>
<thead>
<tr>
<th>Link</th>
<th>Length (m)</th>
<th>Depth, d (m)</th>
<th>Bottom width, b (m)</th>
<th>Left slope</th>
<th>Right slope</th>
<th>Equivalent pipe diameter, D_e (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C12</td>
<td>100.0</td>
<td>1.8</td>
<td>4.3</td>
<td>0.743</td>
<td>0.743</td>
<td>3.5</td>
</tr>
<tr>
<td>C40</td>
<td>290.0</td>
<td>2.5</td>
<td>1.0</td>
<td>1.000</td>
<td>1.000</td>
<td>3.3</td>
</tr>
<tr>
<td>C54</td>
<td>512.6</td>
<td>1.5</td>
<td>1.0</td>
<td>0.667</td>
<td>0.667</td>
<td>2.0</td>
</tr>
<tr>
<td>C76</td>
<td>400.0</td>
<td>4.3</td>
<td>17.4</td>
<td>0.040</td>
<td>0.040</td>
<td>9.8</td>
</tr>
<tr>
<td>C81</td>
<td>400.0</td>
<td>2.0</td>
<td>26.0</td>
<td>1.375</td>
<td>1.375</td>
<td>8.6</td>
</tr>
</tbody>
</table>

Fig. A.1. Computed Nakivubo sub catchment slopes.
Fig. A.2. Layout of adapted UDS (a) centralised storage (CS) and (b) upstream distributed storage (DS) strategy.

Fig. A.3. Simulated flows in the Nakivubo UDS for upstream links C12, C40, C54 and downstream links C76 and C81.

Fig. A.4. Effect of random cumulative link failure on total flood volume (a–c) and mean nodal flood duration (d–f). 200 random link failure sequences (16,400 random link failure scenarios) are simulated for the existing UDS ($ni; i = 1,2,3 \ldots 200$), for the CS Strategy ($csi; i = 1,2,3 \ldots 200$) and for the DS Strategy ($dsi; i = 1,2,3 \ldots 200$). In total, 49,200 link failure scenarios are simulated.

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Supplementary information
This supplement contains the following:

- Procedure for evaluating the minimum number of random failure sequences $r_{x}$ in global resilience analysis.
- Figure S1, a figure showing the full sewer failure scenario solution space for an urban drainage system with 81 links, when two system states that is non-failed and completely failed.
- Figure S2, a figure showing the convergence of global resilience analysis results for the case study UDS after 200 random cumulative link failure sequences.
Methods

1.1 Evaluating the minimum number of random failure sequences $r_s$

In order to calculate the maximum possible flooding impacts, all possible link failure scenarios should in principle be considered (Kellagher et al., 2009). Considering two states for each link (non-failed and complete failure), the total number of link failure scenarios in the entire solution space can be calculated using Equation 1

$$F(N, c_i) = \sum_{i=1}^{N} \frac{N!}{(N-i)!i!}$$

(1)

Where $F$ is the total number of failure scenarios; $N$ the total number of links and $i$ the link failure level (i.e. number of links failed).

Using an UDS with 81 links as an example and assuming the two link states (non-failed and failed), the total number of failure scenarios would be $2^{81}$ ($2.4 \times 10^{24}$) failure scenarios. Analysis of the distribution of the number of failure scenarios at each link failure level indicates a normal (Gaussian) distribution (Figure S2). The total number of scenarios involving random failure of a single link ($N - 1$) is 81. The total number of scenarios involving random failure of two ($N - 2$), three ($N - 3$), four ($N - 4$) links would be 3,240, 85,320 and 1,663,740 respectively. The highest number of potential failure scenarios occurs at the mid-point i.e. $N - 40$. Simulating such a large number ($2.4 \times 10^{24}$) of link failure scenarios would require huge computational resources (computer power, cost and simulation time) which would limit the practicability of the GRA method.
In order to minimise the computational requirements associated with considering all possible link failure combinations (Kellagher et al., 2009), a minimum number of random failure sequences, $r_{f}$ (and hence number of random failure scenarios) that should be analysed so as to achieve consistent GRA results, while covering as many failure states as possible needs to be determined.

Previous studies that employ critical component analysis (CCA) in networked systems suggest that failure of only a small fraction of components results in the significant impacts on level service delivered by the system (Church and Scaparra, 2007; Johansson and Hassel, 2012; Johansson, 2010). Critical component analysis involves an exhaustive exploration of a system state to estimate negative consequences of failure of a single or set of components (Johansson and Hassel, 2012). In UDSs, critical sewers that is; sewers for which the cost of failure would be significantly higher than upgrading costs make up 20% of the system on average (Butler and Davies, 2011). In this study, critical component analysis of single links

**Figure S1:** Distribution of link failure scenarios at each link failure level
(i.e. i.e. N-1 resilience analysis problem) is carried out by targeted failure (as opposed to random failure) of each individual link in the system. The study results suggest that failure of 19.8% of the links in the UDS lead to the highest consequences (Figure S2).

![Figure S2: Percentage increase in total flood volume resulting from critical component analysis involving single link failure](image)

Based on this, it can be suggested that for a given network, a certain minimum number of random failure sequences, $r_s$, is necessary to achieve convergence of the GRA results. This ensures that the set of links that are critical are covered in the global resilience analysis (e.g. Johansson & Hassel, 2012). A convergence analysis (Trellea, 2003) is carried out as described in following steps to determine $r_s$:

a) GRA is carried out using 5 random sequences (410 failure scenarios) and the mean values of the total flood volume are determined.

b) The procedure is repeated for 10 (820 failure scenarios), 25 (2050 failure scenarios), 50 (4100 failure scenarios), 100 (8200 failure scenarios), 150 (12,300 failure scenarios) and 200 (16,400 failure scenarios) sequences.
c) The percentage deviation, PD the between computed mean values is computed for each step-wise increase in \( rs_i \), i.e. for \( i: i = \{5, 10\}; \{10, 25\}; \{25, 50\}, \{50; 100\}, \{100; 150\} \) and \{150;200\} (Figure S3).

The results obtained from 5 random sequences indicate the largest variation in the mean values (up to 7.5%) occurs at lower link failure levels (< 10% of the failed links), with convergence occurring at higher links failure levels (Figure S3).

Figure S3: Convergence of GRA results after 200 random cumulative failure sequences (rs)

The results also indicate that increasing the number of random link failure sequences reduces this variation. A convergence is obtained after 50 random failure sequences with a maximum deviation of 4.5%. The maximum deviation is further reduced to 3.5%, 2.6% and 1.1% by considering 100, 150 and 200 random failure sequences respectively. Considering 200 random failure sequences covers all \( N-1 \) (single link) scenarios and covers a statistically significant proportion of \( N-2 \) (two link) scenarios (6.2%) and \( N-3 \) (three link failure) scenarios (0.23%). Consequently, a minimum of 200 random failure sequences is adopted for the GRA. Overall, a total of 49,200 failure scenarios involving 600 random cumulative link...
failure sequences are simulated to evaluate their resulting effect on performance of the existing and adapted UDSs.

1.2 Derivation of the flood resilience index, $Res_o$

Figure 2 illustrates the theoretical response of an UDS (in which one or more links have been failed) to a single extreme rainfall loading scenario. Even when one of the links in this UDS has failed, the UDS is able to continue functioning from time $t_f$ until when the system starts to fail at time $t_{fs}$. A gradual loss of system functionality occurs until when a maximum failure level is reached at time $t_{mf}$. After occurrence of the extreme rainfall, hydraulic capacity in the UDS is gradually restored to original ($P_o$) or to a lower performance level for example $P_a$.

The concept of severity (Hwang et al., 2015) which is a defined as the level of consequences (e.g. injury, property or system damage) that could result from occurrence given failure mode or threat is used as a measure of the resulting loss of system functionality when the UDS is subjected to a given exceptional loading scenario that leads to failure. It can be estimated as the (shaded) area between the original system performance level ($P_o$) and the actual system performance curve, $P_i(t)$, which represents the magnitude of the loss of functionality for the system being tested (Figure 2). The peak severity, $Sev_p$ is a time independent function of system performance and represents the maximum possible magnitude of loss of functionality when the UDS is subjected to a given threat. The system wide theoretical peak severity, $Sev_p$ is given by Equation 3.

$$Sev_p = \left( \frac{P_o - P_f}{P_o} \right)$$

Where $P_o$ is the original system performance level before system failure (i.e. no flooding) and $P_f$ is the lowest performance level of the system after failure i.e. the maximum possible total flood volume.
The system ‘failure impact’ duration, $t_f$ provides an estimate of the recovery time. The recovery time is defined as the time period between the onset of system failure, $t_{fs}$ (i.e. when system functionality drops below the original levels) and the return time to the original or lower but acceptable system performance levels (Lansey, 2012; Wang and Blackmore, 2009). In this study, it is estimated using the mean surface flood duration that is, the time interval between the onset and subsidence of nodal flooding (Equation 4).

$$t_f = f[t_r - t_{fs}]$$

Where $t_{fs}$ is time of start of flooding and $t_r$ is the return time to original system functionality (end of surface flooding).

In practice, however, the recovery time is dependent on other factors that are generally external to the physical design and layout of a system that is resourcefulness and rapidity. Resourcefulness is defined as the ability to respond to a failure event (Lansey, 2012). It is a measure of the capacity to identify failures, establish priorities and mobilize resources in the event of disruptions resulting from system failure (Wang and Blackmore, 2009). Rapidity on the other hand is defined as the speed at which resources are deployed to restore acceptable functionality levels (Lansey, 2012). In this study, focus is placed on quantifying the influence of inbuilt system properties/attributes on UDS resilience to flooding. Based on this premise, resourcefulness and rapidity have been excluded from the estimation of recovery time.

To estimate the resulting loss of system functionality as function of failure magnitude and duration, *volumetric severity, Sev$_v$*, which is function of the peak severity and the failure duration is computed using Equation 5.

$$Sev_v = f[Sev_p, t_f] = \frac{1}{P_o} \int_{t_o}^{t_f} (P_o - P_i(t)) dt$$
Where $Sev_i$ is the severity, $t_f$ the time of failure, $t_o$ the start time of the simulation and $t_n$ the total elapsed time.

To simplify the integration Equation 5 above, a rectangular shape of the system failure and recovery is assumed (Equation 6)

$$Sev_i = \frac{V_{TF}}{V_{TI}} \times \frac{t_f - t_f}{t_n - t_o} = \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_n}$$ \hspace{1cm} (6)

Finally, the resilience index, $Res_o$, which is a measure of system residual functionality, is estimated as one minus the computed volumetric severity and is computed at each link failure level (Equation 7).

$$Res_o = 1 - Sev_i = 1 - \frac{V_{TF}}{V_{TI}} \times \frac{t_f}{t_n}$$ \hspace{1cm} (7)

Where $V_{TF}$ is the total flood volume, $V_{TI}$ the total inflow into the system, $t_f$ the mean duration of nodal flooding and $t_o$ the total elapsed (simulation) time.
References


Enhancing resilience in urban water systems for future cities
Seith N. Mugume, Kegong Diao, Maryam Astaraie-Imani, Guangtao Fu, Raziyeh Farmani and David Butler

ABSTRACT

In future cities, urban water systems (UWSs) should be designed not only for safe provision of services but should also be resilient to emerging or unexpected threats that lead to catastrophic system failure impacts and consequences. Resilience can potentially be built into UWSs by implementing a range of strategies, for example by embedding redundancy and flexibility in system design or rehabilitation to increase their ability to maintain acceptable customer service levels during unexpected system failures. In this work, a new resilience analysis is carried out to investigate the performance of a water distribution system (WDS) and an urban drainage system (UDS) during pipe failure scenarios. Using simplified synthetic networks, the effect of implementing adaptation (resilient design) strategies on minimising the loss of system functionality and cost of UWSs is investigated. Study results for the WDS case study show that the design strategy in which flexibility is enhanced ensures that all customers are served during single pipe failure scenarios. The results of the UDS case study indicate that the design strategy incorporating upstream distributed storage tanks minimises flood volume and mean duration of nodal flooding by 50.1% and 46.7%, respectively, even when system functionality is significantly degraded. When costs associated with failure are considered, resilient design strategies could prove to be more cost-effective over the design life of UWSs.

Key words | flexibility, pipe failure, redundancy, resilience, urban water systems

INTRODUCTION

Although progress has been made towards achieving more sustainable urban water management, urban water systems (UWSs) are increasingly subject to stresses from emerging threats such as urbanisation, climate change and long-term asset degradation (Djordjević et al. 2011; Butler et al. 2014; IPCC 2014a). The impacts of emerging global climate change threats are concentrated in cities and urban areas due to the potentially high density of people, infrastructure, assets and economic activities exposed to these threats. Urbanisation impacts are also exerting significant pressure of existing UWSs (Urich & Rauch 2014). As of 2011, 52% of the global population lives in urban areas and this is projected to grow to between 64 and 69% (5.1–7.1 billion) by 2050 (IPCC 2014b). The impact of these threats on existing urban water infrastructure could lead to significant consequences such as reduced customer service levels for water supply and flood protection in the event of unexpected system failures.

Conventional (‘Safe’) design of UWSs has been greatly focussed on enhancing system reliability that is, minimising the level of service failure frequency over a given system’s design life when subject to standard loading (Park et al. 2013; Butler et al. 2014; Jung et al. 2014). However, ‘Safe’ design approaches eliminate vital attributes such as buffer and redundant capacity that could enable the system to minimise failure magnitude and duration when subjected to exceptional conditions that lead to failure (Watt & Craig 1986; Wildavsky 1988; Hassler & Kohler 2014). It is
therefore argued that embedding resilience in UWSs is key to minimising their vulnerability to the emerging threats and to maintaining acceptable customer service levels in cities that they serve (Blackmore & Plant 2008; Lansey 2012; Butler et al. 2014). Extensive literature on concepts and definitions of resilience has been led by ecological system research in which resilience is interpreted as a measure of a system’s ability to persist by maintaining its basic structure and function (system integrity) when subject to shocks or disturbances (Holling 1996). In contrast to ecological systems which exhibit dynamic and multiple stability domains, engineering systems are intentionally designed to provide continuous (uninterrupted) services to society in an efficient manner (Holling 1996; Blackmore & Plant 2008; Park et al. 2013). Consequently, the goal of engineering system resilience is essentially to ensure continuity and efficiency of system function during or after occurrence of failure (Lansey 2012; Park et al. 2013; Butler et al. 2014).

This paper builds on recent work on Safe & SuRe Water Management that seeks to ensure that UWSs are designed not only for safe (reliable) provision of services during normal (standard) loading conditions but also to be more resilient to unexpected or exceptional loading conditions (Butler et al. 2014). Resilience is defined as the ‘the degree to which the system minimises level of service failure magnitude and duration over its design life when subject to exceptional conditions’ (Butler et al. 2014). By utilising the Safe & SuRe approach, UWSs can be designed to minimise the level of service failure magnitude and duration when subjected to both standard and exceptional conditions. However, guidelines or standards for operationalising resilience in specific UWSs or sub-systems are still lacking, necessitating further investigation (Butler et al. 2014).

This paper therefore focuses on preliminary testing of both promising resilience characterisation methods and potential resilience-enhancing strategies. To achieve this, model simulations are carried out using a simplified synthetic water distribution system (WDS) and a synthetic urban drainage system (UDS) modelled in EPANET v2.0 (Rossman 2000) and the Storm Water Management Model (SWMM) (Rossman 2010), respectively, to investigate system performance under pipe failure conditions. Adaptation strategies in which flexibility and redundancy attributes are enhanced are tested with the aim of minimising the magnitude and duration of level of service failure, and outline costs associated with each strategy are quantified.

STRATEGIES FOR ENHANCING RESILIENCE IN URBAN WATER SYSTEMS

Potential strategies for enhancing resilience in UWSs are widely known and practised. They can be broadly categorised in three ways: mitigation, adaption and coping (Butler et al. 2014). In this work, the focus is placed on adaptation as an intervention strategy for enhancing UWS resilience. Adaptation entails targeted actions or adjustments carried out in a specific system in response to actual or anticipated threats in order to minimise failure consequences (IPCC 2014a; Jones & Preston 2011). Adaptation is used in this paper to refer to local responses to increasing threats such as modifying specific attributes of the system to enhance its capacity to minimise the magnitude and duration of failure to both standard (i.e. to increase system reliability) and exceptional loading conditions (i.e. to increase general or design resilience). General resilience is used in this context to refer to the state of the system that enables it to limit failure duration and magnitude to any threat (Butler et al. 2014; Hassler & Kohler 2014).

It is postulated in this work that by implementing adaptation strategies in a specific water system, both reliability and resilience could be enhanced. This could be achieved by altering the system configuration to enhance its flexibility and redundancy properties. Flexibility is defined as inbuilt system capability to adjust or reconfigure so as to maintain acceptable performance levels when subject to multiple (varying) loading conditions (Vugrin et al. 2010; Spiller et al. 2015). Flexibility can be increased in a given system through intentional one-off or phased interventions that enhance inbuilt system attributes such as flatness (less system hierarchy), buffering capacity (head room), homeostasis (feedbacks) and omnivory (diversification) (Watt & Craig 1986, Wildavsky 1988; Butler et al. 2014; Hassler & Kohler 2014). It could also be increased by ensuring that more resources (e.g. trained repair crews or emergency water supplies) are readily available at any given time to facilitate rapid response to an unexpected failure event (Lansey 2012; Butler et al. 2014; Hassler & Kohler 2014). Based on this, flexibility could in principle be increased in a given UWS through use of...
spatially distributed (decentralised) systems (e.g. Sitzenfrei et al. 2013), modular systems (e.g. Spiller et al. 2015) or through provision of back-up capacity (e.g. Ahern 2011; Cabinet Office 2011).

Redundancy on the other hand refers to the degree of overlapping function in a system that permits the system to change by allowing vital functions to continue while formerly redundant elements take on new functions (Watt & Craig 1986; Wildavsky 1988). In UWSs, redundancies could be multiple elements or components providing similar functions, to minimise failure propagation through the system or to enable operations to be diverted to alternative parts of the system during exceptional loading conditions (NIAC 2009; Ahern 2011; Cabinet Office 2011).

Table 1 provides examples of potential adaptation strategies that could in principle (a priori) improve UWS flexibility and redundancy properties. However, it is still unclear how each of these adaptation options actually enhances the resilience a given UWS in the event of unexpected system failures (Ofwat 2012; Park et al. 2015; Butler et al. 2014). In this study therefore, a new resilience analysis applied is applied to characterise the resilience of case study UWSs and to quantify the effect of implementing a range of adaptation strategies on enhancement of system resilience.

### STUDY APPROACH

The conventional UWS design and rehabilitation approach is to build reliable systems that can achieve expected customer service levels under normal or standard loading conditions (Butler et al. 2014; Jung et al. 2014). Reliability-based approaches such as least cost design formulations ensure that systems are designed to satisfy minimum acceptable service levels within given standard range of operation conditions but may eliminate vital redundancies that are required to meet the required customer services levels when system loading exceed normal conditions (Lansey 2012; Jung et al. 2014). It is now recognised that UWSs should not only be reliable but also resilient to unexpected or exceptional loading conditions. In this work, the resilience of a given UWS is illustrated through assessment of the effect of failure scenarios that could occur when the system is subjected to a wide range of unexpected threats (e.g. Johansson 2010). Pipe failure is considered for the two case studies. Figure 1 illustrates the study approach based on middle (failed) state-based global resilience analysis (Johansson 2010) that is adopted in this research. The developed approach enables system resilience to be assessed without the need to quantify the probability of occurrence of threats that lead to system failure (Johansson 2010). In the case of the WDS, single pipe failure is investigated using a simplified synthetic network that requires rehabilitation due to increased water demand. Pipe failure is used to model potential structure failure modes in WDSs that lead to pipe breakage such as pipe bursts, leakages and collapse (Kleiner & Rajani 2001). The effect of two rehabilitation strategies involving pipe replacement on resilience is evaluated and the capital costs associated with each strategy computed.

In the UDS case study, cumulative pipe failure is investigated using a synthetic network that requires rehabilitation to minimise the flooding magnitude and duration. Pipe failure is used to model potential structural failure modes in UDSs that include pipe collapse, blockages and bed load sediment deposition (Butler & Davies 2011). Two adaptation strategies in which redundancy and flexibility may be enhanced are investigated, namely, downstream centralised storage (CS) and upstream distributed storage (DS) strategies, respectively. The effect of the strategies on minimising the loss of system functionality is evaluated and the discounted total cost of each strategy is evaluated considering a design life of 50 years.

<table>
<thead>
<tr>
<th>Urban water sub-system</th>
<th>General (design) resilience attribute</th>
<th>Flexibility</th>
<th>Redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>WDS</td>
<td>Pipe replacement (critical pipes)</td>
<td></td>
<td>Increase storage tank size</td>
</tr>
<tr>
<td></td>
<td>Looping (network reconfiguration)</td>
<td></td>
<td>Parallel pipes</td>
</tr>
<tr>
<td></td>
<td>Back-up pumps</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Urban drainage system</td>
<td>Distributed source control</td>
<td></td>
<td>Add centralised storage tanks</td>
</tr>
<tr>
<td></td>
<td>Roof disconnection</td>
<td></td>
<td>Pipe replacement</td>
</tr>
<tr>
<td></td>
<td>Rain water harvesting</td>
<td></td>
<td>Parallel pipes</td>
</tr>
<tr>
<td></td>
<td>Multifunctional urban spaces</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
RESULTS AND DISCUSSION

Case study 1 - WDS

WDS resilience is illustrated by testing the performance of the system under single pipe failure scenarios. The resulting loss of system functionality for a given UWDS can be quantified using social and economic impacts such as loss of supply or repair costs. A WDS is said to be resilient (to pipe failure) if it can maintain the required level of service during the considered failure scenarios.

A synthetic WDS reported in Todini (2000) is used in this work (Figure 2(a)). The system requires rehabilitation due to water demand increase. The system consists of 1 storage tank, 9 nodes and 15 links. The actual pipe lengths are as follows: the pipe connecting the reservoir (node 0) is 2,000m long, the contour pipes are all 1,000 m long and all the internal pipes connected to node 9 are 1,210 m long. Two rehabilitation strategies are compared. Strategy B (Figure 2(b)) is a design scenario in which single pipe failure scenarios are not initially considered during the design but occur during the period of operation. In Figure 2(c), single pipe failure scenarios are taken into consideration and flexibility is introduced into the system by ensuring that for each node at least two possible paths (links) with larger diameters are provided (Strategy C). Table S1 provides details of the design scenario including demand and required minimum head for each node. Further details on the network configuration and the design scenario considered are provided in Todini (2000).

The capital costs of all the considered strategies are calculated according to diameter classes using the cost data provided in Table S2. Given the significant variability of land costs in different city contexts, land costs are not included in the capital cost calculations (e.g. Todini 2000; Farmani et al. 2005). Model simulations are carried out in EPANET to investigate the effect of single pipe failure scenarios on the ability of the UDS to minimise the resulting loss of system functionality. Loss of system functionality is quantified by computing the total water demand at the failed nodes. Comparing the two strategies described, strategy C, in which failure scenarios are considered at the design stage, cost slightly more (1.9%) but enables the system to cope with all single pipe failure events, i.e. no loss of...
supply. Contrarily, one-third of customers would be unserved in the other strategies. The reason why strategy C results in a more resilient WDS is because there are at least two paths formed by large pipes (i.e. diameter ≥203 mm) to each node. This ensures that the system capacity is better allocated compared to the other two designs.

**Case study 2 – UDS**

The existing UDS with no storage (business as usual, BAU) requires rehabilitation because its current configuration leads to unacceptable surface flooding during extreme rainfall events (Figure 3(a)). This system is a baseline configuration in which the pipes provide the hydraulic capacity of the system with no storage devices. The system consists of 26 nodes and 25 links with diameters ranging from 600 to 1,500 mm, and slopes ranging from 0.5 to 2.25% and has been designed based to satisfactory convey flows resulting from an observed 100 minute rainfall event with a total depth = 66.2 mm (Sliuzas et al. 2015) with no flooding at any node in the system. The UDS drains a total catchment area of 22.5 hectares with an average slope of 0.5% and percentage imperviousness of 25%. In addition, two adaptation strategies are modelled and used to test their effect on enhancement of system resilience: (a) CS strategy: a large storage tank is introduced upstream of pipe C24 to minimise downstream flooding by enhancing peak flow attenuation effects (Figure 3(b)) and (b) DS strategy: 9 spatially distributed upstream storage tanks (same total storage volume as the CS strategy) are introduced at the outlets of each sub catchment to enhance flexibility at critical points in the network at sub-catchment level (Figure 3(c)).

Model simulations are carried out in SWMM v5.1 to investigate the effect of cumulative pipe failure on resulting
loss of functionality (surface flooding) for the three system configurations are represented in Figure 3. Flooding is simply modelled using a flood cone with all surface flows returning to the node from which they discharged (Rossman 2010). To test the performance of the BAU system during failure conditions, model simulation is carried out using an up-scaled extreme rainfall event with a total depth of 141 mm and duration of 100 minutes (i.e. change factor of 2.14 is applied to scale the event return period, \( T \) from 2 to 50 years). Simulation results indicate that pipes in existing system (initial state) experiences hydraulic overloading that leads to flooding in most parts of the network (total flood volume of 10,910 m\(^3\) and duration of 42 minutes). A high peak flow rate of 6.95 m\(^3\)/s is attained in the downstream pipes (C24) after a simulation period of 88 minutes (Figure 4).

The system is subjected to increasing (cumulative) pipe failure scenarios (stress) in order to evaluate global performance (loss of system functionality) using total flood volume and mean nodal flood duration as key performance indicators. Pipe failure is modelled by significantly reducing pipe diameters, \( D_p \) in the model from their initial values (non-failed state) to a very small value of 1 mm in order to model pipe failure (failed state), while maintaining hydraulic connectivity required for the solution of the flow equations. A simulation run is carried out for the first random pipe failure scenario, then additionally, a second pipe is randomly failed and a second simulation run carried out. This is done cumulatively until all pipes in the network have failed. In addition, using the proposed approach, the effect of implementing the proposed adaptation strategies on improvement of global resilience is investigated. For comparison of the strategies, the discounted cost, \( PVCT \) for each strategy is computed considering a design life of 50 years, using cost functions presented in Appendix Table S3. Similarly, given the significant local city specific factors that influence land acquisition, land cost is excluded from the capital cost calculations (Swan & Stovin 2007).

The results of the BAU case in Figure 5 indicate that cumulative pipe failure leads to a rather high increase in
the total flood volume and mean flood duration. This suggests that occurrence of pipe failures in UDSs lead to significant loss of system functionality, which progressively increases with increasing pipe failure levels. The occasional ‘plateaus’ in the simulated flooding impacts (Figure 5) are attributed to failure of less critical pipes (e.g. an upstream pipe conveying low flows) with leads to minimal increase in the simulated flood volume and duration despite further increase pipe failure levels. The rapid increase in simulated flooding impacts is attributed to failure of more critical pipes. Globally, the results suggest that the existing system exhibits low of levels of resilience to cumulative pipe failure.

Overall, a total of 78 simulations are carried out and a summary of the results of the analysis is provided in Table 2.

Implementing the CS strategy results in a slight reduction in the flood volume and mean nodal flood duration; with the reduction being effective at low (<48%) pipe failure levels. Globally, the strategy results in minimal reduction of the total flood volume and mean flood duration of 4.8% and 1.9%, respectively, when compared to the BAU case. From the results, it can be interpreted that the CS strategy leads to minimal reduction of the loss of system functionality magnitude and duration and consequently a slight improvement in system resilience to cumulative pipe failure.

Implementing the DS strategy results in a significant reduction in total flood volume at all pipe failure levels implying the effect of cumulative pipe failure (stress) on loss of system functionality magnitude (strain) is reduced. In the case of the flood duration, the strain on the system increases to a maximum of 0.66 hours and remains almost constant when the cumulative pipe failure level exceeds

**Figure 5** | Effect of cumulative pipe failure on (a) flood volume (b) mean duration of nodal flooding.

**Table 2** | UDS performance indicators considering cumulative pipe failure scenarios

<table>
<thead>
<tr>
<th>Statistic</th>
<th>No storage</th>
<th>Centralised storage</th>
<th>Distributed storage</th>
<th>No storage</th>
<th>Centralised storage</th>
<th>Distributed storage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood volume ($\times 10^3$ m$^3$)</td>
<td>19.30</td>
<td>18.37</td>
<td>9.64</td>
<td>1.14</td>
<td>1.12</td>
<td>0.61</td>
</tr>
<tr>
<td>Mean failure</td>
<td>6.98</td>
<td>8.05</td>
<td>5.84</td>
<td>0.29</td>
<td>0.33</td>
<td>0.09</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>12.33</td>
<td>10.32</td>
<td>3.80</td>
<td>0.85</td>
<td>0.79</td>
<td>0.52</td>
</tr>
<tr>
<td>Min failure</td>
<td>26.28</td>
<td>26.42</td>
<td>15.48</td>
<td>1.43</td>
<td>1.45</td>
<td>0.70</td>
</tr>
<tr>
<td>Max failure</td>
<td>26.28</td>
<td>26.42</td>
<td>15.48</td>
<td>1.43</td>
<td>1.45</td>
<td>0.70</td>
</tr>
<tr>
<td>Mean reduction</td>
<td>4.8%</td>
<td>50.1%</td>
<td>-</td>
<td>1.9%</td>
<td>46.7%</td>
<td>-</td>
</tr>
<tr>
<td>[-0.5–16.3]</td>
<td>[41.1–69.2]</td>
<td>-</td>
<td>[-0.9–6.7]</td>
<td>[38.8–51.4]</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>
32%, which suggests that increasing the spatial distribution of control options in upstream parts of the network ensures optimal global performance during pipe failure scenarios. This design strategy is therefore more resilient to cumulative pipe failure when compared to the other two strategies as demonstrated by the significant mean reduction in the total flood volume and duration of nodal flooding of 50.1% and 46.7%, respectively. The results of the discounted cost calculations are presented in Figure 6.

The capital costs of the centralised and DS strategies are higher by 27% and 35%, respectively, due to the addition of storage devices. When direct tangible flooding costs are taken into consideration, the CS strategy leads to 14.5% reduction in discounted total costs. On the other hand, the DS strategy results into a 39.2% reduction in discounted total cost, which is attributed to the reduced direct tangible flooding costs.

**CONCLUSIONS**

The resilience of UWSs to unexpected threats can be enhanced by embedding redundancy and flexibility in system design, retrofit or rehabilitation. In this study, redundancy is enhanced by introducing a CS tank (UDS case). Flexibility on the other hand is enhanced by increasing network connectivity (WDS case) and use of DS tanks (UDS case). Simplified synthetic networks are used to investigate ways to evaluate resilience in existing systems and to test effectiveness of implementing various adaptation strategies in minimising the loss of system functionality and cost during pipe failure scenarios.

In the WDS case study, although the strategy in which flexibility is enhanced (strategy C) initially increases capital costs by 1.9%, it results in minimal loss of system functionality and consequently ensures adequate customer service levels during single pipe failure scenarios. In the UDS case study, the DS strategy minimises the total flood volume and mean flood duration by 50.1% and 46.7%, respectively, and hence is more resilient to cumulative pipe failure. Although this strategy has higher initial capital costs due to the additional cost of DS tanks, it results in a significant reduction of 39.2% in the total discounted total costs when direct tangible surface flooding costs over the system’s design life are taken into consideration.

This study has demonstrated that loss of system functionality during exceptional loading conditions can be minimised if failure scenarios are taken into consideration during UWS design (i.e. resilient design). The study results also suggest that enhancement of system flexibility attributes, for example through increasing connectedness and spatial distribution of control options enhances the ability of UWSs to minimise the resulting loss of functionality during unexpected system failures. The study also indicates that by taking into account the cost of failure, resilient design strategies could prove to be more cost-effective over the design life of UWSs. It is concluded that embedding resilience in UWSs provides a promising and potentially cost-effective approach to maintain acceptable customer service levels during unexpected failures and to contribute to more sustainable water management in cities in view of emerging threats. Further research using the presented approach is recommended to investigate resilience in existing real world systems and to test the effectiveness of strategies that enhance spatial distribution and connectedness properties such as city scale dual-purpose rainwater harvesting systems (e.g. DeBusk 2013) and multifunctional green infrastructure (e.g. Ahern 2011).

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Author Queries


Manuscript: WS-EM1517R1

Q1 Barreto (2012) is not cited in the main text. Please confirm where it should be cited, or delete the reference.
Q2 Please provide publisher name for Cabinet Office (2011).
Q3 Please update page number for Jung et al. (2014).
Q4 Please provide place of publication and publisher name for NIAC (2009).
Q5 In supplied Figure 1 is not sufficient print quality. Please resupply as a high resolution file (300 dpi or above) with sharp lines and text.
Q6 In supplied Figure 2 is not sufficient print quality. Please resupply as a high resolution file (300 dpi or above) with sharp lines and text.
Q7 In supplied Figure 3 is not sufficient print quality. Please resupply as a high resolution file (300 dpi or above) with sharp lines and text.
Enhancing resilience in urban water systems for future cities

APPENDIX

Table S1 | Head and demand values for the WDS case study

<table>
<thead>
<tr>
<th>Node</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Head (m)</td>
<td>200</td>
<td>197</td>
<td>193</td>
<td>192</td>
<td>191</td>
<td>191</td>
<td>192</td>
<td>193</td>
<td>189</td>
<td></td>
</tr>
<tr>
<td>Demand (m$^3$/h)</td>
<td>-180</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
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Table S2 | Diameter classes and cost of pipes

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<tr>
<th>Diameter (mm)</th>
<th>25.4</th>
<th>50.8</th>
<th>76.2</th>
<th>101.6</th>
<th>152.4</th>
<th>203.2</th>
<th>254</th>
<th>304.8</th>
<th>355.6</th>
<th>406.4</th>
<th>457.2</th>
<th>508</th>
<th>558.8</th>
<th>609.6</th>
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<tr>
<td>Cost (units)</td>
<td>2</td>
<td>5</td>
<td>8</td>
<td>11</td>
<td>16</td>
<td>23</td>
<td>32</td>
<td>50</td>
<td>60</td>
<td>90</td>
<td>130</td>
<td>170</td>
<td>300</td>
<td>550</td>
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Table S3 | Capital and operation and maintenance cost functions for an urban drainage network

<table>
<thead>
<tr>
<th>Cost ('000 £)</th>
<th>Cost Equation</th>
<th>Remarks/References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discounted total cost (PVC$_{T_0}$)</td>
<td>$\sum_{t=0}^{x} C_t / (1 + \frac{r}{100})^t$</td>
<td>$r = 3.5%$ for initial 30 yrs, then $r = 3.0%$</td>
</tr>
<tr>
<td>Total cost (C$_T$)</td>
<td>$C_P + C_M + C_L + C_{ST} + C_{OM} + C_{TF}$</td>
<td>Includes cost of failure</td>
</tr>
<tr>
<td>Pipe cost (C$_P$)</td>
<td>$0.455D_p^{1.72}$</td>
<td>Barreto (2012)</td>
</tr>
<tr>
<td>Pipe laying cost (C$_L$)</td>
<td>$70LpD_p d_p$</td>
<td>Unit cost of £70/m length/m diameter/m depth</td>
</tr>
<tr>
<td>Manhole cost (C$_M$)</td>
<td>$300d_m$</td>
<td>Unit cost of £300/manhole</td>
</tr>
<tr>
<td>Storage tank cost (C$_{ST}$)</td>
<td>$738.33V_{ST}^{0.88}$</td>
<td>Barreto (2012)</td>
</tr>
<tr>
<td>O &amp; M cost (C$_{OM}$)</td>
<td>$0.1 \times (C_P + C_L + C_M + C_{ST})$</td>
<td>10% of total capital costs</td>
</tr>
<tr>
<td>Flooding cost (C$_{TF}$)</td>
<td>$\sum_{t=0}^{x} V_{TF} f_c R_t$</td>
<td>Only direct tangible flooding costs considered</td>
</tr>
<tr>
<td>Flood occurrence probability ($R_t$)</td>
<td>$1 - \left(1 - \frac{1}{T}\right)^t$</td>
<td>Butler &amp; Davies (2011)</td>
</tr>
</tbody>
</table>

Where $r$ is the discount rate, $t$ a given time period during the system design life of $x$ years, $D_p$ the pipe diameter, $L_p$ the pipe length, $d_p$ the pipe depth, $d_m$ the manhole depth, $V_{ST}$ the storage volume (m$^3$); $V_{TF}$ the total flood volume; $f_c$ direct tangible flooding cost (£/ cubic meter of flooding); $T$ the rainfall return period; and PVC$_{T_0}$ the discounted total cost of implementing a given adaptation strategy, $y$. 
Author Queries

Manuscript: WS-EM1517R1Supplement

No Queries
**Evaluation of functional resilience in urban drainage and flood management systems using a global analysis approach**

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| Complete List of Authors: | Mugume, Seith; University of Exeter, Centre for Water Systems, College of Engineering, Mathematics and Physical Sciences  
                          Butler, David; University of Exeter, Centre for Water Systems, College of Engineering, Mathematics and Physical Sciences |
| Keywords:         | Vulnerability, Urban flooding, Extreme events, Risk Assessment, Systems analysis |
Evaluation of functional resilience in urban drainage and flood management systems

using a global analysis approach

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Abstract: Enhancing resilience in urban drainage systems (UDSs) requires new evaluation approaches that explicitly consider vital interactions between threats, system performance and resulting failure impacts during both normal and unexpected (exceptional) loading conditions. However, current reliability-based approaches only focus on prevention of functional (hydraulic) failures resulting from a specified design storm. In this study, the global resilience analysis (GRA) approach is further extended for evaluation of UDS performance when subject to a wide range of random functional failure scenarios (extreme rainfall) with varying magnitude, duration, and spatial distribution. The resulting loss of system functionality during the simulated failure scenarios is quantified using total flood volume and mean flood duration. System residual functionality for each considered block rainfall loading scenario is quantified using the \textit{functional resilience index}. The developed approach has been successfully applied to test and characterise the functional resilience to extreme rainfall of an existing UDS in Kampala city, Uganda.

Keywords: global resilience analysis, hydraulic failures, pluvial flooding, spatial rainfall distribution
Nomenclature

\( I_R \)  
\( \) rainfall intensity

\( Res_f \)  
\( \) functional resilience index

\( rs_i \)  
\( \) random sub catchment failure sequence

\( Sev_i \)  
\( \) volumetric severity

\( t \)  
\( \) rainfall duration

\( T \)  
\( \) rainfall return period in years

\( t_c \)  
\( \) time of concentration

\( t_e \)  
\( \) time of entry

\( t_{fa} \)  
\( \) failure duration

\( t_f \)  
\( \) time of flow

\( t_{mf} \)  
\( \) maximum nodal flood duration

\( t_n \)  
\( \) elapsed (simulation) time

\( V_{TF} \)  
\( \) total flood volume

\( V_{TI} \)  
\( \) total inflow volume
1. **Introduction**

The performance of existing urban drainage systems (UDSs) in various cities is increasingly threatened by multiple and uncertain threats such as climate change, rapid urbanisation and infrastructure failure that lead to catastrophic flooding impacts and consequences such as loss of lives or damage to property and critical infrastructure (Djordjević et al., 2011; Hammond et al., 2015; IPCC, 2014). Conventional *hydraulic reliability-based* urban drainage design and rehabilitation approaches focus on minimising the probability of occurrence of hydraulic failures resulting from a given design rainstorm as a basis for determining the flood protection level of a given system (Butler and Davies, 2011; Sun et al., 2011; Thorndahl and Willems, 2008). However, in view of emerging threats, it is now recognized that UDSs should be designed not only to be *reliable* during *normal* (standard) conditions but also *resilient* to *unexpected* (exceptional) loading conditions (Butler et al., 2014; Mugume et al., 2015; Park et al., 2013).

The concept of *resilience*, which has been extensively developed in the field of ecology as a measure of a system’s ability to maintain the system’s basic structure and function (*system integrity*) under dynamic or non-equilibrium conditions (Holling, 1996). In contrast to ecological resilience, engineering resilience seeks to ensure that a given system provides continuous or uninterrupted services to society during both *normal* and *unexpected* loading conditions in an efficient manner (Ahern, 2011; Butler et al., 2014; Lansey, 2012; Park et al., 2013). *Resilience* is formally defined as ‘the degree to which the system minimises the level of service failure magnitude and duration over its design life when subject to exceptional conditions’ (Butler et al., 2014).
However, operationalisation of resilience concepts in urban drainage and flood management systems has been constrained by lack guidelines, standards and suitable quantitative evaluation methods (Butler et al., 2014; Mugume et al., 2015; Ofwat, 2012; Park et al., 2013). Consequently, development of new evaluation approaches that can enable systematic evaluation of resilience of urban drainage and flooding management systems to extreme events (surprises) that lead to flooding is a subject of current research (Mugume et al., 2015, 2014).

In recent work, it is argued that effective characterisation of a given system’s general resilience (i.e. state of system that enables it to limit failure magnitude and duration to any threat) requires explicit consideration of effects of all possible threats (causes) or combinations of threats on its performance. In addition, interactions between the threats, system performance (failed state) and the resulting failure impacts should be systematically evaluated (Butler et al., 2014; Kellagher et al., 2009; Mugume et al., 2015; Ten Veldhuis, 2010). Potential failures in UDSs are broadly categorised as: (a) functional failure which results from hydraulic overloading of the system for example due to occurrence of extreme rainfall, increased dry weather flows or excessive infiltration and (b) structural failure which results from malfunction of system components (Mugume et al., 2015).

In this research, the global resilience analysis (GRA) approach (Johansson, 2010) is extended to investigate the effect of a wide range of functional failure scenarios resulting from extreme rainfall on the ability of an UDS to minimise the resulting loss of system functionality magnitude and duration (pluvial flooding). Pluvial flooding typically occurs when exceptional rainfall with intensities greater than 20 – 25mm/hr occurs over very short durations (≤ 3 hrs) and leads to functional failure of an UDS due to exceedance of the flow
conveyance capacity of the minor system or if the inlet capacity is insufficient to capture the surface runoff (Houston et al., 2011; Maksimović et al., 2009; Ten Veldhuis, 2010). It can also occur following lower intensity rainfalls (~ 10 mm/hr) over longer periods, especially if the ground surface is highly impermeable (Houston et al., 2011).

In order to reliably and realistically evaluate the effect of a wide range of functional failure scenarios on the resulting magnitude and duration of surface flooding, a computationally efficient method of modelling the effect of spatial rainfall distribution (variation), which causes non-uniform system hydraulic loading mostly during convective rainstorms is required (Butler and Davies, 2011; Chen and Djordjević, 2012; Kellagher et al., 2009). However, most urban drainage design/modelling studies apply point rainfall as uniform input over the catchment or use areal reduction factors (ARFs) to account for the differences between point and catchment averaged rainfall volumes. Such an approach may lead to inaccurate quantification of the resulting flooding impacts particularly in large urban catchments where the effect of spatial rainfall variation is considered to be significant (Achleitner et al., 2009; Butler and Davies, 2011; Einfalt et al., 2004; Kellagher et al., 2009; Vaes et al., 2005).

In a limited number of recent urban drainage modelling studies, the effect of spatial rainfall distribution on the resulting flooding impacts has been investigated using two main approaches: (a) use of radar rainfall data and (b) stochastic rainfall models (Achleitner et al., 2009; Blanc et al., 2012; Chen and Djordjević, 2012; Einfalt et al., 2004; Kellagher et al., 2009). On the one hand, widespread use of radar rainfall data in real-world applications is still constrained by insufficient (i.e. short) observed radar rainfall data sets, uncertainties or biases in radar estimates of extreme rainfall, heterogeneities in recorded radar data sets (due to continuous improvements in data processing algorithms) and other organisational
constraints (Einfalt et al., 2004; Svensson and Jones, 2010). On the other hand, although arguably more promising when compared to radar data, the direct use of stochastic rainfall model data (continuous spatial rainfall data) in urban flood modelling studies has also been constrained by significant computational burden (time/resources) required to run the simulations, need for additional pre-processing of the generated rainfall data to identify/filter significant events and unresolved inaccuracies in mathematical modelling of non-stationary local convective rainstorms patterns (Chen and Djordjević, 2012; Kellagher et al., 2009; Willems et al., 2012). Consequently, new and computationally efficient approaches that enable the practical use of spatially varying rainfall in real-world UDS resilience evaluation are required.

In this study, the developed GRA method applies block rainfall events derived from observed extreme rainfall data (IDF curves) to evaluate the effect of spatial rainfall distribution on UDS performance during extreme rainfall loading conditions. Using the developed methodology, the following key research questions are investigated:

a) What is the effect of a change in the functional loading magnitude on the ability of the UDS to minimise the resulting flooding impacts?

b) What is the effect of a change in the functional loading rate on the ability of the UDS to minimise the resulting flooding impacts?

c) How does the spatial rainfall distribution affect the performance behaviour of an UDS during extreme rainfall events?

To address these research questions, block rainfall events with varying magnitudes and intensity are derived from a set of IDF curves are used to represent the functional loading scenarios at various return periods. To model the effect of spatial rainfall distribution over the
catchment, individual sub-catchments are randomly and increasingly loaded (i.e. ‘failed’) with the selected block rainfall events until all the sub catchments in the case study have ‘failed’. The process of random and cumulative extreme rainfall loading of the sub-catchments simply represents the stochastic and distributed nature of rain cell arrivals over the catchment. It also models the effect of storm movement across the catchment (e.g. due to changes in wind direction in a convective storm) on the performance of the UDS (Vaes et al., 2005).

The developed GRA method is applied to quantify the effect of a large number of random cumulative functional failure scenarios on UDS performance. System performance (loss of functionality) is quantified at each sub-catchment ‘failure’ level using two key performance indicators that is: total flood volume and mean nodal flood duration. Based on the results of the analysis, sub-catchment failure envelopes which represent the resulting loss of system functionality (impacts) at each sub-catchment ‘failure’ level are determined by computing the upper and lower limits of the model solutions obtained from simulations involving a total of 51,200 sub catchment failure scenarios derived from 1,600 random cumulative sub catchment failure sequences, \( rs \). Finally, the resilience index, \( Res \), which quantifies system residual functionality (hence the level of functional resilience) as a function of the failure magnitude and duration, is computed for each considered block rainfall loading scenario (Mugume et al., 2015). The computed indices indicate when the design functional resilience of the existing UDS is exceeded as a result of occurrence of each considered block rainfall loading scenario.
2. Methods

A case study of the Nakivubo UDS that drains a highly urbanised central business district in Kampala, Uganda is used in this research (Mugume et al., 2015). A model of the existing system (Figure S1) has been built using the Storm Water Management Model (SWMMv5.1) and is described in detail in Mugume et al., (2015). The system was designed for a flooding return period of 10 yrs (KCC, 2002). However, during the last 10 years, the frequency, magnitude and duration of flooding incidences during extreme convective rainfall events have increased and led to negative consequences such as property damage, traffic disruption, shallow ground water contamination and structural failure of the existing paved road network (Lwasa, 2010; UN-Habitat, 2009).

2.1 Intensity-Duration-Frequency (IDF) curves and design storms

In this research, rainfall frequency analysis for Kampala city, Uganda is carried using the Annual Maximum Series (AMS) method (Butler and Davies, 2011). The total number of available daily rainfall observation years for the considered rain gauge stations is as follows: Makerere University (19), City Hall (30) and Kampala municipality (51). Because the observations have been recorded over a relatively short for reliable estimation of extreme rainfall with higher return periods, the AMS method was applied to determine the $T = 2$ yr rainfall depths where the prediction accuracy is high. Thereafter, a generalised Gumbel equation is applied to determine the 24 hr point rainfall for $T = 5, 10, 25, 50$ and 100 years. Temporal disaggregation is carried out to determine rainfall depths and intensities for $t = 15$ min, 30 min, 1 hr, 2 hr, 4 hr, 6 hr and 12 hrs using existing rainfall ratios for Kampala (Equations 1 and 2) which relate the average rainfall intensity, $I$ (mm/hr) to the duration, $t_d$ (hr) for a given return period for Kampala (Fiddes et al., 1974; MoWT, 2010).
\[
I = \frac{a}{(t+b)^c}
\]  

(1)

Where \(a\), \(b\) and \(c\) are constants (\(b = 0.33\), \(c = 0.95\))

By eliminating \(a\), Equation 1 can be simplified into Equation 2.

\[
R_T = \frac{t}{24} \left(\frac{24+b}{b+t}\right)^c \times R_d
\]  

(2)

Where \(R_T\) is the rainfall depth for any duration, \(t\), \(R_d\) is the 24 hour rainfall.

IDF curves are derived by plotting a graph of rainfall intensity, \(I_R\) against duration, \(t\) for the respective return periods (Figure 1).

### 2.2 Functional loading scenarios

In contrast to application of uniform spatial rainfall loading over the whole catchment, block rainfall events are applied randomly and progressively to the sub-catchments using the GRA method that is described in detail in section 2.3. The block rainfall events have a constant intensity over their duration, \(t\) that is greater than or equal to the time of concentration, \(t_c\) and are consequently chosen for subsequent resilience analysis. For a given duration and return period, each block rainstorm represents an engineering ‘worst case’ functional loading scenario (Butler and Davies, 2011). Consequently, it is argued that using block rainfall events for UDS model simulations enables assessment of maximum loss of system functionality (i.e. hydraulic overloading) for a given return period and duration. The main steps taken to derive the block rainfall events include: (a) computation of the time of concentration, \(t_c\), and (b) derivation of block rainfall events as a function of constant rainfall intensities (read off the IDF curves) and time \(t : t > t_c\).

#### 2.2.1 Computation of time of concentration, \(t_c\) for Nakivubo UDS

For a given rainfall intensity, \(I_R\), the critical storm duration that causes the catchment to operate at steady state (equilibrium) and to generate maximum flows equals the time of...
concentration, \( t_c \) (Butler and Davies, 2011). In this study, the \( t_c \) is estimated using the TR-55 method, which is recommended for large urban catchments (NRCS, 1986). A detailed description of the computation of \( t_c \) is provided in Supplementary information section 1.3.

The time of entry and average time of flow are computed as 13.1 minutes and 52.1 minutes respectively (i.e. \( t_c = 65.2 \) minutes). The computed value of \( t_c \) for the Nakivubo catchment is rather short considering a total contributing area of 2,793 ha. However, this is attributed to the steep sub catchment slopes, high imperviousness levels (52.3 – 85.7%) and urbanisation effects that have increased channelization of the previously natural drainage system leading to high channel flow velocities (Sliuzas et al., 2013). Based on these results, a duration of 70 minutes is taken as the critical storm duration for subsequent functional resilience analysis.

2.2.2 Derivation of block rainfall events

Two sets of block rainfall events are chosen that is: \( t = 2t_c \) (140 minutes) and \( t = t_c \) (70 minutes). The two sets of block rainfall events are derived by reading off corresponding intensities, \( I_R \) from the IDF curves at \( t = 140 \) minutes and \( t = 70 \) minutes respectively for \( T = 5, 25, 50 \) and 100 years (Figure 2). The block rainfall durations, \( t: t \geq t_c \) are chosen to ensure that UDS performance is assessed at steady state conditions (Butler and Davies, 2011). The derived 70 minute block rainfall events have higher rainfall intensities (63%) but slightly lower total rainfall depths (19%) when compared to the 140 minute block rainstorms.

2.3 GRA implementation

Global resilience analysis is applied to characterise the performance of an existing UDS when subject to a wide range of functional failure scenarios resulting from extreme rainfall.
Functional failure is modelled by *random* and *cumulative* loading of the sub catchments with the derived block rainfall events to represent system hydraulic overloading that leads to surface flooding. The adopted approach of random and cumulative ‘failure’ of sub catchments models the effect of spatial rainfall distribution (variation) over the catchment, which leads to spatially non-uniform hydraulic loading in the UDS. For each sub catchment, 2 system states are considered:

(a) *Non-failure*: The sub catchment is loaded with an insignificant (dummy) block rainfall event (constant $I_R = 6$ mm/hr, $t = 100$ minutes) that does not cause flooding at any of the nodes in the UDS.

(b) *Failure*: The sub catchment is loaded (‘failed’) with a specified block rainfall event (Figure 2) that leads to hydraulic overloading of the links and flooding in parts of UDS.

Given the significant computational burden involved in simulating such a large number of scenarios, the minimum number of sub catchment failure sequences, $r_s$, necessary to achieve consistent GRA results is determined using *convergence analysis* (Mugume et al., 2015). In addition, a simple 1D modelling of surface flooding (i.e. nodal flooding of the minor system) is applied, rather than using more complex 2D overland flow models.

Model simulations are carried out in a MATLAB environment linked to the Storm Water Management Model (SWMM v5.1) to quantify the UDS performance at each failure level, using total flood volume and mean nodal flood duration as system performance indicators. A time period of 7 hours is used for the wet weather simulation. Surface flooding is modelled using the ponding option inbuilt in SWMM which allows exceedance flows to be stored atop
of the nodes and to subsequently re-enter the system when the capacity allows (Rossman, 2010). The main steps taken in applying the GRA approach include:

a) A simulation is run to quantify the initial state performance of the UDS i.e. with all sub-catchments in a non-failure state.

b) A randomly selected sub-catchment, $S_i: i = 1, 2, 3...S_N$ is ‘failed’ and a simulation is run to quantify the UDS performance, where $S_N$ is the total number of sub-catchments.

c) In the next iteration, two randomly selected sub-catchments are ‘failed’ and a second simulation is run.

d) The procedure is repeated by running simulations at each failure level until all the sub-catchments, $S_N$ in the catchment area have been failed.

e) Convergence analysis is carried out by repeating the procedure in (a) – (d) for a range of random sub-catchment failure sequences $rs_i$ for $i = 1, 2, 3...m$; where $m$ is the minimum number of $rs_i$ that should be evaluated to achieve consistent GRA results. The study results suggest that at least 200 random failure sequences are sufficient (Refer to supplementary information section 1.4).

f) The minimum, mean and maximum values of all model solutions (total flood volume and mean nodal flood duration) are computed at each considered sub catchment failure level and used to derive the resulting sub catchment failure envelopes. The envelopes represent the upper and lower limits of the resulting loss of functionality.

g) The procedure described in (a) – (d) and (f) is then carried out for other block rainfall events i.e. $T = 5, 25, 50 & 100$ years.
In addition, the GRA results are compared with simulation results obtained by applying an areal reduction factor computed for the Nakivubo catchment (ARF = 0.9) to the derived design storm profiles for each return period (Figure S2).

2.4 Computation of functional resilience index

The functional resilience index, $Res_f$, is used to link the resulting loss of functionality to the system’s residual functionality and hence the level of resilience during the considered block rainfall loading scenarios. The resulting loss of system functionality is estimated using the concept of volumetric severity, $Sev_i$, which provides a measure of the level of consequences (e.g. injury, property or system damage) that could result the simulated failure impacts (e.g. Hwang et al., 2015). In this study, $Sev_i$ (Equation 3) is estimated as a function of maximum surface flooding magnitude and duration which effectively assumes that the system failure and recovery curve is rectangular (Mugume et al., 2015).

$$Sev_i = \frac{V_{TF}}{V_{TI}} \times \frac{t_{fn}}{t_{mf}}$$  \hspace{1cm} (3)

Where $V_{TF}$ is the total flood volume; $V_{TI}$ the total inflow into the system; $t_{fn}$ the mean duration of nodal flooding and $t_{mf}$ the maximum nodal flood duration.

However, it is noted that using the simulated surface flood duration (obtained using the 1D surface flood model), does not consider the duration of flooding that occurs in the major system (i.e. overland flow paths such as roads, paths and grass ways) during extreme events which could lead to underestimation of the mean flood duration. In addition, it is noted that the simulated surface flood duration represents the ‘failure impact’ time and does not include other factors that affect system recovery time such as ‘system repair’ time and the ‘failure consequence’ e.g. time taken to repair a property affected by flooding (Mugume et al., 2015).
The functional resilience index, $Res_f$, is estimated using Equation 4 and ranges from 0 to 1; with 0 indicating the lowest level of functional resilience and 1 the highest level functional resilience to the considered extreme rainfall loading scenarios (Mugume et al., 2015). It is computed at 100% (full) sub catchment failure level and hence represents the most severe functional loading scenario for each considered block rainfall event.

$$Res_f = 1 - \text{Se} \nu_i = 1 - \frac{v_{TF}}{v_{TI}} \times \frac{t_{fn}}{t_{mf}}$$

(4)

In addition, Equation 4 is used to compute the design functional resilience, $Res_{f,d}$ for the Nakivubo UDS (designed for a 10 yr flooding return period). For the computation, it is assumed that the 10 year design flooding return period corresponds to a 2 yr design rainstorm. Consequently, $Res_{f,d}$ is computed by simulating the effect of the 2yr design rainstorm on the resulting loss of system functionality.

3. Results

3.1 Effect of spatial rainfall distribution on flooding

3.1.1 Effect on total flood volume

Simulation results obtained using the 140 minute block rainfall events indicate the effect of increasing spatial rainfall distribution on the ability of the UDS to minimise the loss of system functionality is less pronounced at lower return periods (e.g. $T = 5$ yrs) but increases with increasing rainfall return periods. This can be observed in Figure 3 b, c and d where the simulated total flood volume at higher sub-catchment failure levels significantly increases with increasing rainfall return periods, implying that increased spatial loading of the sub catchments leads to disproportionally high loss of system functionality magnitude.
Secondly, the results show that applying uniform, areally reduced rainfall over the catchment (i.e. use of ARFs) over estimates the total flood volume at spatial rainfall loading levels less than 70% and that the overestimation increases with increasing $T$ (Figure 3). On the other hand, the results also indicate that use of ARFs could lead to underestimation of the total flood volume at higher spatial rainfall loading levels; for example in Table 1, the total flood volume at a spatial rainfall loading level of 90% (which corresponds to the applied ARF factor of 0.9) underestimates the total flood volume by 15.9 – 33.9% at higher $T$ ($T \geq 25$ yrs).

### 3.1.2 Effect on mean flood duration

The results generally suggest that for all rainfall return periods, ‘failure’ of about 20% of the sub-catchments results in the highest increase in the mean flood duration. When the sub-catchment ‘failure’ exceeds 20%, minimal variation in the mean flood duration is observed for all considered $T$ (Figure 3). A slight reduction in the mean flood duration is observed at higher sub-catchment ‘failure’ levels, which is due to the effect of ‘averaging’ i.e. the number of flooded nodes increases with increasing total flood volume. Subsequently, the effect of ‘averaging’ leads to more stable results and in some instances lower mean values of the flood duration. The results also suggest for higher $T$ (i.e. 25, 50 and 100 years), that use of ARFs (with the assumption of uniform loading) slightly underestimates the mean flood duration (by 3.4 – 10.1%) when sub-catchment ‘failure’ levels exceed 15%.

### 3.2 Effect of a rapid increase in rainfall intensity on flooding

#### 3.2.1 Effect on total flood volume

To model the effect of a rapid increase in rainfall intensity, the GRA is carried out using the 70 minute block rainfall events as functional loading inputs. The GRA results indicate that when compared to the 140 minute block rainfall events, the 70 minute block rainfall events...
result in higher loss of system functionality magnitude at all considered rainfall return periods (Figure 4). The effect on total flood volume is more pronounced for when the spatial rainfall loading exceeds 40%. The results indicate that the 70 minute block rainfall events result in a significant increase of 41 – 135% in the simulated total flood volume (at 90% sub catchment ‘failure’ level) when compared to the 140 minute block rainfall events for all considered $T$.

### 3.2.2 Effect on mean flood duration

In contrast to the flood volume results, the 70 minute block rainfall events result in slightly lower mean flood duration values when compared to corresponding 140 minute block rainfall events. The effect is pronounced when sub catchment ‘failure’ levels exceed 10% (Figure 4). The results show that the 70 minute block rainfall events result in a reduction of 25 – 40.8% in the simulated mean flood duration (at 90% sub catchment ‘failure’ level) when compared to the 140 minute block rainfall events for all considered $T$.

### 3.3 Functional resilience index

The computed functional resilience indices for the considered block rainfall loading scenarios are presented in Figure 5. The computed design functional resilience index (0.91) represents the design flood protection level of service delivered by the existing UDS. The results also indicate that occurrence of shorter duration, high intensity rainstorms with higher return periods, significantly reduces the residual functionality of the UDS and hence it’s functional resilience to extreme rainfall. For example occurrence of the 50yr70 minute and 100yr70 minute block rainfall events result in a reduction of functional resilience of 24% and 32% respectively when compared to the UDS’s design functional resilience.
4. Discussion of results

The developed GRA method enables systematic evaluation of functional resilience in UDSs with reduced computational complexity. Specifically, the results of the study suggest that the resulting loss of functionality of the existing UDS increases with increasing block rainfall event magnitudes. In addition, the study results indicate that the loss of system functionality is more sensitive to functional loading resulting from the short duration, high intensity block rainfall events when compared to corresponding lower intensity block rainfall events and that this sensitivity is higher when the spatial rainfall loading extent exceeds 40%. This therefore suggests that the existing UDS exhibits low levels of resilience to extreme rainfall that could result from anticipated future climate change or climate variability.

Secondly, the study results also suggest that current approaches which use uniform rainfall loading inputs (with ARFs applied) may lead to overestimation of the magnitude of flooding resulting from a given rainfall event when the spatial rainfall loading is less than 70%. However, for rainfall events that cover that entire catchment, use of uniform spatial rainfall loading underestimates the resulting magnitude of flooding. These results suggest that effective design (or sizing) of catchment scale resilience enhancement strategies such as distributed storage or rainwater harvesting systems should apply spatially distributed rainfall inputs to achieve accurate results.

Thirdly, the generated sub catchment ‘failure’ envelopes suggest that in addition to the areal rainfall extent, storm movement, which may result from a change of wind direction (e.g. Vaes et al., 2005) during a given extreme rainfall event affects UDS performance and hence its functional resilience. The effect of random and increasing spatial rainfall loading from upstream to downstream parts of the catchment results in higher failure impacts. On the other
hand, random and increasing spatial rainfall loading from downstream to upstream parts of the catchment resulting in lower flooding impacts. These results are attributed to the non-uniform system hydraulic loading during non-stationary rainstorms. As the spatial rainfall loading is gradually extended to cover downstream parts of the catchment, the generated flows from upstream parts of the catchment reach downstream links just when the local (downstream) storm run-offs are entering the UDS leading to higher flooding impacts.

5. Conclusions

In this chapter, the global resilience analysis (GRA) method has been developed and applied to evaluate the functional resilience of an existing UDS in Kampala, Uganda when subject to a wide range of extreme rainfall loading conditions. The developed methodology facilitates improved understanding of the hydraulic performance behaviour of existing UDSs during unexpected extreme events. It also enables the effect of spatial rainfall distribution to be explicitly considered in UDS resilience evaluation with reduced computational complexity.

From the study, the following conclusions specific to the Kampala city are drawn:

- Occurrence of short duration, high intensity rainfall events leads to significant loss of system functionality magnitude but has less effect on failure duration when compared to corresponding lower intensity rainfall events. Globally, it is concluded that short duration, high intensity rainfall events (i.e. 70 minute block rainfall) result in more significant reduction (24 – 32%) of the existing UDS’s functional resilience.

- Because the short duration events lead to higher loss of functionality magnitude but less effect of duration, it is suggested that implementation of multifunctional infrastructure for example intentional design of specific road network sections ( major
system) to enable safe conveyance of exceedance flows during extreme rainfall events could provide a promising option for enhancement of the system’s functional resilience. Other promising strategies that focus on upstream source control of stormwater inflows into the UDS for example distributed storage and dual purpose rainwater harvesting are recommended for further investigation.

- Use of areal reduction factors can lead to overestimation of the magnitude of flooding resulting from extreme rainfall events with higher return periods (T > 25 yrs) when the actual spatial rainfall extent is less than 70% of the total catchment area. This suggests that future planning and design of resilience enhancement strategies should apply spatially distributed rainfall inputs to enable effective design/size of potential adaptation strategies and therefore to minimise erroneous and costly adaptation decision making (e.g. Gersonius et al., 2013).

Furthermore, the following general conclusions on evaluation of functional resilience in UDSs are drawn:

- For large urban catchments, the effect of spatial rainfall variation can lead to spatially non-uniform system hydraulic loading, which significantly influences the hydraulic performance behaviour and hence functional resilience of UDSs during extreme (convective) rainfall conditions.
- The developed GRA approach provides a realistic, practical and computationally efficient method that can be applied by water utilities/companies for diagnostic assessment of functional resilience in existing or planned UDSs.
- The developed approach can be applied to inform decision making processes for example during prioritisation of investments in capital or asset management
interventions that are required to build resilience in UDSs in view of emerging climate related and urbanisation threats.
Acknowledgement

This research is financially supported through a UK Commonwealth PhD scholarship awarded to the first author. The work is also supported through the UK Engineering & Physical Sciences Research Council (ESPRC) funded Safe & SuRe research fellowship (EP/K006924/1) awarded to last author. Acknowledgement is given to the Water Resources Management Department, (Ministry of Water and Environment) for provision of the long term observed rainfall data sets for Kampala.
References


Figures captions

Figure 1: Derived intensity-duration-frequency curves for Kampala.

Figure 2: 140 minute block rainfall events derived from an IDF curve for Kampala, Uganda for T = 5, 25, 50 and 100 years. The blue dashed lines show the corresponding block rainfall events derived from the IDF curves at t = 70 minutes.

Figure 3: Generated UDS failure envelopes showing the effect of spatial rainfall distribution on total flood volume (a-d) and mean nodal flood duration (e-h) for 140 minute block rainfall events with various rainfall return periods. The red dashed dot horizontal line (ARU) shows computed values of total flood volume and mean nodal flood duration using corresponding areally reduced uniform rainfall (Design storms with an ARF of 0.9 applied).

Figure 4: Mean values of GRA results obtained using 140 minute and 70 minute block rainfall events showing the effect of increased rainfall intensity on total flood volume (a-d) and mean duration of nodal flooding (e-h) for various rainfall return periods.

Figure 5: Computed functional resilience indices for the existing UDS at various block rainfall loading scenarios.
Figures

**Figure 1:** Derived intensity-duration-frequency curves for Kampala
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Supplementary information

Evaluation of functional resilience in urban drainage and flood management systems using a global analysis approach

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This supplement contains the following

- Figure S1, a figure showing the layout of the modelled Nakivubo urban drainage network (Mugume et al., 2015)
- Figure S2, a figure showing the derived design storm profiles for Kampala
- A description of the computation of the time of concentration, \( t_c \) for the Nakivubo UDS
- Figure S3, a figure showing the simulated link velocities in the Nakivubo UDS
- Figure S4, a figure showing the computed average link velocities along the longest flow path in the Nakivubo UDS
- Description of a methodology based on convergence analysis for evaluation of the minimum number of random sub catchment failure sequences required to achieve consistent GRA results
- Figure S5, a figure showing convergence of global resilience analysis results after 200 random cumulative sub catchment failure sequences
1.1 Model of the existing Nakivubo UDS in Kampala

![Figure S1: Layout of the modelled Nakivubo urban drainage network (Mugume et al., 2015)](image)

1.2 Derived design storm profiles for Kampala

![Figure S2: Design storms for Kampala for various return periods, T = 2, 5, 10, 25, 50 and 100 years. $I_R$ is the rainfall intensity. The design storm profiles (symmetric histograms) are derived from the IDF curves using a procedure described in Fiddes et al., (1974) and MoWT, (2010).](image)
1.3 Computation of time of concentration, $t_c$ for the Nakivubo UDS

The time of concentration (Equation E1), is defined as the time required for surface run-off to flow from the remotest part of the catchment area to a point under consideration (Butler and Davies, 2011).

$$t_c = t_e + t_f$$  \hspace{1cm} (E1)

Where $t_e$ is the time of entry (overland flow time) and $t_f$ the time of flow.

In this work, the time of concentration is estimated using the TR-55 method, which is recommended for large urban catchments (NRCS, 1986). This approach subdivides the time of entry, $t_e$ into two components i.e. sheet flow, $t_{sf}$ and shallow concentrated flow, $t_{sc}$. Having identified the longest channel flow path and hence the remotest sub catchment (S3), the two components are computed and $t_e$ estimated. To estimate the time of flow, $t_f$, average velocities in the UDS links along the longest channel flow path are required. Model simulations are carried out in SWMM model using the 2 yr 24 hr design storm profile with a total depth of 70 mm (Figure S2a) to compute the link velocities, $v_i$ at each 5 minute simulation time step.
Figure S3: Simulated link velocities in the Nakivubo UDS

The average link velocity, $v$, is calculated using the simulated $v_i$ for the middle 80% of the time steps (i.e. $v_i$ computed at the lower and upper 10% of the time steps respectively are excluded to avoid underestimation of $v$). This is illustrated in Figure S3 for selected links C3, C24, C45, C76 and C81. The results of the computed average link velocities are presented in Figure S4.
The results indicate that flow velocities range from a minimum of 1.7 m/s to 3.7 m/s and show an increasing trend along the channel length i.e. from upstream to downstream links in the UDS. Although relatively high, the computed average velocities are comparable to observed flow conditions during extreme rainstorms in Kampala (e.g. Sliuzas et al., 2013). Based on the average velocities, the individual times of flow, $t_{c,i}$, are computed in each link and the time of flow for the entire catchment computed by summing up the individual $t_{c,i}$ for all links along the longest channel flow path.

### 1.4 Convergence analysis

To fully explore the sub-catchment failure scenario space, a large number of simulations is required. For example for a catchment with 31 sub catchments, and assuming the two system states above, the full failure scenario space would be $2^{31} = 2.15 \times 10^9$ failure combinations. In this study, convergence analysis (e.g. Trelea, 2003) is carried out to determine the minimum number of random sub catchment failure sequences, $r_{s_x}$, required to achieve consistent GRA results. A methodology for convergence analysis described in (Mugume et al., 2015) is applied in this study. The study results suggest that at least 200 random sub
catchment failure sequences, i.e. $200 \times 32 = 6,400$ sub catchment failure scenarios should be
simulated for each block rainfall event (Figure S5).

![Figure S5](image)

**Figure S5:** Convergence of GRA results after 200 random cumulative sub catchment failure sequences

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Resilience-based evaluation of urban drainage systems: The ‘Safe & SuRe' Approach

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Keywords: extremes, functional failure, reliability, resilience, structural failure

Abstract

The need to develop more resilient urban drainage systems (UDSs) is now widely recognised as key to maintaining acceptable flood protection service levels in cities in view of emerging climate-related, urbanisation and ageing infrastructure threats. In the UK water sector, the goal of resilience is well understood and supported by a suite of promising intervention strategies (Hepworth, 2015). However, operationalisation of resilience in urban flood management is still constrained by lack of suitable quantitative evaluation methods (Butler et al., 2014). Current approaches only focus on prevention of hydraulic failures for example due to occurrence of an extreme rainfall event of a given return period. New evaluation approaches that consider ‘all possible threats’ including existing network capacity and asset failures such as equipment malfunction, sewer collapse or blockage are required (Kellagher et al., 2009; Mugume et al., 2015).

This paper builds on recent work on Safe & SuRe Water Management that seeks to ensure that urban drainage systems are designed or redesigned not only for safe (reliable) provision of services during normal conditions but also to be more resilient to unexpected or exceptional loading conditions (Butler et al., 2014; Mugume et al., 2015). A new and computationally efficient global resilience analysis (GRA) approach (Figure 1) that shifts emphasis from accurate quantification of threat occurrence probabilities to evaluation of UDS performance under a wide range of possible failure scenarios is developed (Johansson, 2010; Mugume et al., 2015).

Figure 1: Middle-state based global resilience analysis of UDSs
This paper describes the Safe & SuRe framework, the developed GRA method and presents results of recent work where the method has been successfully applied to evaluate the effect of both structural (sewer failure) and functional (extreme rainfall) failures on the ability of an existing UDS in Kampala, Uganda to minimise the magnitude and duration of flooding and to test effectiveness of potential adaptation strategies.

References
Moving from reliability to resilience-based evaluation of urban drainage infrastructure: A case study of Kampala, Uganda

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Keywords
Extreme rainfall, functional failure, resilience, urban flooding

Extended Abstract
The performance of existing urban drainage systems (UDSs) in various cities is increasingly threatened by multiple and uncertain threats such as climate change, rapid urbanisation and infrastructure failure which lead to negative flooding impacts and consequences. However, conventional urban drainage design and rehabilitation approaches tend to focus on minimising the probability of hydraulic failures resulting from a chosen design storm as a basis for determining the flood protection service level delivered by a given system (Butler and Davies, 2011; Sun et al., 2011; Thorndahl and Willems, 2008). Such hydraulic-reliability based approaches may be insufficient for ensuring acceptable flood protection levels in cities during unprecedented extreme events. Consequently, to enhance the resilience of UDSs, new and computationally efficient evaluation approaches that can enable explicit consideration of vital interactions between threats, system performance and resulting failure impacts during both normal and exceptional loading conditions are required (Butler et al., 2014; Kellagher et al., 2009; Mugume et al., 2015).

UDS resilience is investigated using a case study of the Nakivubo UDS that drains a highly urbanised catchment in Kampala, Uganda (Figure 1). Over the last decade, Kampala has experienced an increase in the number of pluvial flooding incidences with negative consequences such as property damage, traffic disruption and shallow ground water contamination among others (Lwasa, 2010; Sliuzas et al., 2013; UN-Habitat, 2009). The main causes of flooding in Kampala include: extreme rainfall (caused by climate change and variability), rapid urbanisation, insufficient drainage infrastructure and inadequate system cleaning and maintenance.

Figure 1: Layout of the modelled Nakivubo urban drainage network (Mugume et al., 2015)
In this research, the Global Resilience Analysis (GRA) approach (Mugume et al., 2015) is extended to investigate the effect of a wide range of random functional failure scenarios (extreme rainfall) with varying magnitude, duration and spatial distribution on the ability of the case study UDS to minimise the resulting magnitude and duration of flooding (loss of system functionality). The developed GRA method applies block rainfall events (Figure 2) derived from observed extreme rainfall data (IDF) curves for Kampala as opposed to use of design rainstorms. Use of block rainfall events for UDS model simulations enables more accurate assessment of maximum loss of system functionality for a given return period and duration (Mugume and Butler, 2015). Functional failure is modelled through random and cumulative loading of subcatchments with specific block rainfall events to simulate hydraulic overloading that leads surface flooding. A large number of model simulations are run in a MATLAB environment linked to the Storm Water Management Model, SWMM v5.1 (Rossman, 2010). UDS performance is quantified at each considered failure level using total flood volume and mean duration of nodal flooding as key system performance indicators.

Figure 2: 140 minute block rainfall events derived from IDF curves for Kampala, Uganda for T = 5 & 50 years. The blue dashed lines show the corresponding 70 minute block rainfall events (Mugume and Butler, 2015).

The study results indicate that the 70 minute block rainfall events lead to significant loss of system functionality magnitude but have less effect on flood duration when compared to corresponding 140 minute rainfall events. The results also indicate that for both 140 and 70 block rainfall events, degradation of system functionality is exacerbated by increasing spatial rainfall distribution and return period (Figure 3). This suggests that the residual hydraulic conveyance capacity of the existing UDS is significantly reduced by occurrence of short duration high intensity rainstorms, indicating that the system exhibits low levels of resilience to extreme rainfall. Because the short duration events lead to higher loss of functionality magnitude but less effect of duration, it is suggested that implementation of innovative multifunctional infrastructure for example intentional design of specific road network sections to enable safe conveyance of exceedance flows could provide a promising option for enhancing global resilience to extreme events in Kampala.
Figure 3: GRA results for the existing Nakivubo UDS when subject to random cumulative functional failure

It is further concluded that using the proposed GRA approach facilitates more realistic evaluation of system performance under a wide range of spatially distributed rainfall inputs and could thus minimise potentially erroneous and costly adaptation decisions by ensuring more accurate design (sizing) of resilience enhancement strategies such as distributed storage or dual-purpose RWH systems in cities.

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Quantifying the Resilience of Urban Drainage Systems Using a Hydraulic Performance Assessment Approach

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ABSTRACT
Although considerable progress has been made towards achieving sustainable urban water management, urban drainage systems (UDSs) are increasingly threatened by multiple and uncertain drivers of future change. Building the resilience of UDSs to flooding is increasingly recognised as an imperative to promoting the long term sustainability of the urban areas they serve. This paper describes a methodology that combines the use of hydraulic performance assessment with utility performance functions to quantify the resilience of UDSs during flooding (exceedance) conditions. Utility performance functions, which relate the overall UDS performance to flood depths, are derived from existing flood depth-damage data for UK residential properties for various rainfall return periods and are used to estimate UDS residual functionality and hence resilience to pluvial flooding. The study shows that by introducing a storage tank for flow attenuation, the duration of nodal flooding and the flooded volume can be reduced by 6 to 10% and 18 to 38%, respectively and the overall system resilience to flooding can be increased by 8.0 to 9.5%.

KEYWORDS
Hydraulic performance assessment, resilience, restorability, robustness, urban flooding, utility performance functions

INTRODUCTION
Building resilience in urban drainage systems (UDSs) is increasingly recognized as being important to minimise flooding impacts and consequences under uncertain future climate change and urbanisation conditions (Blockley et al., 2012; Butler and Davies, 2011; Djordjević et al., 2011; Gersonius et al., 2013). The concept of resilience provides a paradigm shift from conventional ‘fail-safe’ approaches to a holistic ‘safe-to-fail’ view that accepts, anticipates and plans for failure under exceptional (non-design) conditions that could occur over the design life of the system (Ahern, 2011; Francis and Bekera, 2014). In the context of urban flood management, resilience can be defined as the robustness and restorability of the system over its design life when subjected to exceptional conditions. Robustness refers to the degree to which an UDS minimises the level of service failure magnitude over its design life when subject to exceptional conditions. Restorability (recoverability) on the other hand refers to the degree to which a system minimises level of service failure duration over its design life when subjected to exceptional conditions (Francis and Bekera, 2014; McDaniels et al., 2008).

In recent studies, significant progress has been made towards understanding and quantifying resilience in water distribution systems (Jung et al., 2013; Lansey, 2012). However, few studies have focused on developing suitable methodologies for quantitative assessment of resilience in UDSs. This paper therefore defines resilience in the context of UDSs and
describes a methodology that combines hydrologic and hydrodynamic simulations with the use of derived utility performance functions to quantify the performance of UDSs and their resilience to flooding. Utility performance functions are mathematical models that relate a system performance attribute of interest to an index that ranges from 0 to 1; with zero given to the performance attribute valued least by the decision maker (Cardoso et al., 2004; Gharaibeh et al., 2006).

**RESILIENCE OF URBAN DRAINAGE SYSTEMS**

Urban drainage infrastructure projects are often large, capital intensive and with long design lives. These characteristics introduce uncertainties in the planning and design of an UDS to guarantee a given level of service over the system’s design life (Djordjević et al., 2011; Mailhot and Duchesne, 2010). Building UDS resilience to extreme rainfall events is therefore vital to maintain acceptable flood protection levels in urban areas that they serve in view of anticipated future conditions. Resilience can either be focused on the level of service afforded to customers (and the environment) or on the systems, assets or networks that deliver the services (Mott MacDonald, 2012). From a review of resilience literature, three distinct interpretations of resilience can be identified: i) as a way of thinking - *epistemic* ii) as a quantifiable characteristic of a specific system in respect to a specific threat or known unknown - *specified resilience* and iii) as a system-wide state that determines the capacity to absorb threats of all kinds including unknown unknowns - *general resilience* (Carpenter et al., 2012, 2001; Cumming et al., 2005; Folke, 2006). This paper focuses on specified resilience of UDSs to extreme rainfall induced pluvial flooding. Resilience is interpreted as the ability of the UDS system to minimize the magnitude and duration of flooding resulting from extreme rainfall events.

**Quantifying resilience in urban drainage systems**

Developing suitable quantitative resilience assessment methodologies can enable characterization and testing of the performance behavior of UDSs during flooding conditions. With improved understanding of system behavior, potential mitigation and adaptation strategies aimed at providing appropriate customer service levels can be tested and prioritised. Figure 1 presents a theoretical system performance curve in which *robustness* and *failure* are represented as *time independent* functions of *system performance*, $P_i$, while *response* and *recovery* are represented as both *system performance* and *time dependent* functions.

![Figure 1: Theoretical system performance curve for an UDS](Adapted from Henry and Ramirez-Marquez, 2012; McDaniels et al., 2008; Mens et al., 2011).
**Robustness** is dependent on in built multiple ‘fail-safe’ mechanisms (e.g. parallel pipes, storage tanks or flood retention basins) that enable the system to maintain system functionality or to minimise failure magnitude when subjected to exceptional loading (Jung et al., 2013; Lansey, 2012; NIAC, 2009). In Figure 1, the theoretical system robustness, \( R_{ob} = f[P_s - P_a] \); where \( P_s \) is the original (stable state) performance level before system surcharging and onset of surface flooding and \( P_a \) is the minimum acceptable system performance level which corresponds to no property flooding. In utility theoretic terminology, it can be postulated that robustness is maximized if flooding depth is minimized. A robust UDS, which conveniently conveys runoff generated by a given extreme rainfall event with minimal flooding is highly preferred by the decision maker and would consequently be allocated a higher utility performance value compared to one that leads to higher flood depths.

**Response** refers to the system’s ability to buffer shocks so as to enable graceful as opposed to rapid degradation of system functionality when subjected to exceptional conditions. The gradient of the ‘response’ part of the system performance curve is an indicator of the sensitivity of the UDS functionality (Lansey, 2012). It is given by \( f(P_f - P_o)/(t_f - t_o) \); where \( P_f \) is system failure which corresponds to flood depths, \( 0.6 < x < 3.0 \) m, \( t_f \) the time to start of system performance degradation and \( t_o \) the time to failure.

**Restorability** can be expressed as a function of the return time to original (or lower but acceptable) system functionality following failure. It is mainly dependent on available human and capital resources, efficient contingency planning, and competent emergency response operations among others (McDaniels et al., 2008; NIAC, 2009). In Figure 1, system restorability, \( Restore = f[t_r - t_f] \); where \( t_r \) is the return time to original system functionality. In utility theoretic terms, restorability can be maximized by minimizing the return time to original performance levels. A highly restorable system that quickly recovers to original functionality after failure is most preferred by the decision maker and can consequently be allocated a higher utility performance value.

**SYSTEM PERFORMANCE EVALUATION**

**System configuration and simulation options**

A synthetic urban drainage system (UDS) consisting of 9 nodes and 9 links with diameters ranging from 400 mm to 800 mm and draining five 4-hectare sub catchments with an average slope of 0.5% was used for used for hydrologic and hydrodynamic simulations using the Storm Water Management Model (SWMM) v.5.0 (Figure 2). SWMM is a physically based discrete time hydrological and hydrodynamic model that can be used for single event and continuous simulation of run-off quantity and quality primarily built for urban areas. SWMM utilizes both the kinematic wave and the full dynamic wave models (St. Venant equations) to route flows through a network of pipes, open channels, storage or treatment units and diversion structures and can model various flow regimes such as backwater, surcharging, reverse flow and surface ponding (Rossman, 2010). The ponding option in SWMM allows exceedance flows either to be lost or to be stored atop of the nodes and to subsequently re-enter the UDS when the capacity allows.

Two UDS configurations were compared: i) configuration 1 - without storage and ii) configuration 2 - with a storage tank with a maximum volume of 4,933 m³ (maximum depth = 3m; surface area = 5,000 m², ponded area = 5,000 m²). The storage tank performs the function of flood peak attenuation to enhance the robustness and restorability of the UDS (Figure 2). In UDS configuration 2, the diameter of link C5 (inlet into the tank) was increased from 600 mm
to 800 mm to improve the hydraulic conditions during filling and draining of the tank (e.g. Kim et al., 2013). The outlet from the tank was modelled as bottom type orifice with a height of 1 m, width of 0.5 m and an inlet offset of 0.5 m. Infiltration was modelled using the Green-Ampt model and flow routing was modelled using dynamic wave model with ponding was allowed atop of each node.

**Figure 2(a) UDS without storage (b) UDS with a storage tank for flood peak attenuation**

**Event based rainfall data**

Model simulations were carried out to investigate the performance of the synthetic UDS in respect to extreme rainfall induced pluvial flooding. For the simulations, an observed 2 year, 100 minute convective rainfall event for Kampala, Uganda with a resolution of 10 minutes and a total rainfall depth of 66.2 mm was used in the study (Mhonda, 2013). To account for the effect of increasing intensity of extreme rainfall events resulting from climate change, rainfall depths for events with higher return periods, $T$ of 5, 10, 25, 50 and 100 years were estimated based on the observed rainfall event characteristics using a generalized rainfall-duration frequency relationship (Shaw 1994) for short duration tropical convective rainstorms (Equation 1).

$$R_T^t = (0.35\ln T + 0.76)(0.54t^{-0.25} - 0.50)R_{260}$$

for $2 \leq T \leq 100$ years and $5 \leq t \leq 120$ minutes; where $R$ is the rainfall depth (mm), $t$ is rainfall duration (min). Two key assumptions that formed the basis for applying this approach are: i) that the recurrence interval of extreme rainfall events changes under future conditions (for example a 1 in 10 year event becomes a 1 in 2 year event), (ii) that temporal characteristics of the rainfall events remain unchanged under anticipated future conditions (Mugume et al., 2013). Based on these assumptions, the rainfall depths (in mm) and corresponding climate change factors (in brackets) were estimated for $T = 5, 10, 25, 50$ and 100 years as 87.9 (1.33), 104.0 (1.57), 125.3 (1.89), 141.4 (2.14) and 157.5 (2.38) respectively (Figure 3).

**Figure 3: Observed extreme rainfall event on 25th June 2012 for Kampala (Obs) and estimated future extreme rainfall events with return periods, $T = 5, 10, 25, 50$ and 100 years.**

Developing flood depth-based utility performance functions
Existing depth-damage data for UK residential properties (Penning-Rowsell et al., 2010) for various flood depths thresholds, \( x \) and return periods, \( T \) was used to derive utility performance functions \( u(x)_T \) for an UDS during failure conditions. The functions relate overall performance of an UDS to flood depths; with the most preferred system performance level by the decision maker (no flooding, \( u(x=0) \)) and the least preferred system performance level by the decision maker (flood depths greater than or equal to 3 m, \( u(x\geq 3.0) \)) being allocated utility performance values of 1 and 0 respectively. Equation 2 was applied to estimate utility performance values, \( u(x)_T \) for \( x = 0.1, 0.3, 0.6, 0.9 \) and 1.2 m.

\[
u(x)_T = 1 - \frac{D_x}{D_{max}}
\]

Where \( D_x \) is the flood damage attributed to a flood depth \( x \), occurring after an elapsed time \( i \), and \( D_{max} \) is the maximum flood damage for a particularly rainfall return period, \( T \). Figure 4(a) shows the depth-damage curves for UK residential properties and Figure 4(b) shows the derived utility performance functions for the respective return periods.

**Figure 4 (a):** Depth-damage curves for single UK residential properties  
**Figure 4 (b):** Computed flood depth based utility performance functions

Estimation of UDS resilience
The derived utility performance functions, \( u(x)_T \), are used to estimate the system’s residual functionality by assigning utility performance values, \( u(t) \) to the system based on the simulated flood depths at each 5 minute time step. A higher utility performance value (close to 1) represents a higher proportion of system functionality retained after a flooding event and consequently a high level of system performance. Conversely, a low utility performance value (close to 0) implies that a lower residual functionality is retained by the system after a flooding event. Therefore, a system with a high average performance value over all simulation time steps can be considered to be more resilient compared to one with a lower average performance value because it has higher residual functionality. This therefore implies that a highly resilient system maintains higher residual functionality levels relative to original or pre-event levels after a flooding event. A surrogate measure of overall UDS resilience, \( Res_i \), which combines robustness and restorability, can therefore be estimated by

\[
Res_i = \frac{1}{t_n - t_0} \int_{t_0}^{t_n} u(t) dt,
\]

where \( t_0 \) is the start time of the simulation and \( t_n \) is the total elapsed time at the end of the simulation as represented in Figure 1.

**RESULTS AND DISCUSSION**

Derived utility performance functions
The derived utility performance functions indicate that system performance is negatively correlated to increasing flood depths. The 5-year extreme rainfall event that results in flood depths of up to 0.6 m degrades the system hydraulic performance by 84%. Beyond flood depths of 0.6 m, the marginal degradation in hydraulic performance decreases significantly. This is explained by the steep slope of depth-damage curves up to flood depths of 0.6 m, which indicate that maximum damage to residential property occurs between flood depths of 0 - 0.6 m. Secondly, the effect of duration of flooding also affects the nature of the derived utility performance functions. Higher rainfall return periods result into higher flood durations and hence higher degradation of UDS performance. At very higher return periods (e.g. \( T = 50 \) or 100), the shape of the derived utility performance functions is almost identical.

**Hydrological and hydrodynamic simulation results**

Simulation results for UDS configuration 1 result in a maximum flood duration of 0.79 hours and flood volume of 14,319 m\(^3\) for the 25 year rainfall event. The maximum flood depth of 1.24 m occurred after an elapsed time, \( t = 70 \) minutes. The effect of addition of a storage tank reduces the average duration of nodal flooding and the flood volume to 0.72 hours and 8,486 m\(^3\) respectively for the 25 year rainfall event, with a maximum flood depth of 1.07m occurring after an elapsed time, \( t = 70 \) minutes. Figure 5 provides a plot of computed average flood depths against elapsed time for the both UDS configurations. The effect of introduction of a storage tank is reflected in the downward shift of the peak flood depths for \( T = 5, 10 \) and 25 years. However, the effect is minimal for high magnitude events i.e. \( T = 50 \) and 100 years.

![Nodal flooding for UDS without storage](image1)

![Nodal flooding for UDS with storage](image2)

**Figure 5** (a) Nodal flooding for UDS without storage (b) Nodal flooding for UDS with storage

Overall, the addition of a storage tank reduces the average duration of nodal flooding and the flooded volumes by 6 – 21% and 18 - 58% respectively (Figure 6).

![Average duration of nodal flooding](image3)

![Total flood volume](image4)

**Figure 6:** (a) Duration of flooding and (b) total flood volume for various extreme rainfall event return periods
Computed UDS resilience
The overall system resilience ranges from 0.76 \((T = 5)\) to 0.59 \((T = 100)\) for UDS configuration 1 (Table 1). The effect of the addition of a storage tank increases system resilience to 0.83 \((T = 5)\) and 0.64 \((T = 100)\). System resilience is therefore increased by 8.0 – 9.5\% and the hydraulic performance of the UDS is restored to its original level before the end of the simulation period for all rainfall return periods (Figure 7).

![Figure 7: Urban drainage system performance curves (a) without storage (b) with storage](image)

However, the introduction of additional storage does not completely eliminate nodal flooding. This could be attributed to the capacity and positioning of the storage tank, the sewer network configuration or the characteristics of inlet and outlet control devices (Kim et al., 2013). To achieve considerable improvements in system performance and hence resilience to flooding a number of strategies require further investigation (i) effect of changing the drainage network configuration (including the positioning of the storage tank) and ii) implementation of sustainable drainage systems (SuDs) in the upstream catchments.

<table>
<thead>
<tr>
<th>Table 1: Overall system resilience for various return periods</th>
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<tbody>
<tr>
<td>Return period, T (T)</td>
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<tr>
<td>System resilience (without storage)</td>
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<tr>
<td>System resilience (with storage)</td>
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<tr>
<td>% Increase in system resilience</td>
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CONCLUSIONS
Resilience is defined as the ability of an UDS to minimize the magnitude and duration of flooding. Utility performance functions derived from depth-damage data for UK residential properties are applied to estimate the residual functionality (and hence resilience) of an UDS by assigning utility performance values to the system based on SWMM v.5.0 model simulation results. The proposed methodology provides a promising approach for quantifying resilience of UDSs. It can also be applied to evaluate and prioritize potential, cost effective mitigation and adaptation strategies aimed at providing appropriate customer service levels. Further work will focus on developing separate performance metrics for system robustness and restorability and investigating the effect of different failure modes i.e. pipe failure and sediment deposition on UDS resilience.
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STATISTICAL DOWNSCALING METHODS FOR CLIMATE CHANGE IMPACT ASSESSMENT ON URBAN RAINFALL EXTREMES FOR CITIES IN TROPICAL DEVELOPING COUNTRIES – A REVIEW

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ABSTRACT

Results of most global and regional climate model simulations cannot be directly applied in future change impacts and adaptation studies of urban drainage and flood risk management. A form of downscaling is required to increase the spatial and temporal resolution of the modelled rainfall data. This paper provides a critical review of the current state of the art statistical downscaling techniques that can be applied to quantify climate change impacts on urban rainfall extremes. Emphasis is placed on delta change methods and Poisson cluster stochastic rainfall models. The paper discusses the applicability and key limitations of statistical downscaling in climate impact and adaptation studies for cities in tropical developing countries. From the review, it can be concluded that simpler statistical downscaling techniques with modest resource requirements such as climate impact sensitivity analyses, use of simple Markov chain or semi-empirical models, construction of climate analogues and spatial interpolation of grid point data are appropriate for scoping of climate impacts and evaluation of mitigation and adaptation strategies at the city scale. Emerging resilience based approaches that combine both scenario based climate model projections and acceptability thresholds defined by key flood risk management stakeholders are promising for application in climate impact and adaptation studies for cities in tropical developing countries.

KEYWORDS
Climate change, tropical developing countries, delta change, stochastic rainfall models

1. INTRODUCTION

The impact of climate change on local extreme rainfall patterns and flood risk in urbanised catchments is a subject of current research (Chen and Djordjević, 2012; Djordjević et al., 2011). Global climate model (GCM) projections indicate that climate change can lead to changes in frequency and intensity of extreme rainfall events which consequently impacts urban drainage and flood risk management decisions (Butler and Davies, 2011; IPCC, 2007a). Climate change has the potential to exacerbate flood risk particularly in cities in tropical developing countries due to the anticipated increase in extreme rainfall events, finite design capacity of existing systems and changing socio-economic trends among other urbanisation challenges (Djordjević et al., 2011; IBRD/WB, 2009). However, results of global and regional climate model simulations exhibit coarse spatial and temporal resolutions and hence cannot be directly applied in urban drainage and flood risk studies (Onof and Arnbjerg-Nielsen, 2009; Willems, Arnbjerg-Nielsen et al., 2012). Statistical downscaling techniques offer a viable approach to generate accurate and reliable high spatial and temporal resolution rainfall data that is relevant for urban drainage and flood risk management (Bates et al., 2008; Chen and Djordjević, 2012; Onof and Arnbjerg-Nielsen, 2009; Sunyer et al., 2012; Willems, Olsson, et al., 2012).

This paper therefore provides a critical review of the current state of the art on statistical downscaling techniques that can be applied to quantify climate change impacts on urban rainfall in tropical developing countries. The paper differs from previous review papers on downscaling such as Willems, Arnbjerg-Nielsen, et al., (2012); Wilby and Wigley (1997); Wilby et al., (2004) and Fowler et al., (2007) which have not specifically addressed its applicability in the context of urban areas and cities in tropical developing countries. In this review, emphasis is placed on two promising statistical downscaling methods with relatively limited resource requirements: delta change methods (Olsson et al., 2012) and Poisson cluster stochastic rainfall models i.e. the Bartlett-Lewis Rectangular Pulse (BLRP) and the Neyman-Scott Rectangular Pulse (NSRP) models (Burton et al., 2008; Butler and
2. EVIDENCE OF CLIMATE CHANGE IMPACTS ON EXTREME RAINFALL EVENTS

Results of global and regional climate modelling experiments suggest a general trend towards more frequent and intense extreme rainfall events especially in tropical and high latitude regions (IPCC, 2007a; Olsson et al., 2012; Willems, Olsson, et al., 2012). In Northern Europe, climate model projections indicate a clear tendency towards increases in both annual mean winter precipitation and extremes of daily precipitation (IPCC, 2007a; Jenkins et al., 2009; Willems, Olsson, et al., 2012). Jenkins et al., (2009) projected an increase of up to 33% in winter precipitation in western UK and up to 40% decline in summer precipitation in South England by 2080 in UK against the 1961-1990 baseline using probabilistic multi-model projections. In tropical and high latitude regions such as East Africa, South, East and South East Asia, climate model projections indicate a general increase in both annual mean precipitation and an increase in the frequency and intensity of extreme precipitation events (Bates et al., 2008; IPCC, 2007b). Such climatic trends have vital implications for urban drainage and flood risk management particularly in cities and urbanised areas in tropical developing countries that are highly vulnerable to future change impacts. Consequently, high spatial and temporal resolution rainfall data sets are required by impact modelers for climate impact assessments, proposition and evaluation of context specific mitigation and adaptation measures at the city scale (Butler and Davies, 2011; Sunyer et al., 2012; van Vuuren et al., 2011; Willems, Olsson, et al., 2012).

3. STATISTICAL DOWNSCALING METHODS

Downscaling offers an appropriate methodology that can be used to generate high spatial and temporal resolution data of between 1 - 5 km and 5 - 15 minutes respectively, which satisfies the data requirements of urban drainage and flood risk studies (Butler et al., 2007; Onof and Arnbjerg-Nielsen, 2009). Two main approaches that can be employed to refine coarse climate model data to generate high resolution data include dynamic and statistical downscaling. Dynamic downscaling utilises Regional Climate Models (RCM) set up for specific regions of interest and nested within a Global Climate Model (GCM) to simulate local scale climate features such as orographic precipitation, extreme climate events and regional scale climate anomalies at high spatial (between 12 – 50 km) and temporal (daily time step) resolutions using a physically based approach (Fowler et al., 2007; Sunyer et al., 2012; Willems, Arnbjerg-Nielsen, et al., 2012). In a recent study, Kendon et al. (2012) used a very high resolution convection-permitting RCM with a spatial scale of 1.5 km and compared it with a 12 km RCM to study the realism of simulated hourly heavy rainfall events in the UK. However, RCM model results inherit biases of the driving GCM, and increase with increasing intensity of rainfall events (Kendon et al., 2012). Furthermore, the use of RCMs may still necessitate an extra statistical downscaling step to attain the necessary spatial and temporal resolution for urban drainage studies (Fowler et al., 2007; Sunyer et al., 2012; Willems, Arnbjerg-Nielsen, et al., 2012).

Statistical downscaling methods on the other hand can be used to generate ensembles of daily climate that evolve in line with the transient, large scale changes of the host climate model (Diaz-nieto and Wilby, 2005). Statistical downscaling is premised on the concept that regional climates are fundamentally a function of the large scale atmospheric state and that the relationship can be expressed as a stochastic or deterministic function between the large scale atmospheric variables and the local or regional climate variables (Fowler et al., 2007; Sunyer et al., 2012; Wilby et al., 2004). This approach further assumes that the ratios of large scale (spatial) to local point statistics remains constant under climate change, an assumption that is considered a major limitation to the approach (Diaz-nieto and Wilby, 2005; Fowler et al., 2007; Onof and Arnbjerg-Nielsen, 2009; Sunyer et al., 2012). Statistical downscaling methods can be broadly classified into four main groups: delta change, regression based, weather typing (re-sampling) methods and stochastic rainfall models (Fowler et al., 2007; Onof and Arnbjerg-Nielsen, 2009; Sunyer et al., 2012; Willems, Arnbjerg-Nielsen, et al., 2012). Figure 1 below graphically illustrates the respective methods and their interrelationships.
In this paper, emphasis is placed on delta change methods and the use of stochastic rainfall models which have received greater prominence in recent studies on downscaling of extreme rainfall for urban drainage impact studies (Onof and Arnbjerg-Nielsen, 2009; Sunyer et al., 2012). The other methods i.e. regression based methods and weather typing methods have been found to be inadequate for simulation of extreme events mainly due to their inadequacy in representing extreme events (Fowler et al., 2007; Wilby et al., 2004).

### 3.1 Delta change methods

In this method, change factors (CF) are used to quantify changes in rainfall frequencies and intensities between a control period and a future period for specified return periods (Olsson et al., 2012). The computed change factors can be applied to baseline observations by simply adding or scaling the mean climatic change factor to each period. In order to account for annual variability, change factors can be computed separately for each month or season (Olsson et al., 2012; Sunyer et al., 2012). The main advantage of this approach is the ease and speed of application and the direct scaling of the scenario in line with the changes resulting from the climate model results (Diaz-nieto and Wilby, 2005). Climate change factors are dependent on both the aggregation level (temporal scale) and the return period (rainfall intensity level) and can be formulated using various statistical distributions e.g. probability distributions of rainfall intensities, rainstorm cumulative volumes and wet and dry spell lengths (Willems, Olsson, et al., 2012).

The method can be applied for both continuous and event based applications. In the continuous case, short term precipitation from climate projections is analysed using the partial duration series method to estimate delta change factors associated with different percentiles in the frequency distributions of non-zero intensities. In the event based case, Intensity-Duration Frequency (IDF) curves for a given location can be downscaled using extreme value analysis of annual maxima series.

Semadeni-Davies et al., (2008) computed climate change factors for 6-hour rainfall intensities based on two RCM model runs to study the combined effects of climate change and urbanisation on sewer flows in Helsingborg, Sweden. The computed monthly change factors varied from a 50% decrease to over 500% increase in rainfall intensity for the future period 2071 - 2100. Olsson et al., (2009) extended the delta change methodology to downscale rainfall time series of Kalmar city, Sweden by calculating changes in the probability distribution of rainfall intensities and modelling the delta change factors as a percentile function. The results of the assessment indicated that summer and autumn
rainfall intensities would increase by 20% to 60% and would lead to an increase of 20% to 45% in the number of surface floods the year 2100.

In a recent study, Olsson et al., (2012) applied the delta change method to both continuous time series and event based analytical applications using precipitation data from climate model results for Linz, Austria and Wuppertal, Germany. In the continuous time series case, delta change factors were obtained by computing the ratio between a certain percentile in the future period by the same percentile in the reference period. In the event based case, delta change factors were computed by dividing the Gumbel estimate for a certain return period in a future period by the same estimate in a reference period. The computed change factors were thereafter used to estimate future design storms as illustrated in Figure 2 (Olsson et al., 2012).

![Figure 2: Example of historical 30 year 1-hour EULER II design storm for Wuppertal (OBS) and downscaled version based on future climate model projections (ECHAM5 and HADCM3 denoted as ECH and HAD respectively) (Olsson et al., 2012)](image)

The main limitations of the delta change method include its deterministic nature, dependence on the reliability of the driving GCM and RCM climate models, requirement of equivalent climate model and observational data sets and assumption that the number of wet and dry days remains constant under climate change (Diaz-nieto and Wilby, 2005; Olsson et al., 2012; Sunyer et al., 2012). These limitations can be tackled either by making the delta change approach entirely event based or by using probabilistic multi-model (ensemble) projections to account for uncertainty (Fowler et al., 2007; Olsson et al., 2012).

### 3.2 Stochastic rainfall models

Stochastic rainfall models (also referred to as weather generators) are used to simulate plausible daily or hourly rainfall series of any length conditioned upon large-scale atmospheric information. The statistical parameters of the stochastic model are computed based on statistical analysis of time series of observed data and climate model results (Kilsby et al., 2007; Onof and Arnbjerg-Nielsen, 2009; Willems, Arnbjerg-Nielsen, et al., 2012). Recent studies argue that stochastic rainfall models are more appropriate for extreme event generation and hence are very relevant for urban drainage modelling studies (Sunyer et al., 2012; Willems, Olsson, et al., 2012).

The first rainfall models were simplified and based on Markov chain models and semi empirical models for wet and dry periods. First order Markov chain models simulate rainfall occurrence and amounts using transition probabilities and gamma distributions respectively. Second and third order Markov chain models were aimed at improving simulation of precipitation occurrence and persistence. Markov chain models are generally inefficient in modelling the clustered nature of rainfall occurrence (Fowler et al., 2007). Semi-empirical weather generators on the other hand use partly empirical distributions (e.g. histograms with uniform distributions and a fixed number of intervals) to separately describe precipitation occurrence and volume and the length of wet and dry spells (Sunyer et al., 2012).

Poisson-cluster based models which have been extensively developed and evaluated over the last 25 years offer a plausible physically based approach to stochastic rainfall modelling (Onof and Arnbjerg-
Nielsen, 2009). Such models assume that any rainfall event is triggered by arriving ‘storm origins’ that generate a sequence of ‘rain cells’ clustered using rectangular pulses (Butler and Davies, 2011). Rectangular pulse models generally assume that each storm origin arrives and generates a random number of rain cells according to statistical Poisson processes. Clustering of the rain cells is accomplished using either the Bartlet Lewis Rectangular Pulse (BLRP) or the Neyman-Scott Rectangular Pulse (NSRP) models (Butler and Davies, 2011; Kilsby et al., 2007; Onof et al., 2000).

### 3.2.1 Bartlett-Lewis Rectangular Pulse Model

The Bartlett-Lewis Rectangular Pulse (BLRP) model assumes that each storm arrive in a Poisson process with rate $\lambda$, and that within each storm, cells arrive according to another Poisson process with a rate $\beta$, and the duration of activity of the storm is a random variable. Each cell is assigned a random depth and duration and the total rainfall at time, $t$ is the sum of the contributions of all the cells alive at that time. The duration of the activity of the storm is exponential (parameter, $\gamma$) and the number of cells has a geometric distribution, $\mu_c = 1+\beta/\gamma$. The cell depth and duration are also exponentially distributed with parameters $1/\mu_d$ and $\eta$ respectively and all model parameters are mutually independent (Kilsby et al., 2007; Onof & Ambjerg-Nielsen, 2009; Onof et al., 2000).

Butler et al., (2007) applied Balerep, a six parameter BLRP model to the study the impacts of climate change on storm sewer tank design. The model was used to downscale results of rainfall data sets generated by Hadley RCM for 10 year control (1980 – 1990) and future (2080 – 2090) periods. The results of the study indicated a 35% increase in the number of storm events that fill the tank and a 57% increase in the required average storage volume. Segond et al. (2007) used a statistical multi-site Generalized Linear Model combined with a six parameter BLRP model, a disaggregation model and inverse distance weighting function to downscale multi-site daily rainfall to hourly time series for Dalmuir, UK. Although some bias was detected in the proportion of dry day results, the simulation generally preserved the rainfall properties of the observed statistics. Onof and Arnbjerg-Nielsen, (2009) used an 8 parameter BLRP model in combination with a multi-scaling disaggregator to downscale rainfall data from an RCM model for Holbaek, Denmark. The parameters were fitted using the generalised method of moments for both observed rainfall time series and RCM model results on a monthly basis. The results of the study indicated an increase of between 2% and 15% in extreme rainfall in Holbaek, in the next 80 years (Onof and Arnbjerg-Nielsen, 2009).

### 3.2.2 Neyman-Scott Rectangular Pulse Model

The Neyman-Scott Rectangular Pulse (NSRP) Model is based on similar assumptions as the BLRP model. The following differences between the two models can be identified. Unlike the BLRP model, rain cells within each storm in the NSRP model arrive randomly according to a geometric or Poisson process with mean, $\mu_c$ and the number of cell arrivals are independent and identically distributed around the storm centre. The intensity of each rain cell is exponentially distributed with parameter $\xi$ and is equal to the sum of the intensities of all active cells at that instant. The NSRP model ably represents changes in extreme rainfall amounts for both single site and multi-site applications, explicitly represents skewness of extreme rainfall events and is capable of producing high resolution rainfall time series of arbitrary lengths (Onof et al., 2000; Sunyer et al., 2012; Willems, Olsson, et al., 2012).

The NSRP model parameters can be estimated by minimizing the weighted sum of squared differences followed by optimisation, validation and temporal downscaling of the simulation results (Kilsby et al., 2007). The model has undergone significant development and currently forms the basis for standard UK urban drainage design software (Jones et al., 2007). In the recent UK climate projections report (UKCP09), a weather generator based on the NSRP model was applied in combination with monthly change factors to simulate synthetic rainfall time series with a 5 km spatial resolution for the UK (Jones et al., 2009). The requirement of an adequately long observed time series data set for use in model parameter estimation, fitting and validation is the main limitation of this approach (Fowler et al., 2007).

#### 4. APPLICATIONS IN URBAN AREAS IN TROPICAL DEVELOPING COUNTRIES
Most climate change impacts and adaptation studies employing statistical downscaling have been carried out using case study cities in temperate and mid-latitude regions (Chen and Djordjević, 2012; Olsson et al., 2012; Semadeni-Davies et al., 2008) and only a few studies of a similar nature have been carried out using case study areas in semi-arid or tropical regions (Fowler et al., 2007; Wilby et al., 2004). A recent impact study on a city scale applied results of a coupled climate model (ECHAM5/MPI-OM) to assess the impacts of climate change on urban water supply and drainage infrastructure in Khulna city, Bangladesh (ADB, 2011). Other recent studies in tropical developing countries were not focused on a city scale but a regional scale and included Cowden et al., (2008) who applied both a first order Markov Chain and a stochastic weather generator (LARS-WG) to assess the potential for domestic rainwater harvesting in West Africa and Kigobe et al., (2011) who developed and applied a stochastic rainfall model based on the Generalised Linear Modelling (GLM) approach to infill and extend and historical rainfall data sets in Uganda.

In all these studies closely related issues that limit the application of statistical downscaling in urban areas in tropical developing countries can be identified. First of all, most statistical downscaling methods use results of global and regional climate models and therefore are limited by their resource (i.e. people, time and computational) intensity, uncertainties cascading from the parent models and less reliability in regions where local convective processes greatly influence local climate (Cowden et al., 2008; Pouget et al., 2011; Willems, Olsson, et al., 2012). Secondly, statistical downscaling require long observed rainfall times series data (up to 30 years) with comparable spatial-temporal resolution as the regional climate model results (Willems, Olsson, et al., 2012). However, most urban areas in tropical developing countries have limited or incomplete observed climate data sets which is attributed to sparse gauge networks, limited or no automation of weather stations, equipment down time, funding challenges and operator absence among others (Cowden et al., 2008). Thirdly, existing research and commercial stochastic rainfall models have been developed for application in temperate climates which presents a considerable challenge in adapting the models to other climatic regions (Semadeni-Davies et al., 2008; Willems, Arnbjerg-Nielsen, et al., 2012).

Consequently, climate impact and adaptation studies in cities in tropical developing countries require context appropriate tools and methodologies. Cowden et al., (2008) and Wilby et al., (2004) argue in favour of less sophisticated statistical downscaling techniques such as simple Markov Chain models and semi-empirical models (e.g. LARS-WG) due to their limited input data requirements, fast computations and ease of use. Other approaches with modest resource requirements include spatial interpolation of grid point data, climate sensitivity analysis of impact models, construction of climate analogues using historical data (Wilby et al., 2004, 2009). The delta change approach could also be favourable in the case of availability of reliable observed and climate model data sets such as those provided by Climate Information Portal hosted by the Climate Systems Analysis Group (CSAG, 2013) and the CORDEX Africa experiments (Hernández-Díaz et al., 2012)

Statistical downscaling techniques can generally be categorised as top-down (scenario led) in nature. An emerging and promising approach that could be suitable for application in cities in tropical developing countries is the use of resilience based approaches which combine elements of both top down and bottom up approaches for decision making under uncertain future conditions (Gersonius, 2012; Wilby and Dessai, 2010). Resilience based approaches such as the robust adaptation framework (Wilby and Dessai, 2010) and adaptive policy making (Haasnoot et al., 2013) among others do not entirely rely on climate model projections but also incorporate acceptability thresholds predefined by key flood risk management stakeholders.

5. CONCLUSIONS

Statistical downscaling techniques offer a suitable approach to estimate changes in extreme rainfall events at high spatial-temporal resolution. The above challenges notwithstanding, simple and straightforward statistical downscaling techniques that include climate impact sensitivity analyses, use of simple Markov chain models, construction of climate analogues and spatial interpolation of grid point data are appropriate for use in urban drainage and flood risk management studies for cities in tropical developing countries. The general applicability of statistical downscaling could be improved through measures aimed at increasing availability and dissemination of pre-processed regional climate model data, increased stakeholder engagement and through development of suitable guidelines and decision support tools to guide selection and matching of available methodologies to the requirements of specific impact studies (Fowler et al., 2007; Wilby et al., 2009). From this review, the need to develop
resilience based approaches for application in impacts and adaptation studies in cities in tropical developing countries is also evident.

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