WATEFCON2022

Water Resources Resilience for Small Island Developing States (SIDS)

CONFERENCE PROCEEDINGS

Thematic sessions:

- Water Security Challenges & Strategies in the 21st Century
- Flooding and Resilience against Climate Change

7th Water Efficiency Conference
14-16 December 2022

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PREFACE

The Water Efficiency Conference 2022 will, for the first time, be held outside Europe in the Caribbean. This is because, true to the network's enduring ethos, we continue to advocate for collaborative solutions to the water and resilience challenges faced globally and experienced more intensely by Small Island Developing States (SIDS). These challenges reflect the changing climate, increasingly unpredictable weather, and the efforts towards sustainable development necessary for social equity and economic growth. Governments across the Caribbean are juggling timely response to disasters and other environmental shocks, with effective service and infrastructure programs that sustains the health, social, economic, anthropological, and ecological wellbeing of their societies. Proactive partnerships and collaborations across civil society is necessary to succeed in this effort.

I am often asked questions such as: if water on earth is finite and has been the case for millions of years, why is it abundant in some parts, and scarce in others? Why is safe, affordable, accessible water still a challenge for many? Why is there an increased risk of adverse hydrological events globally? If we know of climate change, why are the many people exposed to such risks still without the capacity to mitigate and adapt to these risks in and for their own circumstances and context? These questions manifest daily in our own lived experiences: what we see, hear, breath, eat and do. These are valid, yet complex questions to answer, but the world needs answers.

A conference on water efficiency and resilience is justified in the face of these challenges and during an energy, cost-of-living, food, and other crises. In an increasingly uncertain world, water is constant and central to most things that matter to us all: the economy, energy, transport, agriculture, health, leisure, wellbeing, social and cultural life. A more equitable world needs adjustments to our water activities as well as our approach to managing our water in nature. It is therefore our privilege as academics, practitioners, and policy makers, to be able to contribute, in our own unique ways, solutions to these fundamental questions. It is with this hope and encouragement that I am grateful for the participation of all the delegates at this conference, whether attending in person or online. Our presentations, deliberations, ideas, and proposals at this year's Water Efficiency Conference moves us closer to our goal for a better water world.

I would like to use this opportunity to thank my friend and colleague, Dr Kiran Tota-Maharaj, for an interesting and engaging conference programme. I acknowledge and appreciate our hosts at the University of the West Indies, Trinidad, especially Prof Edwin Ekwue and Dr Vincent Cooper, and the Engineering Institute. Special thanks to Engr Anthony Chadee and our sponsors including the Water and Sewerage Authority of Trinidad and Tobago. Thank you to our keynote speakers: Profs. Maya Trotz, Michelle Mycoo, and Sarah Hainsworth; and our august and exceptional special guest speakers from across the stakeholder spectrum. The British High Commissioner Harriet Cross and her staff for hosting the welcome reception, and the Minister of Public Utilities, the Hon. Marvin Gonzales, MP. Minister of Public Utilities, Trinidad and Tobago.

Together, we have, and will continue to inspire current and future generations to the levels of creativity and innovation needed to, collectively, and positively, transition in our changing world.

Dr Kemi Adeyeye
Water Efficiency Network Lead
University of Bath, UK.

EDITORIAL

A very warm Caribbean welcome to all of you at the Water Efficiency Conference 2022 (WATEF 2022), Faculty of Engineering, University of the West Indies, Saint Augustine Campus, Trinidad. This Water Engineering, Scientific and Technical conference is intended to provide a forum of practitioners, academics, engineers, scientists and researchers across various universities, research centres and industry to present their works, share their experiences and knowledge exchange focusing on water and the environment. Professionals across the Water, Wastewater and Environmental industries and various sectors need to stay on the cutting edge to face the challenges of the future.

For over 10 years, WATEF has been at the forefront of water conferences with specific focus and engagement on engineering education at the undergraduate, postgraduate, and professional level. WATEF has been and continues to be a platform for educators in this field, exploring challenges, sharing experiences, discussing approaches, and generating new impulses for education and training in the water sector. A special emphasis has always been on the transfer of knowledge and pedagogical paradigms between academia and industry. It is a wonderful opportunity for a Caribbean Small Island Developing State (SIDS) such as Trinidad and Tobago to be centred around such a truly international interaction and event, focusing on sustainability, resilience and climate change impacting on water resources. In the past 15 years, the demands on water infrastructure facing Caribbean SIDS have increased considerably. Little did we know, just how pertinent our chosen conference motto "Water Resources Resilience for Small Island Developing States (SIDS)" were to become for this event with Trinidad and Tobago being best suited for this conference, geographically as well as technically. This dynamic twin-island republic has a relatively high reliable water infrastructure (including desalination plants) for is population but in recent times water resources have been impacted by unforeseen climate change events.

As conference chair, I am blessed with an astute organising team, who worked tirelessly over the past two (2) years to make this conference a successful event. We sincerely thank the Faculty of Engineering at the University of the West Indies, St. Augustine Campus for their support in holding this event in Trinidad and their representatives and academic staff for engaging in this conference.

We trust you will enjoy this conference in the Caribbean as well as your stay in Trinidad and Tobago, West Indies. With your engagement and participation, we believe that WATEF 2022 will become a great milestone in the history of the University of the West Indies and the country of Trinidad and Tobago, West Indies. We do hope you will find this conference stimulating and enjoyable. WATEF 2022 offers opportunities to renew old friendships and to make new collaborations. Have a great and memorable time in the Caribbean.

Dr Kiran Tota-Maharaj
Conference Chair WATEF 2022

SCIENTIFIC COMMITTEE

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WELCOME ADDRESS

Small island developing state (SIDS) like Trinidad & Tobago are vulnerable to climate change and changing weather conditions. The increasing frequency and severity of weather events such as flooding, tropical storms, slow onset sea level rise, land erosion have severe impact to people's health, wellbeing, livelihoods and the resulting damage impacts human and natural ecology. Caribbean countries including Trinidad & Tobago also experience water stress and face challenges in water security. Without proactive action, these challenges to Caribbean SIDS will adversely affect economic activity such as agriculture and tourism and increase infrastructure costs. The UK recognises these challenges faced by SIDS and values collaborative and partnership working to devise proactive solutions. The UK works in partnership with Caribbean nations to develop and deliver strategies to improve water and climate resilience. You will hear more about this during the special session on 'Flooding and resilience against climate change'. We are also proud to support knowledge and research exchange activities by groups and organisations like the Water Efficiency Network, University of Bath. My team look forward to meeting you and hearing more about your water and resilience activities at the welcome reception at my Residence. I also warmly welcome you to the Water Efficiency Conference 2022 at the University of the West Indies, St Augustine, Trinidad.

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KEYNOTE SPEAKERS

Prof. Michelle Mycoo. The University of the West Indies (UWI), Trinidad and Tobago

Professor Michelle Mycoo is Professor of Urban and Regional Planning at UWI, St. Augustine, she has conducted extensive research and training on water management, climate change, disaster risk reduction and urban planning. She is currently the Coordinating Lead Author of the Small Islands Chapter of the Intergovernmental Panel on Climate Change Sixth Assessment Report, Working Group II, is a member of the International Science Council, the Scientific Advisory Committee of UNESCO's Management of Social Transformations. She has conducted capacity building workshops for UNDP's Global Water Partnership and holds a PhD (McGill University), MSc in Urban Planning (The University of Hong Kong) and a BA in Geography and Social Sciences (UWI).

Prof. Maya Trotz. University of South Florida, USA

Prof. Maya Trotz is a Guyanese born professor of Civil and Environmental Engineering at the University of South Florida. She directs STRONG Coasts, a National Research Traineeship program to foster food, energy, and water solutions with coastal communities, and leads the knowledge management component of a Green Climate Fund project, "Water Sector Resilience Nexus for Sustainability in Barbados." She is a past President of the Association of Environmental Engineering & Science Professors and a board member of Fragments of Hope Corp, a coral restoration NGO in Belize. She holds a BS in Chemical Engineering from MIT, and MS and Ph.D. degrees in Environmental Engineering from Stanford University.

Prof. Sarah Hainsworth. The University of Bath, UK

Professor Sarah Hainsworth OBE CEng FIMMM FREng CSci is Pro-Vice-Chancellor (Research) at the University of Bath, UK and is a Professor of Materials and Forensic Engineering. Sarah is a Fellow of the Royal Academy of Engineering, a Fellow of the Institute of Materials, Minerals, and Mining, and a Fellow of the Women's Engineering Society. She was awarded an OBE in 2019 for her services to engineering and forensic sciences. Sarah is strongly committed to research and teaching in sustainability and the circular economy throughout several research institutes. Sarah holds a BEng (Hons) in Science of Engineering Materials and a PhD (both University of Newcastle upon Tyne).

SPECIAL SESSIONS

Special session 1: Flooding and resilience to climate change

Flooding in Trinidad and Tobago, and indeed across the Caribbean, is occurring more frequently, with greater magnitudes, and with increasing scale of damage. Flood risk management approaches based solely on flood defence works would prove to be inadequate unless they are complemented with other strategies such as flood prevention, property floodproofing, preparation and recovery. Such a diverse set of strategies would strengthen resilience even from the short term when flood defence works are still being planned. The presentation by the Director of Drainage describes the plans by the government to improve flood defence measures, and the time frame for their completion. In the intervening time, several sectors of the society would remain vulnerable to flooding and so it is important to develop these other strategies that would achieve an acceptable flood risk level. The presentation by the ODPM is on the status of the governance system for flood risk management, involving flood hazard mapping for land zoning, early flood warning systems for reducing losses and for evacuation and the requirement for floodproofing at the household level. The presentation by Pat Shako, the UK FCDO (Caribbean region) will highlight the role and activities of development partnerships to support resilience efforts. The presentation by Col Dave Williams is on the growing awareness of households to take charge to protect their property using some of the products that are fast becoming available in the country. The discussion will be guided by a panel of experts who will be able to respond to the entire range of strategies to be employed in a healthy flood risk management system.

Speakers:

Katherine Badloo-Doerga	Director, **Drainage Division, Ministry of Works and Transport, Trinidad**	**Topic:** The national drainage plan

Mrs. Doerga has worked with the Ministry for the past eighteen (18) years, in three (3) major Divisions, ten (10) years in the Construction Division, seven (7) years as Chief Planning Engineer in the Highways Division, and currently as the aforementioned. Mrs. Badloo Doerga graduated from the University of the West Indies, Trinidad with a B.Sc. (Hons) in Civil Engineering and holds a master's degree in construction engineering and Management. Katherine is a registered Engineer with the Board of Engineering as well as a Member of the Association of Professional Engineers of Trinidad and Tobago (APETT). The Director has served on multiple committees ranging from Disaster Prevention and Preparedness to Structural Assessments and Retrofitting, as well as the Community Flood Early Warning System on behalf of the Ministry.

Anwar Baksh	Planning and Developing Office, **Office of Disaster preparedness and Management, Trinidad and Tobago**	**Topic:** Flood risk management governance

Anwar Baksh presently serves as the Planning and Development Officer and lead of the Mitigation, Planning and Research Unit at the Office of Disaster Preparedness and Management Trinidad and Tobago. Anwar's role has presented him with great opportunities to utilize his full cadre of skills to promote disaster risk reduction and management with Trinidad and Tobago and by extension the Caribbean Region. His expertise lies in communications and networking where technical capacity meets human interaction. Critical to his function is the focus on the advancement of the

Governance framework for Disaster Risk Reduction and Management in Trinidad and Tobago in support to the implementation of the Sendai Framework for Disaster Risk Reduction (2015-2030). To this end, Anwar has worked in several areas of programmatic areas: Early Warning Systems, Recovery Planning, strategic planning, partnership and resilience building and several other areas to support the national DRRM agenda of Trinidad and Tobago. Through his work, Anwar has been able to strengthen and forge partnerships across the regional and global landscape with the aim of strengthening the resilience our country and the region. With the support of those around him, he is determined and committed to disaster management and loss prevention in the face of climate change for the small island developing state of Trinidad and Tobago.

Fazir Khan	Managing Director, **Alpha Engineering & Design (2012) Ltd.**	**Topic:** Flooding resilience

Fazir Khan has over 30 years' experience as a practicing professional Civil Engineer and Project Manager, working locally and within the Caribbean Basin. He graduated from the University of the West Indies, Trinidad with a B.Sc. (Hons) in Civil Engineering. He has obtained a diploma in Management for the Henley University and certificates in Project Risk Management, Project Cost and Schedule Management, and Contracting and Negotiation Skills from the Arthur Lok Jack Graduate School. Fazir is a certified A+ modeler for Detention Pond Systems; Urban Storm Water Management; and Hydrological Modeling for sustainable solutions on large sites. He has a certificate in Integrated Coastal Zone Management from IHE Delft University. Apart from the Association of Professional Engineers of Trinidad and Tobago (APETT) where he was a Past President (2016/2017), he is also a registered member of the Board of Engineering of Trinidad and Tobago and the British Hydrological Society, England. During his 21 years with Alpha, Fazir has worked extensively on large integrated developments, infrastructure designs, water and wastewater projects and construction activities for various public sector and private Clients. Previous to his employment with Alpha, he worked at Trinidad Contractors Limited for 5 years as a Project Engineer, assigned to several engineering and construction projects within the Caribbean region.

Pat Shako	Climate & Disaster Resilience Adviser, **UK Foreign, Commonwealth & Development Office (Caribbean)**	**Topic:** Development and resilience

Pat Shako joined the Department for International Development (DFID) Caribbean - now UK FCDO in 2018 as the Climate and Disaster Resilience Adviser. She currently helps to conceptualise and oversee the implementation of a portfolio of projects involved in building resilience and disaster risk reduction support for 7 ODA eligible countries across the Caribbean. Pat also provides technical support to the UK on policy and political climate change issues for an extra 5 Caribbean countries. Work areas focus on climate change adaptation across multiple sectors including incorporating resilience systems into public financial management and disaster management systems (including social protection systems); ecosystem-based solutions for resilience; renewable energy; as well as blue economy and disaster risk finance instruments for small island developing states. Prior to this role, Pat has worked as staff with the World Bank, the Inter-American Development Bank (IDB), the Caribbean Development Bank (CDB), and has provided leadership on conservation and regional water issues during her positions with Global Water Partnership – Caribbean, and The Nature Conservancy. Pat holds a M.Sc. in Natural Resource Economics, and a MBA Finance and has worked across the Caribbean for over 25 years.

Dave Williams	President, **Emergency Management Agency of Trinidad and Tobago**	**Topic:** Property-level floodproofing

Colonel Williams served as a commissioned officer in the Trinidad and Tobago Defense Force for thirty-four years, retiring in April 2009. With respect to disaster risk reduction, he has been in the business for just over twenty years to date, amassing experience in directing response operations, leading rapid needs assessment teams, and adapting, developing and teaching disaster management subjects. He spent seven years as Director of the now-defunct National Emergency Management Agency (now re-incarnated as the ODPM) and two years (August 2010 –August 2012) as the Chief Disaster Management Coordinator at the Ministry of Local Government. Most recently, he held the appointment of Deputy CEO at the ODPM for a brief period of 10 ten months.

Discussion/ Q&A
Panelists:

Dr Vincent Cooper	University of West Indies, Trinidad and Tobago
Ms Stacey-Ann Pi Osoria	PODS - Emergency management consultancy and solutions

Special Session 2 (Industry Day): Water security challenges and strategies in the 21st century

Chair: *Engr Anthony Chadee*

According to climate model predictions, the Caribbean is facing a very uncertain future on providing the water for its development. In the first session of this special session Dr. Cashman explores the predicted changes in water availability and what this might mean for Caribbean development. In the meantime, expectations for growing the economy may increase water demands in various sectors. The presentation by WASA, Mr Lord and Dr Sammy explores the technical innovations and projections for the future, focusing specifically on solutions for effective water demand management including for major water consumers in Trinidad and Tobago. In the final paper, Mr. Meade from the Water Resources Agency considers these supply and demand predictions and discusses strategies that need to be fully developed for achieving water security. The talks are followed by a discussion with a panel of experts.

Speakers:

Adrian Cashman	**Consultant, AKWATIX Water Resources Management**	**Topic:** Water bombs and the future of water

Dr Adrian Cashman has over 40 years of experience in the water sector. He has been working in the Caribbean, based in Barbados for the past 15 years, first with the University of the West Indies and now as an international water resources management consultant. He has worked on numerous research and consultancy projects across the Caribbean. Prior to 2018, he spent 12 years with the University of the West Indies and served as the Director of the Centre for Resource Management and Environmental Studies (CERMES), training and mentoring many postgraduate students who have gone on to play important roles in the water sector across the Region. Dr Cashman's published works cover a diverse range of fields including critical accounting, geography, water and climate change, water policy, resource management and future studies. He has worked with a wide range of international and regional organisations on water and climate related matters. Up until 2022 he was a member of Global Water Partnerships Technical Advisory Committee and continues to serve the Caribbean as Deputy Chair of the Global Water Partnership Caribbean's Technical Committee. In 2020, he received the Caribbean Water and Wastewater Associations Gold Award for services to the Caribbean water sector.

Kennedy Lord	**Manager,** Seven Seas Water Group, Trinidad	**Topic:** Desalination, the impact to Trinidad and Tobago Water Sector

Kennedy Lord is the Plant Manager at Seven Seas Water Group Point Fortin Trinidad. He holds a BSc Mechanical Engineering Technology specializing in Manufacturing from the University of Houston and a MBA Management from Florida Institute of Technology. He along with his Team has guided the operations of the 6.7 million gallons per day Desalination Plant at Point Fortin Trinidad to achieve all Operations Performance KPI over the last 36 months. Mr. Lord worked in heavy Manufacturing as the Production Superintendent and Manufacturing System Specialist before joining Seven Seas Water. He has published articles for the International Cement Review and the Florida Tech Magazine. He is Married and father of two adult children, when he is not engaged in assisting to improve his community, he is either involved in Fishing, Hiking or Golfing.

Keith Meade	**Director,** Water Resources Agency	**Topic:** Strategies for water security in the future

Keith Meade is a water resources management professional with over 20 years' experience in the water sector. He has practical experience in hydrology, water resources assessments and planning, strategic business management, disaster management, environmental management and forest management. At present, Keith Meade is a Senior Manager at the Water and Sewerage Authority in Trinidad and Tobago and heads its Water Resources Agency. Keith possesses a MSc. In Water Resources Management and a BSC. Forestry from the University of Michigan, USA. Keith has conducted independent research on the impact of climate change on the water resources of the Great Lakes, and runoff patterns under various land-use practices in the Northern Range of Trinidad. He is presently involved in an Inter-Agency partnership which is establishing a Community Flood Early Warning System, and in the expansion and modernization of the hydrological monitoring network of Trinidad and Tobago. Mr. Meade spends time in fitness and faith-based activities.

George Sammy	**CEO,** EcoEngineering Consultants Limited, Trinidad	**Topic:** Water engineering

Dr. George Sammy is an Environmental Engineer based in Trinidad and Tobago and offering environmental consultancy services throughout the West Indies. He earned a doctorate in Engineering from the University of Oklahoma in 1983, and his expertise in Environmental Impact Assessment is internationally recognized. In July 2021, Dr Sammy was lead author of "A Handbook for Environmental Impact Assessment Practitioners in the Organization of Eastern Caribbean States". He has also developed and presented a large number of EIA training courses as part of university degrees and as stand-alone courses. Dr. Sammy's experience with water supply projects has spanned many decades. He has worked on projects such as the Arena Dam and Tumpuna Weir, the Northern Range Valleys Projects and the intake for the original North Oropouche River Project. He was initially the Assistant Resident Engineer and subsequently Resident Engineer on the construction of the Arena Dam and Tumpuna Weir. In 1983, he was assigned the Construction Manager of the entire Caroni-Arena Water Supply Project. Between 1985 and 1986, Dr. Sammy was the Project Manager on the design of the Point Fortin Water Supply Project. George's Environmental Engineering experience covers a wide range of project types including water supply pipelines in Trinidad and Water Supply Projects in St. Lucia, Tobago and Dominica.

Discussion/ Q&A Panelists:

Mr Frankie Balkissoon	Director, Engineering Division, Ministry of Agriculture
Ms Candice Santana	Caribbean Water and Wastewater Association

Contents

#1. Flooding and resilience

Optimal Design Storm Frequency for Flood Mitigation

Vishwanath Maraj[1*], Vincent Cooper[2], Matthew Wilson[3]

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[3] Geospatial Research Institute, University of Canterbury, New Zealand

ABSTRACT
Determining the right amount of money or budget to be spent on flood mitigation works has always been a challenge in developing countries. Recent work has identified a recommended budget based on the value accepted for flood mitigation works in developed countries, measured as the Cost per Inhabitant. In addition to this approach, the industry also utilizes the hydroeconomic analysis (HEA) to determine an optimal design storm frequency that yields the most economical budget and design approach to flood mitigation works. This study investigates both economic approaches to determine the level of protection or the optimal design storm frequency for flood mitigation works. These economic approaches were executed for flood mitigation works within the North Valsayn community of Trinidad. To facilitate these economic assessments in determining the optimal design storm frequency, flood hazards were identified using a calibrated 2D hydraulic model done in LIS-FLOODFP. A Flood Damage Curve was used as a measure of the community's vulnerability using data collected from social surveys. Flood mitigation works were identified for the various design storm frequencies and the associated life cycle costs were determined. Upon execution of both economic approaches, the HEA indicated that it is most economical to maintain the existing flood condition as the annual cost of mitigation works far outweighs the annual damage cost. On the contrary, when implementing the Cost per Inhabitant approach, flood mitigation works performed for a design storm frequency of 1 in 50years was found to be optimal or comparable to the recommended budget. The study shows a disparity in defining a project's budget and the Optimal Design Storm Frequency using both approaches for decision/policy makers, and various stakeholders although both are acceptable.

1. INTRODUCTION

Over the years, countries have suffered many losses from floods which can be described as monetary and non-monetary or tangible and non-tangible (Institute for Catastrophic Loss Reduction 2008, 98:1-10). Managing the risk of these losses is very important for economic stability of a community and by extension, a country. In doing so, decision makers are faced with determining the hydrologic design level (the design storm frequency) for which infrastructure, people and their properties are to be protected when implementing flood mitigation projects.

A hydrologic design level or hydrologic design storm frequency is the magnitude of the hydrologic event considered for a safe design of a hydraulic structure or project. It is not always economical to design such structures or projects for the worst case or limiting value (Chow et al. 1989, 420-421). Designing for the limiting value may render a high capital and life cycle cost for a project and may not provide equally or greater beneficial savings to the client or country's economy. Therefore, designing for an optimal design storm frequency is important for the execution of projects that make best use of public funds and promote sustainable growth of a country.

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Recent reports such as the *Rehabilitation and Improvement of Drainage Infrastructure in Trinidad* done by Witteveen + Bros in 2019 have justified budgets for flood mitigation works by comparing the cost of the recommended works to the value of same in the Netherlands (a developed country) using the cost per inhabitant per annum as the comparative measure. This approach gives the developing country such as Trinidad and Tobago an idea of the acceptable amount of money to be spent on flood mitigation works. This identified budget can be correlated to the design storm frequency that yields the closest cost for mitigation works and therefore identifies the design storm frequency or level of protection that can be acceptably afforded.

This study investigates the optimal design storm frequency that can be afforded from the Cost per Inhabitant (CPI) approach described above to that which will be derived from executing a Hydroeconomic Analysis (HEA) described by Chow et. Al. 1989 for flood mitigation works within the North Valsayn community of Trinidad. The HEA is feasible for implementation of flood mitigation works if the probabilistic nature of a hydrologic event and the respective damage that will occur are both known over a feasible range of hydrologic events as described by Chow et al. 1989. As the design storm frequency or level of protection for flood mitigation works increases the cost of works (infrastructure) increases but the expected damage decreases. The total sum of the mitigation works and the expected damage cost for a particular design storm frequency can be defined as the total cost to the owner or to the country's economy. Therefore, by estimating the total cost to the owner for each design storm frequency within a feasible range, the optimum design storm frequency having the lowest total cost can be found (Chow et.al., 1989, 423-427). The HEA approach described by Chow et.al (1989) was applied in this study.

For application of the HEA in this study, the costs of damage or damage cost referred is an association to flood risk. Flood risks can be defined in terms of hazard and vulnerability. Flood hazard assesses the intensity and the probability of flood scenarios and vulnerability assesses the direct impacts such as physical loss or damage to material and property. Vulnerability in other instances can also include indirect impacts such as loss in production time in businesses and inflation of prices in the market. (Institute for Catastrophic Loss Reduction 2008, 98 1-2).

Quantifying these flood risks into cost of damage includes estimating both flood hazard and vulnerability. Flood hazards can be defined by representing the physical characteristics of flood events in terms of its frequency, depth and velocities, aided by flood maps developed from models (Cancado et al. 2008, 1-2). When using a modelling approach to estimate a flood's characteristic, one could either utilize one dimensional (1D) or two dimensional (2D) numerical models. The 1D numerical model can be used for simple rivers and streams in which the designer can visualize the expected extents. The 2D numerical models can be used for more complex rivers and streams with multiple structure crossings, significant bends and meanderings with expected complicated flow patterns (State of Florida Department of Transport 2012, 38).

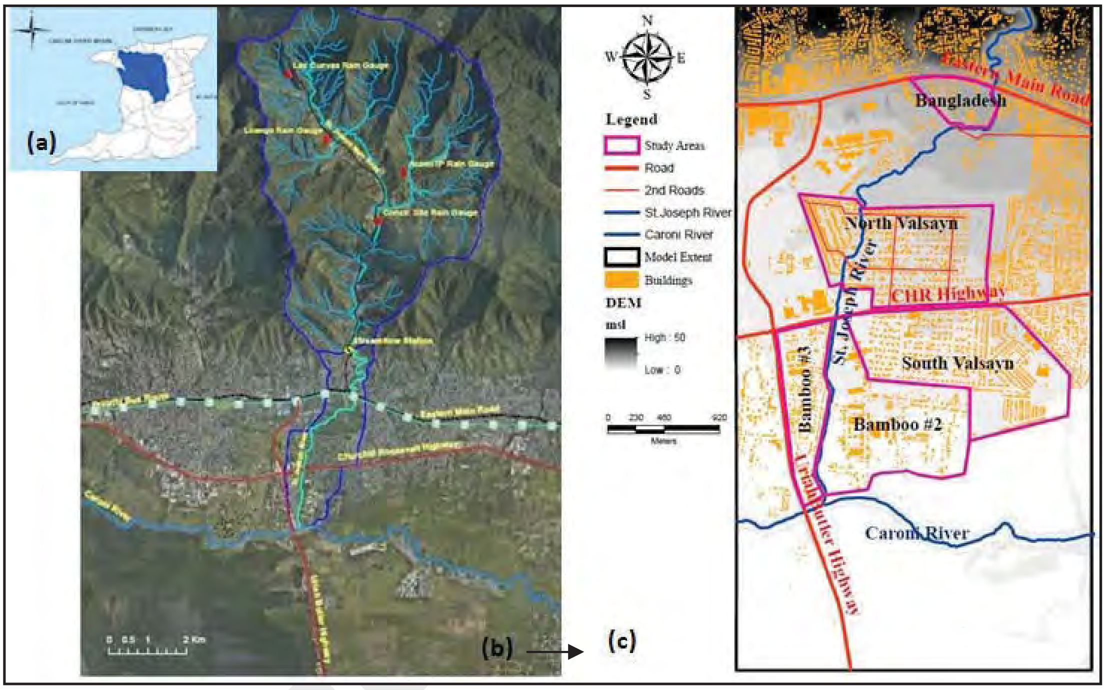
Vulnerability can be measured by assessing the potential damage or impacts that can occur and the recovery capacity of flood prone areas (Cancado et al. 2008, 1-2). However, the assessment of these damage and impacts carries many uncertainties as these are highly dependent on flood depths, velocities, contamination, building characteristics and even by the level of personal flood prevention measures used. Therefore, in most studies, not all of these are usually considered when assessing the damage and impacts of floods (Institute for Catastrophic Loss Reduction 2008, 98:1-2).Flood depth for example is the only variable considered by some authors such as Dutta et al. 2003. The loss estimation model was formulated based on stage-damage relationships for various flood inundation parameters and land use features.

Using similar practices, flood hazard maps using 2D hydraulic models for various designs storm frequencies, the associated damage costs, the mitigation works and their associated costs are identified to be used in the HEA and the CPI assessments in determining the optimal budget and design storm frequency. This has been done for flood mitigation works within the North Valsayn community of Trinidad. The flood mitigation works investigated in this study are: (i) increasing the hydraulic capacities of the adjacent segment of the nearby St.

Joseph River with the introduction of vertical concrete levees, and (ii) increasing the hydraulic capacity of one (1) bridge located within the adjacent river segment.

The North Valsayn community is among several communities located on the foothills of the Northern Range of Trinidad within the St. Joseph River Catchment that experiences frequent riverine flooding. Some of the flood prone areas within this catchment are Bangladesh, North Valsayn, South Valsayn, Bamboo No. 2 and Bamboo No.3. These areas are identified in Figure 1 (c) below for ease of reference.

Figure 1: (a) Caroni Catchment within Trinidad, (b) St. Joseph River Catchment and (c) Map Showing Affected Communities within the Lower Reach of the St. Joseph River



As shown in Figure 1, these communities were developed along the banks of the lower river reach. The St. Joseph River is one of many tributaries to the Caroni River. The Caroni River watershed is the largest in Trinidad and experiences spatially varied rainfall typically moving in a western direction, influenced by the North East Trade Winds. The St. Joseph River discharges into the lower reaches of the Caroni River before it enters into the Caroni Swamp. As such, stream flow in the St. Joseph River frequently encounters high water levels at the Caroni River confluence. This typically results in the St. Joseph River breaching its banks and flooding the areas and communities listed earlier as there is inadequate capacity within the Caroni River to receive the discharge.

The aim of this study is to determine the optimal design storm frequency for flood mitigation works for North Valsayn. The objectives are:

- To perform hydrodynamic modelling to determine the extent of the flooding at various frequencies;
- To estimate the associated costs;
- To apply a hydroeconomic analysis (HEA) and a Cost Per Inhabitant (CPI) analysis to determine the optimum design storm frequency.

2. METHODOLOGY

This section presents the approaches for executing an HEA and a CPI.

2.1 Hydroeconomic Analysis (HEA)

The HEA described by Chow et. al. 1989 was one of the economic approaches used to identify the optimal design storm frequency and budget. The HEA relies on the determination of the *Total Annual Cost* of the flood mitigation works that represents the sum of the Annual

Damage Cost and the Annual Life Cycle Cost (LCC) of the mitigation works. The design storm frequency that yields the lowest Total Annual Cost is identified to be the optimal level of protection for the works. In this study, the HEA described by Chow et al. 1989 was modified to account for the LCC and not just the Capital Cost. In addition, it was also modified to account for the flood risk transferred downstream of the study area caused by the mitigation works and is termed the "Residual Damage." The modification to account for the residual damages/risks was accounted for via the introduction of the columns highlighted below within the original HEA Assessment Table format by Chow et. al. 1989.Refer to Table 1.0 below for the HEA assessment format used:

Table 1.0 HEA Assessment Format

1. Increment (i)	2. Return Period (T) /yrs	3. Annual Exceedance Probability (AEP)	4. Damage (US$)	5. Incremental Expected Damage per Year (US$)	6. Annual Damage (US$)	7. Residual Damage (US$)	8. Incremental Expected Residual Damage per Year (US$)	9. Total Annual Damage = Annual Residual Damage + Col.6 (US$)	10. LCC per Year (US$)	11. Total Cost per Year (US$)
1	t1	100/t1	$D(x_{t1})$			$D_r(x_{t1})$			y_{t1}	
2	t2	100/t2	$D(x_{t2})$	ΔD_1		$D_r(x_{t2})$	ΔDr_1		y_{t2}	
3	t3	100/t3	$D(x_{t3})$	ΔD_2		$D_r(x_{t3})$	ΔDr_2		y_{t3}	
4	t4	100/t4	$D(x_{t4})$	ΔD_3		$D_r(x_{t4})$	ΔDr_3		y_{t4}	
5	t5	100/t5	$D(x_{t5})$	ΔD_4		$D_r(x_{t5})$	ΔDr_4		y_{t5}	
Annual Damage (US$)					Annual Residual Damage (US$)					

Where: t_1 is the Design Storm Frequency, $D(x_{t1})$ and(x_{t1}) are the damage cost and residual damage cost incurred from the respective storm frequency (refer to Section 2.1.1), ΔD_1andΔDr_1 are the damage cost and residual damage cost reduced to annual values, and y_{t1} is the Annual Life Cycle Cost for the flood mitigation works for the respective Design storm Frequency (refer to Section 2.1.2). The Incremental Expected Damage is calculated using the following equation:

$$\Delta D_1 = \left[\frac{D(x_{t1}) + D(x_{t2})}{2}\right]\left[P(x \geq x_{t1}) - P(x \geq x_{t2})\right]$$

Where: $P(x \geq x_{t1})$and$P(x \geq x_{t2})$ are the annual exceedance probabilities for the *t1* year and*t2* year storm frequencies respectively.

2.1.1 Flood Risks or Damage Cost

The Damage Cost was estimated by correlating the expected flood depths expected at the affected homes within the North Valsayn community to a Flood Damage Curve. This was done for each design storm frequency to obtain the corresponding Damage Costs. The flood depths and the number of homes affected were defined using flood hazard maps developed in accordance with Section 2.1.1.1 below. The Flood Damage Curve was developed as a representation of the community's vulnerability to floods and was defined in accordance with Section 2.1.1.2.

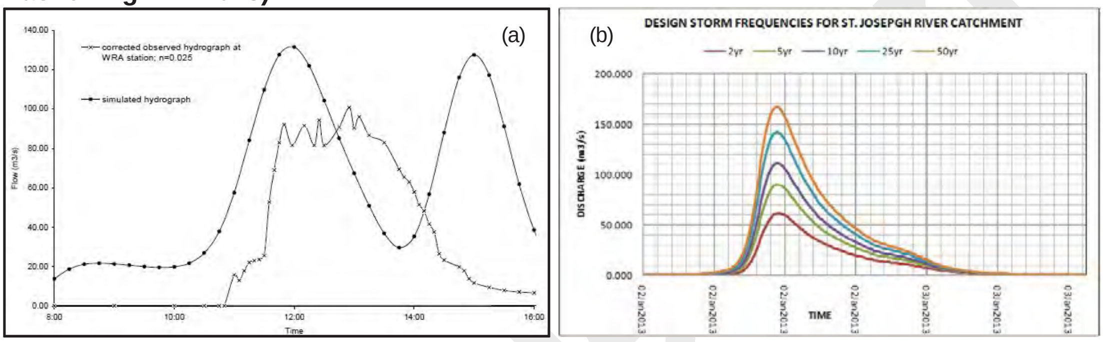
2.1.1.1 Flood Hazard Mapping

Flood Hazard Maps were developed for each design storm frequency (2yr, 5yr, 10yr, 25yr and 50yr) using a calibrated 2D hydraulic model. The hazard maps present the flood depths and flood extents within the affected communities, including North Valsayn. The 2D hydraulic model was developed using the sub-grid version of LISFLOOD-FP developed by Neal et al. (2012) primarily to account for the hydraulic effects of bridges.

A Digital Elevation Model (DEM) was prepared using the 2012 LiDAR data for Trinidad with a 12m resolution to accurately represent the topography of the area under investigation. The DEM was processed such that the St. Joseph River, the Caroni River at the outfall, all road embankments, and all major drainage channels were included and accurately represented. Other hydraulic features such as the pumping station and the bridges within the model's extent were defined within LISFLOOD-FP. The pumping station located south of Bamboo Settlement #3 (Figure 3(c)) was represented as a weir to control the flood discharge from Bamboo Settlement #3 to the Caroni River while simultaneously preventing backflow. Within the model's extent, four bridges were included using the surveyed geometry provided by Royal HaskoningDHV, consultants working in the study area.

Having defined the physical components of the model, the hydrologic and hydraulic boundary conditions were then defined. As stated in the introduction, the St. Joseph River is frequently encountered by high water levels at its discharge point into the Caroni River. This was also reported to be the condition for a large event in Nov 05, 2002 for which a considerable amount of field work had been done to quantify the impacts. Hence the downstream boundary condition was set to represent a constant water level equivalent to a bank full condition of the Caroni River. The upstream boundary condition was set as inflow hydrographs for the calibration process and for the design process. The hydrograph used for the calibration process was that of the Nov 05 2002 flood event and the hydrographs used for the designs were that produced from the Haskoning DHV 2013 Study. These are presented in Figure 2.

Figure 2: (a) Simulated and Observed Hydrographs for November 5th 2002 Flood Event (Cooper 2003, 16), and (b) Hydrographs for Various Design Storm Frequencies (Royal HaskoningDHV 2013)



The calibration of the hydraulic model was done by comparing the observed flood depths to the simulated flood depths for various floodplain and channel roughness coefficients. Observed flood depths were surveyed flood depths within and adjacent to the Nestle Compound, near the Cipriani Labour College and on Farm Road done post the Nov 05 2002 event (See Figure 3). The model was then validated using video footage provided by a local television station (TV6). This footage confirms a common area within the Grand Bazaar compound to be flooded. The validation was limited to flood extent only and not flood depth. Using this calibrated model, flood hazard maps were developed for various storm frequencies using the design hydrographs in Figure 2(b).

2.1.1.2Community Vulnerability – Flood Damage Curve

A questionnaire was developed, and a field survey was conducted within the study area. The survey gathered information on reported flood depths and reported flood damages experienced by each household. The data collected would have shown some disparity due to the individual level of protection adopted by each household, the home design and the valuables exposed to flood waters, and so the median damage cost was found for the flood depth ranges 0 to 0.3m, 0.3 to 0.1m and above 1m. These flood depth ranges were the most frequent ranges reported by the residences in the survey. The median of the damage cost for each flood depth range was then plotted to develop the Flood Damage Curve. This curve was then used to estimate the cost of damages required for input in Column 4 of the HEA shown in Table 1.0.

2.1.2Annual Life Cycle Cost (LCC)

The HEA described by Chow et. al. (1989) focused on the capital cost of the flood mitigation works. This study, however, considers the Life Cycle Cost (LCC) of the works to capture the design and planning costs, construction costs and operational &maintenance costs. The construction costs were estimated using industry rates for the construction of bridges and concrete structures for the levees. The design and planning costs were taken as 10% of the construction costs, as guided by the local engineering practice and associations in Trinidad and Tobago. The operational and maintenance cost was taken to be 1% per annum of the construction cost (Zhang et al. 2008, 7). This LCC was then divided by 50 years (the expected life span of the structures) to obtain the Annual LCC. This was done for the flood mitigation works required for all the design storm frequencies and was input into Column 10 of the HEA table (Refer to Table 1.0 above).

2.1.2.2 Engineering Designs of Flood Mitigation Works

The flood mitigation works investigated in this study are: (i) increasing the hydraulic capacities of the adjacent segment of the St. Joseph River with the introduction of vertical concrete levees, and (ii) increasing the hydraulic capacity of the one (1) bridge located within the adjacent river segment on the Mayfield Road (Figure 3(b)). The calibrated hydraulic model was used as the design tool to determine the bridge size and the levee heights required to mitigate floods within the North Valsayn community for each design storm frequency.

2.2 Cost per Inhabitant (CPI) Approach

This study utilizes this approach to identify an acceptable project budget benchmarked to a developed country. In the Witteveen + Bros (2019) report, a Cost per Inhabitant per annum of US$90 was used as a benchmark for assessing an acceptable budget for flood mitigation works. This was the annual value per inhabitant for flood mitigation works within the Netherlands.

Since the flood mitigation works are related to North Valsayn only, an "acceptable or recommended budget" was identified using US$90 per inhabitant per year for the total estimated population of the community. The population of North Valsayn was taken as four (4) inhabitants per household identified in Figure 1.0(c). The budget obtained from this approach was developed independent of the design storm frequency or level of protection that the works can provide. However, the type of works and hence the level of protection is dictated by this budget. Hence, this budget was then compared to the various Annual LCCs for the flood mitigation works designed for various design storm frequencies to identify the level of protection that the policy holder or decision maker can afford.

3. RESULTS AND DISCUSSION

The following sections present the result from each analysis in the order of input into the HEA and Cost per Inhabitant (CPI) approach.

3.1 Flood Model, Hazard Mapping and Mitigation Works

A calibrated hydraulic model was successfully identified for the November 05, 2022 storm event using a floodplain friction coefficient and a channel friction coefficient of 0.065 and 0.025, respectively. Figure 3(a)(i) shows no flooding at one of the observed flood points located adjacent to the Farm Road bridge. This may have been attributed to localize urban flooding in reality versus the riverine flooding which was primarily modeled. This observed data point was located close to the upper boundary of the model extent and approximately 1km away from our site of interest (North Valsayn). The model, however, showed very good response when compared to the other four calibration points located north of the CRH Bridge which are located within our site of interest, as shown in Figure 3(a)(ii). The model also confirmed flooding at Grand Bazaar (Refer to Figure 3(a)(iii)) and hence compared well with the TV6 television station's footage for the validation process.

This calibrated model was then used to simulate the flood hazard maps for the various design storm frequencies. Figure 4(a) shows an overlay of the flood extents for each design storm frequency. These models were used to increase the size of the bridge on Mayfield Road which was found to have inadequate hydraulic capacity for the design storms. The hydraulic capacity of the CRH bridge was found to be adequate. However, it was noted that the Mayfield and the CRH bridge heights and connecting roads were higher than the surrounding lands which led to the river channel being breached prior to the full hydraulic depth of the bridges are used. Therefore, levees were introduced along the riverbanks between these bridges as shown in Figure 4(b) to utilize the full hydraulic depth of the bridges. The levees were conceptualized as reinforced concrete and not earthen due to the limited space between the riverbanks and private properties. Refer to Figure 4(c) below for a summary of the design structures for the mitigation works. The Life Cycle Cost (LCC) of the mitigation works are presented below in Table 2.0.

Figure 3: (a) Calibration and Validation Data (Observed) Point Locations, (b) Simulated Nov' 05 2002 Flood Map

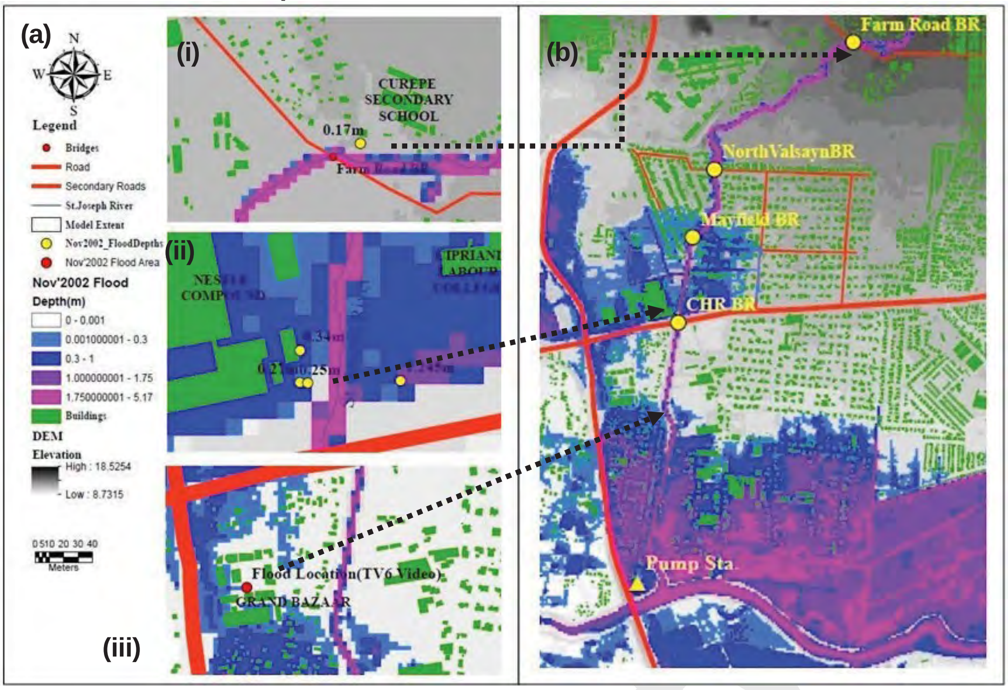
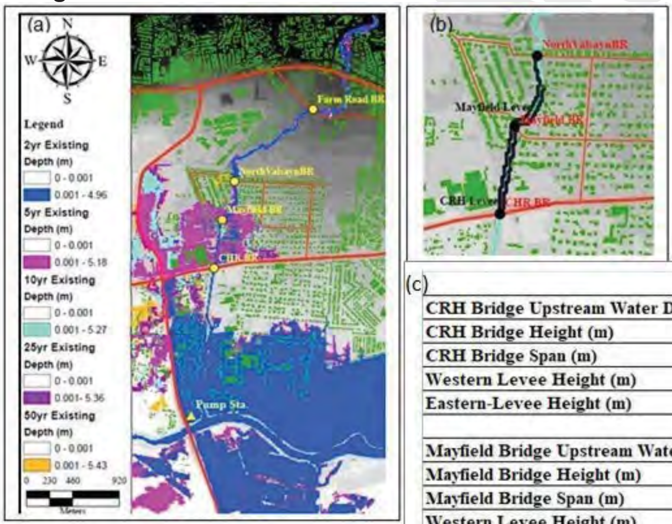


Figure 4: (a) An Overlay of the Flood Hazard Maps for various Design Storm Frequencies, (b) Extent of Levee on Western and Eastern sides of the St. Joseph River Banks between the CHR Bridge and the Mayfield Bridge, and (c) Summary Table of Design Structures



(c)

	2yr	5yr	10yr	25yr	50yr
CRH Bridge Upstream Water Depth (m)	3.04	3.5	3.8	4.14	4.32
CRH Bridge Height (m)	Existing Bridge Span (29.5m) and Depth (6.6m) are adequate				
CRH Bridge Span (m)					
Western Levee Height (m)	N/A	2.04	2.44	2.44	2.44
Eastern-Levee Height (m)	N/A	1.87	2.27	2.27	2.27
Mayfield Bridge Upstream Water Depth (m)	3.42	3.96	4.27	4.5	4.55
Mayfield Bridge Height (m)	4.2	4.2	4.2	4.2	4.7
Mayfield Bridge Span (m)	13.5	22.6	22.6	22.6	22.6
Western Levee Height (m)	N/A	1.34	1.54	1.94	2.34
Eastern-Levee Height (m)	N/A	1.29	1.49	1.89	2.29

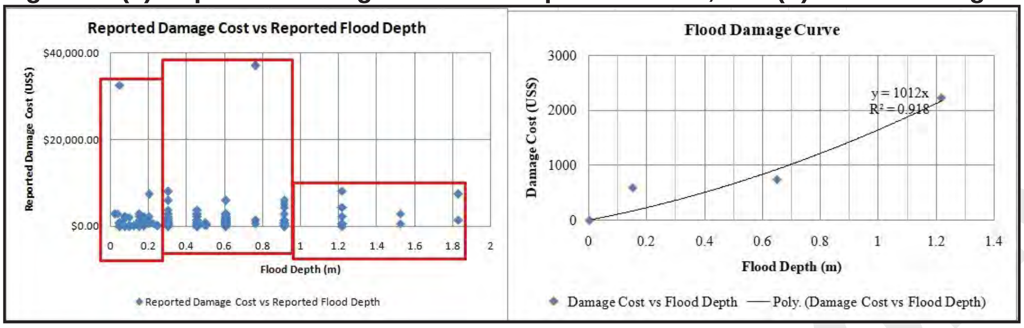
Table 2.0: Life Cycle Cost of Flood Mitigation Works for each Design Storm Frequency

	COSTS PER DESIGN STORM FREQUENCY (US$)				
	2yr	5yr	10yr	25yr	50yr
CHR Bridge Constr. Cost	$0	$0	$0	$0	$0
Mayfield Bridge Constr. Cost	$1,040,661	$1,861,225	$1,861,225	$1,861,225	$2,157,521
Levees Constr. Cost	$0	$4,160,446	$5,193,184	$6,597,554	$7,781,552
Total Construction Cost	$1,040,661	$6,021,670	$7,054,409	$8,458,778	$9,939,073
Design Cost (10% * Constr. Cost)	$104,066	$602,167	$705,441	$845,878	$993,907
Maintenance Cost (1%*50yrs)	$520,330	$3,010,835	$3,527,204	$4,229,389	$4,969,536
Life Cycle Cost / LCC	**$1,665,057**	**$9,634,672**	**$11,287,054**	**$13,534,045**	**$15,902,517**
Annual LCC (LCC/50yrs)	**$33,301**	**$192,693**	**$225,741**	**$270,681**	**$318,050**

3.2 Community Vulnerability - Flood Damage Curve and Flood Damage Costs

Using the information collected from residents via the field interview and surveys, Figure 5.0(a) shows the spatial distribution of reported flood depths and the associated damage costs. As stated in the methodology, the median of the reported damage costs was plotted against the flood depth ranges of 0-0.3m, 0.3-1.0m and above 1m to obtain a Flood Damage Curve as a mathematical representation of the community's vulnerability. Refer to Figure 5.0(b) below.

Figure 5: (a) Reported Damage vs Flood Depth Data Plot, and (b) Flood Damage Curve



The function for the Flood Damage Curve was correlated to the design flood depths obtained from the flood hazard maps (refer to Figure 4.0) for each design storm frequency to obtain the projected damage costs, should no mitigation works be implemented. The damage cost for each design storm frequency is presented in Table 3.0 below.

Once the models were adjusted to incorporate the mitigation works, it was noted that the volume of water that is typically detained within the North Valsayn community has now been conveyed downstream. This resulted in an increase in the flood risk and damages downstream. This increased cost in damages downstream was termed "Residual Damages" and is presented in Table 3.0. These values were found to be small as the increase in flood depths were small due to a significant volume of water being stored within the increased channel capacity due to the levees.

Table 3.0: Damage Cost and Residual Damage Cost

Design Storm Frequency	1 in 2yr	1 in 5yr	1 in 10yr	1 in 25yr	1 in 50yr
Damages in N.Valsayn (US$)	$3,653.92	$40,891.22	$72,258.09	$104,652.49	$112,673.51
Residual Flood Damages (US$)	$911.67	$2,546.35	$1,506.27	$4,263.35	$7,895.88

3.3 Costs per Inhabitant (CPI) Approach:

The estimated population of North Valsayn was identified as 4056 inhabitants and an acceptable budget for the flood mitigation works using the Cost per Inhabitant approach as described in Section 2.2 is US$365,040 per annum. Table 4.0 below compares this budget to the Annual LCCs of the mitigation works designed for various design storm frequencies. Figure 6.0 below shows the plot of the various components and highlights that the mitigation works designed for a 1 in 50 year design storm frequency can be afforded based on the recommended budget. The study was restricted to the 1:50yr storm due to limited data, however, it is expected that the Absolute Cost Difference from the CIP Budget would eventually increase as the design storm frequency increases resulting in a bell curve should the data become available.

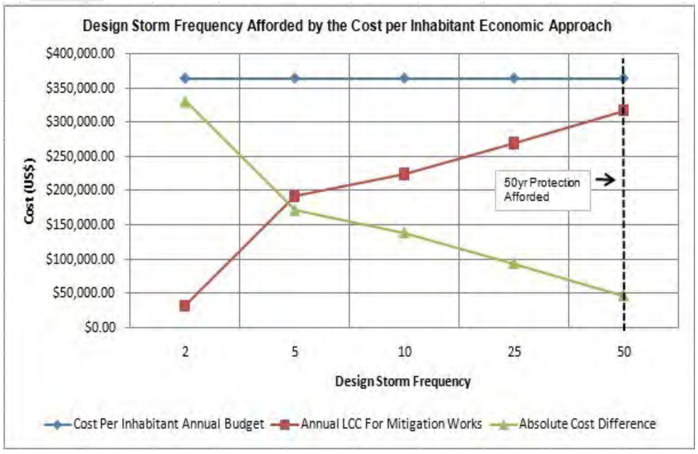


Figure 6.0: Shows the Design Storm Frequency Afforded should the recommended Budget derived from the Cost per Inhabitant Approach

Table 4.0: Comparison of an Acceptable Budget based on the Cost per Inhabitant Approach and the Annual LCC

Design Storm	1 in 2yr	1 in 5yr	1 in 10yr	1 in 25yr	1 in 50yr
Cost per Inhabitant per Annum	$365,040.00	$365,040.00	$365,040.00	$365,040.00	$365,040.00
Annual LCC	$104,931.80	$274,950.25	$307,997.88	$352,937.71	$390,825.65
Absolute Difference	$260,108.20	$90,089.75	$57,042.12	$12,102.29	$25,785.65

From Table 4.0 and Figure 6.0 above, it can be seen that the project budget recommended to a policy holder or decision maker is irrespective of the design storm frequency. However, when correlated to the annual LCC, the budget can afford the people of North Valsayn a level of protection against a 1 in 25-year storm event.

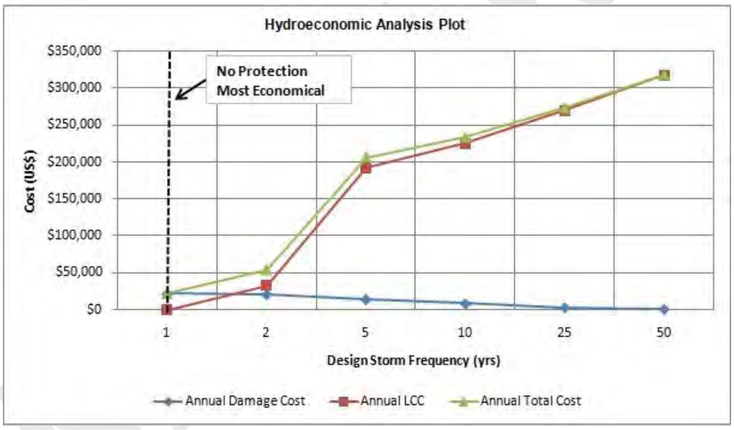
3.4 Hydroeconomic Analysis (HEA):

Using the annual LCC, Damage Costs and Residual Damage Costs from Tables 2.0 and 3.0 respectively, the HEA was conducted and presented in Table 5.0 and Figure 7.0:

Table 5.0: Hydroeconomic Analysis (HEA)

1. Increment (i)	2. Return Period (T) /yrs	3. Annual Exceedance Probability (AEP)	4. Damage (US$)	5. Incremental Expected Damage per Year (US$)	6. Annual Damage (US$)	7. Residual Damage (US$)	8. Incremental Expected Residual Damage per Year (US$)	9. Total Annual Damage = Annual Residual Damage + Col.6 (US$)	10. LCC per Year (US$)	11. Total Cost per Year (US$)
1	1	1	$0	$0	$20,733	$0	$0.00	$21,977	$0	$21,977
2	2	0.5	$3,654	$913	$19,820	$912	$227.92	$21,064	$33,301	$54,365
3	5	0.2	$40,891	$6,682	$13,138	$2,546	$518.70	$14,382	$192,693	$207,075
4	10	0.1	$72,258	$5,657	$7,481	$1,506	$202.63	$8,725	$225,741	$234,466
5	25	0.04	$104,652	$5,307	$2,173	$4,263	$173.09	$3,417	$270,681	$274,098
6	50	0.02	$112,674	$2,173	$0	$7,896	$121.59	$1,244	$318,050	$319,294
Expected Annual Damage in NValsayn(US$1000)				$20,733	Expected Annual Damage by Residual Risks (TT$1000)		$1,244			

Figure 7.0: Hydroeconomic Analysis (HEA) Plot



From Figure 7.0, the HEA indicated that it is most economical to maintain the existing flood condition as the annual cost of mitigation works far outweighs the annual damage cost.

3.5 Costs per Inhabitant Approach versus Hydroeconomic Analysis (HEA)

Upon execution of both economic approaches, the HEA indicated that it is most economical to maintain the existing flood condition as the annual cost of mitigation works far outweighs the annual damage cost. On the contrary, when implementing the Cost per Inhabitant approach, flood mitigation works performed for a design storm frequency of 1 in 50years was found to be comparable to the recommended budget. The study shows a disparity in defining an Acceptable Budget and an Optimal Design Storm Frequency for decision/policy makers, and various stakeholders although both are acceptably used in the industry.

The HEA approach described by Chow et. al. 1989 focuses on comparing the cost of the mitigation works to the direct and tangible impacts of flooding, namely the annual damage cost caused by floods. It does not account for simultaneous benefits outside of flood mitigation works that some hydraulic structures carry. In this case, while the total cost for the reconstruction of the Mayfield Bridge was used as part of the LCC to compare with the respective floods damages, in reality the bridge also serves as part of a road network and provides access and connectivity to the community. Hence, in addition to the improvements made to the HEA in this study, it can be further improved by quantifying the portion of the total construction cost that can be realistically assigned to flood mitigation works or the hydraulic component versus what portion can be assigned to the road network component. However, the Cost per Inhabitant is a more empirical approach determined from the cost estimates of the mitigation works accepted by developed countries such as the Netherlands in this case. It should be noted that the cost recommended by the Cost per Inhabitant approach states a factor between direct and total cost of 1.7 (Witteveen + Bros. 20197-8). It implies that this factor accounts for indirect and intangible impacts in addition to unforeseen costs or value. There is a possibility that the Cost per Inhabitant approach may yield an uneconomical result due to the differences between the developing and developed countries' physical, economic and climatic conditions.

The choice of which approach to be used generally depends on data availability and time constraints. Although the HEA accounts for only direct and tangible impacts of floods when used as a tool to recommend the most economical level of protection, it requires data to quantify the cost of damages for the various storm frequencies. Such data is not always readily available and so is the time and cost to collect and process said data. However, the Cost per Inhabitant approach in advising an acceptable budget for flood mitigation works can be done in a shorter space of time in the absence of flood damage data. Work needs to be done to determine how much persons in the country is willing to pay for flood protection and set that against the CPI values reported for developed countries.

4. CONCLUSION

The study successfully executed two (2) economic approaches used to inform a developing country on the budgets and the associated optimal design storm frequencies to be used for flood mitigation works. The results show a disparity in recommendations provided by the Hydroeconomic Analysis (HEA) and the Cost per Inhabitant approach adopted from the Witteveen + Bros 2019 study. While both methods are acceptably used in the industry, it is important that policy holders, decision makers and other stakeholders are aware of the disparity in results that are possible when implementing each approach and understand why such occurs to aid in managing the process and risks.

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The Value of Hydrometry in Reducing Fluvial Flooding Footprints across The Caribbean – A Case Study in Dennery, Saint Lucia

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ABSTRACT

The work reported in this paper was initially carried out by Alpha Engineering & Design (2012) Limited (Alpha) for the Government of Saint Lucia (GOSL) on a World Bank (WB) Disaster Vulnerability Reduction Project (DVRP) for Flood Mitigation. Part of the scope involved setting up rainfall and streamflow gauges in the Dennery Watershed to observe rainfall depth and river stage for a limited period congruent with the engineering consultancy contract, to guide the selection of hydrological parameters and calibrate hydraulic models. Alpha then sought to extend the observation period so that larger datasets could be captured to improve the reliability and utility of the data. The aim of this paper is to present the real-world benefit of investing in hydrometric instrumentation to increase one's capacity when Analysing the hydrological and hydraulic impacts of storm events in Caribbean Watersheds and improve reliability in flood mitigation analyses for more resilient solutions. This is done through a case study in Dennery Village, Saint Lucia. This paper briefly presents the challenges associated with setting up and maintaining gauging stations, describes the technology used, lists the high benefits for the comparatively low cost of the investment, and finally the analyses of the data using standard methodologies in engineering hydrology and hydraulics to generate catchment-specific information relating to rainfall and runoff especially in terms of flooding, such as the role of antecedent moisture content on the severity of floods and the impact of rainfall structure, in space and time, on a flood hydrograph for specific catchments.

Keywords: *Rain-Gauge, Streamflow-Gauge, River Stage, Stage Discharge, Flood Hydrograph, Hydrological Parameters, Watercourse baseflow, Irrigation*

1. INTRODUCTION

The Eastern Caribbean Island of Saint Lucia is exposed to a range of natural hazards, particularly meteorological phenomena, chief among them being hurricanes, strong winds, storm surges and heavy rainfall events, which lead to flooding and landslides (WB 2013). In addition, climate change-related impacts are expected to intensify precipitation patterns, thereby generating more extreme storms, hurricanes, floods and rises in sea level (IPCC 2021). Historical records show that extreme natural events have caused considerable destruction to the Island's infrastructure and social and economic sectors as they typically devastate coastal and low-lying areas, in which most of the Island's population and main commercial activities are situated. Such disasters impose exorbitant costs on the Country's fragile economy and can exacerbate poverty levels (Benson and Clay 2004). The Village of Dennery on the northeast coast of Saint Lucia has suffered severe and repeated flooding from extreme storm events including Hurricane Tomas (2010) which produced approximately 563mm of rain in 23 hours (UN ECLAC 2011). To this end, assessment of the hydrological/ hydraulic conditions of Dennery is important for the ongoing development and implementation of flood mitigation solutions including early warning systems to protect the residents.

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2. DESCRIPTION OF THE WATERSHED

The Dennery Watershed consists of three main sub-catchments: Trou a L'Eau Ravine (L'Eau River), Central Drain and Dennery/Mole River. These are illustrated in **Fig.** 1. Historical flooding records indicate flood levels of up to 1m in the coastal village following storms classified as having an intensity exceeding a 100 Yr. R.I. (E.g., Hurricane Tomas, 2010). Effective flood defense and resilience mechanisms as well as early warning systems are therefore fundamental to the economic, social and environmental well-being of the Village, and other similar watersheds across the island.

Fig. 1. Sub-Catchments within Dennery Watershed

3. DATA MONITORING IN DENNERY VILLAGE

Alpha currently monitors rainfall depth and river stage (to derive streamflow) in two sub-catchments within Dennery Village. Rainfall depth (mm) is measured hourly using a Global Water RG600 8" Tipping Bucket Rain Gauge (see **Fig. 2**). Each time the bucket is tipped, within a 135ms timeframe, it measures 0.25mm of rainfall and the attached magnet triggers a magnetic switch connected to an event/pulse counter to record the accumulated rainfall for the hour-long period. The instrument has an accuracy of ± 1% at 25mm per hour.

Fig. 3. Rainfall Gauge in Bois Joli, Dennery

River stage (m) is measured at a 15-minute time-step using a Nile 502 Pulse Radar Water Level Sensor with a measuring range of 20m and ± 2mm accuracy. The sensor detects the surface of the water and records the vertical distance (called the Nile distance) from the instrument's screen to the water surface through non-contact microwave transmission. The procedure requires two input arguments to derive streamflow. Firstly, the premeasured distance from the screen to the river's invert is used to subtract the Nile distance to derive the river's stage. The premeasured channel cross-section is then applied to Manning's Equation for Open Channel Flow to determine the corresponding streamflow.

The larger of the two is called the Dennery/Mole River Catchment (shown in **Fig. 5**). The rain gauge is installed at Errard Estate at an approximate elevation of 75m AMSL and the streamflow gauge was installed on the Highway Bridge crossing Mole River at an approximate elevation of 5m AMSL. The latter was decommissioned for reasons explained in the following section.

The second catchment is known as Trou a L'Eau (shown in **Fig. 5**). The rain gauge is installed in Bois Joli at an approximate elevation of 100m AMSL and the streamflow gauge is installed on the paved Trou a L'Eau River at an approximate elevation of 4m AMSL (**Fig. 4**).

Fig. 4. Nile Streamflow Gauge in Trou a L'Eau River, Dennery

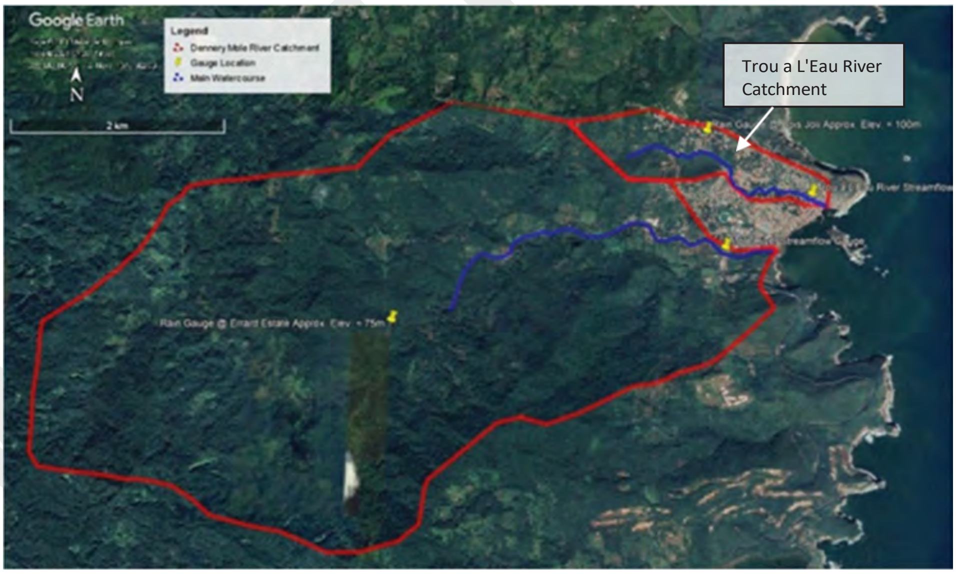
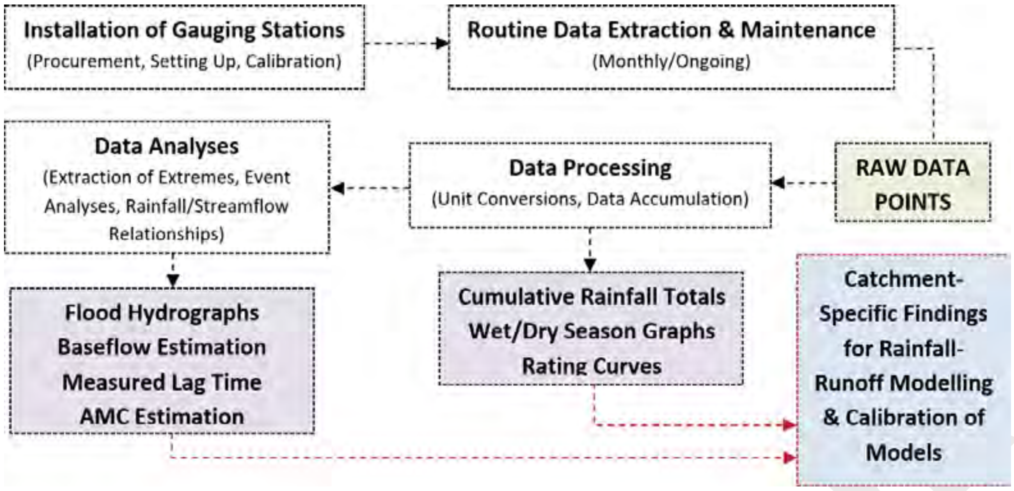


Fig. 5. Dennery/Mole River Catchment & Trou a L'Eau River Catchment

Methodology

The flow chart below illustrates the main steps taken to derive catchment-specific findings from the raw data points obtained from the gauging stations installed.



4. SETTING UP & MAINTAINING GUAGING STATIONS - CHALLENGES

Table 1 summarizes the procedures, challenges and lessons learnt during the course of setting up and maintaining the gauging stations in Dennery Village.

Table 1. Procedures, Challenges & Lessons Learnt from the Dennery Gauging Stations

Description	Global Water RG600 8" Tipping Bucket Rain Gauge (Rainfall Depth)	Nile 502 Pulse Radar Water level Sensor (Streamflow)
Site Selection Considerations	Open air away from obstructions (e.g., trees, buildings) to avoid interference with rain drops and/or the development of wind vortices, resulting in skewed measurements; High areas, avoiding low-lying areas such as valleys where strong surges of katabatic winds can distort the trajectory of rain drops, leading to a collection deficit; Secured location to avoid tampering with the equipment.	A straight river course for approx. 100m upstream of the gauging site with well-formed banks; No bypass channels to 'split' the channel flow; Channel is free of aquatic plants and flow obstructions such as mountain cobbles; Away from the confluence with another stream or from tidal effects to avoid any possible impacts on the equipment and/or stage heights recorded; Away from any flow constrictions such as culverts which would result in artificially increased stages; Secured location to avoid tampering with the equipment.
Installation Procedure Outline (Guided by the Manufacturers' Manual)	i. The tipping bucket was mounted on a vertical mast casted into the ground at a height approx. 1.8m above the existing ground level; ii. Checks were done to ensure any vibration during high winds is kept to a minimum;	i. The Nile Radar Unit was levelled and attached to an elevated horizontal bar mounted across the channel; ii. The Nile radar beam (having an elliptical/parabolic shape type of footprint) was aligned

	iii. The positioning and operation of the tipping bucket was then checked by removing the gold funnel and lightly pressing on either ends; iv. A GL500 Data Logger was then attached to the Tipping Bucket using the two wires from the bucket to the pulse channel of the GL500; v. Calibration procedures recommended in the manufacturer's manual were then performed.	such that the footprint of the beam encounters no obstructions on the way to the water surface; iii. The Nile Radar Unit was then attached to the beam by locating and aligning the vertical ticks on the device in the view of the water (away from the mounting structure); iv. The unit was collar-mounted in place to ensure no allowable rotation about its axis; v. A Storm 3 Data Logger was then placed in a Turnkey Integrated Enclosure and attached to a mast with equipped with solar panels for the main power source.
Security	Security measures were put place for the rain gauges, chief among them, being situated within a closed compound, in addition to locking features for the data loggers, etc. These are effective in maintaining operation of both rain gauges in Dennery with no tampering reported by ground staff to date.	Since the streamflow gauges are located within the channel corridors which are public spaces, security measures were limited to locking facilities and alarms. This has proven effective for the Trou a L'eau gauge to date. However, data records for Dennery/Mole River abruptly ended in July 2020 following a vandalism incident which resulted in loss of the main components of the gauge.
Data Retrieval	The gauges are currently equipped with data loggers to store recordings. Each site is visited on the same day each month and data cables are used to extract the raw data in Microsoft Excel Format to a laptop. Moving forward, the feasibility of investing in data transmitters for each gauge is being assessed. More on this below.	
Challenges Faced	Small debris frequently gets trapped in the grate and requires physical cleaning; Accessing the gauges for data collection, on-site maintenance and troubleshooting after storm events; Time-consuming to generate graphs from raw data, examine for anomalies and update each time new data is added.	Accessing the gauges for data collection, on-site maintenance and troubleshooting after flood events; Securing the facilities in public spaces; Rare occurrences of failure to connect to IP site during data retrieval exercises; Difficulty in identifying the cause of anomalies in the data. Time-consuming to generate graphs from raw data, examine for anomalies and update each time new data is added.

Lessons Learnt	Consideration to be given to accessing the rain gauges in the remote locations of the watershed during/after high-intensity storm events when access routes are compromised. See section on Data Transmitters below.	Gauges should be elevated at least 2m higher than the top of bank to increase the limits of the data by capturing the ebb and flow of flood depths during high-intensity long-duration storm events as well as to reduce interference of the beam by debris.

The greatest challenge faced in this effort was accessing the gauging sites for data collection and maintenance after significant events. **Table 2** summarizes the available options to overcome these challenges and improve the efficiency and utility of the data collection undertaking.

Table 2. Available Options to Overcome the Challenges Faced with the Existing Instrumentation

Available Options[1]	Use/Application	Costs per Site[1]	Additional Requirements
Option 1: Xylem HydroSphere Cloud Based Data Hosting Platform (compatible with the existing Storm 3 Data Logger)	Customizable graphs, tables, and dashboards; data-driven alarms and customizable escalation; Building networks of monitoring sites; Creating public websites for visualization of curated data; Cloud based design allowing access to data from any web-enabled device at any time (Xylem 2020).	USD $132/Yr.	Sierra Wireless RV50X Modem ($1,200 USD); Mast Antenna Kit, 10 FT ($520 USD).
Option 2: Campbell Scientific LoggerNet Data Logger Software Base Station Connect and Scheduling Software.	A suite of applications for datalogger programming, data collection, network monitoring and troubleshooting, as well as data display including graphs of real-time or historical data (Campbell Scientific, Inc. 2018).	This cost of this option will exceed Option 1 due to the need for replacing the existing data loggers with a compatible version in addition to a Wireless Modem & Antenna Kit. As such, the cost of implementing this option to overcome the challenges currently faced with the existing instrumentation cannot be ascertained at this time.	

[1]*Information provided by ROSE Environmental Limited of Trinidad & Tobago.*

Table 3 and **Table 4** summarize the costs incurred in the procurement, implementation and ongoing operation/maintenance of the rainfall and streamflow gauges by Alpha.

Table 3. Costs Associated with Procuring, Implementing and Maintaining the Rain Gauges

Component[1]	Unit Cost USD (Not Incl. VAT)	CIF[2]	Brokerage & Delivery Fees[2]	Installation Costs/Unit[2]	Maintenance Fees/Month[2]
Global Water Rg600 8" Tipping Bucket Rain Gauge	$621.00	$270.00	$40.05	$950.00	$250.00
Gl500 Data Logger	$441.70				
K-114a Dcx Usb Communication Cable	347.00				
Total Cost Of Rain Gauge =	**$1409.70**	USD Per Unit (Not incl. VAT)			

[1]Supplier – Global Water; [2]Costs Incurred by Alpha Not Included in Total Cost of Gauge

Table 4. Costs Associated with Procuring, Implementing and Maintaining the Streamflow Gauges

Component[1]	Unit Cost USD (Not Incl. VAT)	CIF[2]	Brokerage & Delivery Fees[2]	Installation Costs/Unit[2]	Maintenance Fees/Month[2]
Nile 502 - Pulse Radar Water Level Sensor - 0 To 20 M	$2 600.00	$550.00	$55.31	$2100.00	$250.00
Storm3-00 - Storm3-00 Data Logger With Web User Interface	$1 071.00				
Turnkey Integrated Enclosure,14x12, User Defined Data logger And Telemetry	$2 900.00				
Grounding Rod Kit, Light Duty For Tk Enclosures	$103.53				
H-Sdi-Cable-15 Weather/Sun Resistant Sdi-12 Cable,4 Conductor,22 Awg,15 Meters	$375.00				
Nile Mb - Mounting Bracket Kit, Nile Radar	$113.00				
Total Cost Of Streamflow Gauge =	**$7162.53**	USD Per Unit (Not incl. VAT)			

[1]Supplier – Xylem; [2]Costs Incurred by Alpha Not Included in Total Cost of Gauge

5. QUALILY & QUANTITY OF DATASETS

Table 5 summarizes the data collected to date and the following graphs illustrate the accumulative Daily Rainfall and Nile Distance at each gauge site.

Table 5. Summary of Rainfall and Streamflow Data Records Acquired to Date in the Dennery Watershed

Gauging Station	Observation Period		Data Resolution	Accumulated Period to Date	Reference Charts from Raw Data Extraction
	From	To (*ongoing)			
Errard Estate Rain Gauge	27-Mar-2020	22-Jul-2022*	hourly	28 Months	**Fig. 6**
Bois Joli Rain Gauge	4-Mar-2020			29 Months	**Fig. 7**
Dennery/Mole River Streamflow Gauge	27-Mar-2020	9-Jul-2020	15-min intervals	3.5 Months	**Fig. 8**
Trou a L'Eau Streamflow Gauge	27-May-2020	22-Mar 2022*		28 Months	**Fig. 9**

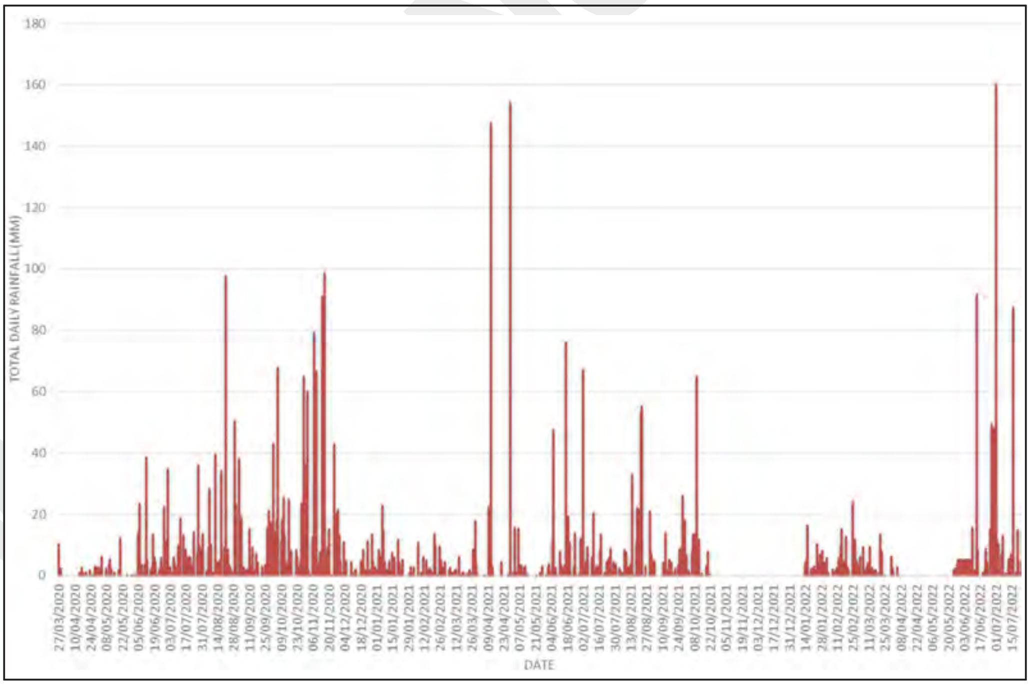


Fig. 6. Rainfall Data Records at Errard Estate Up to 22-Jul-2022

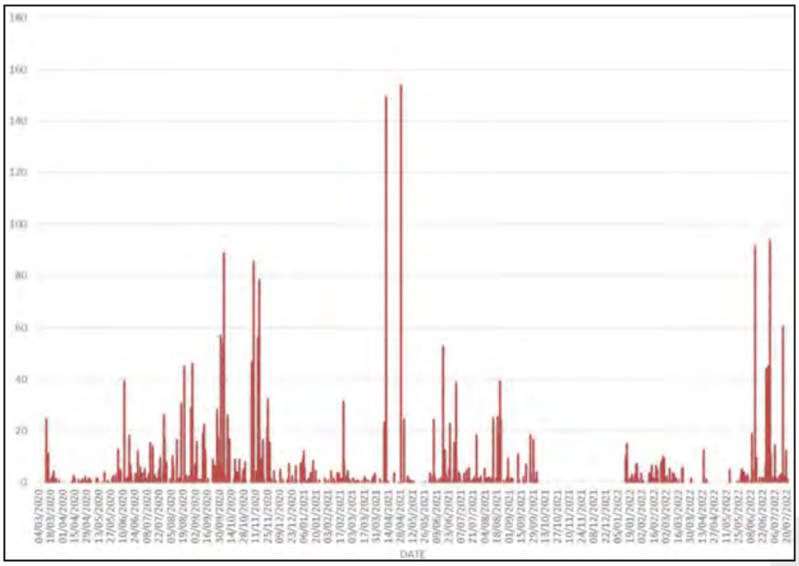


Fig. 7. Rainfall Data Records at Bois Joli Up to 22-Jul-2022

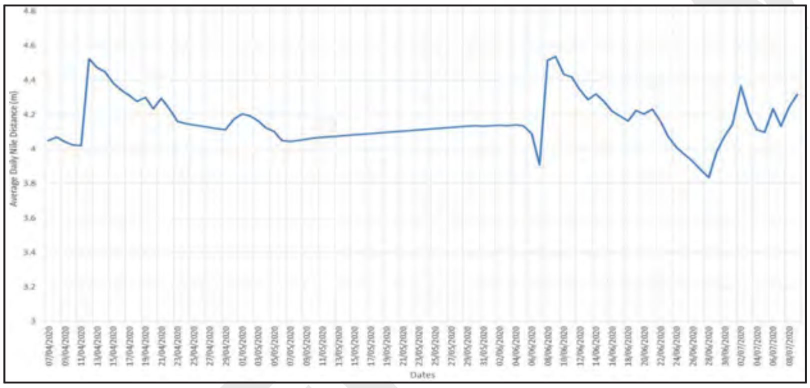


Fig. 8. Nile Distance Measurements Recorded in Mole River Up to 9-Jul-2020

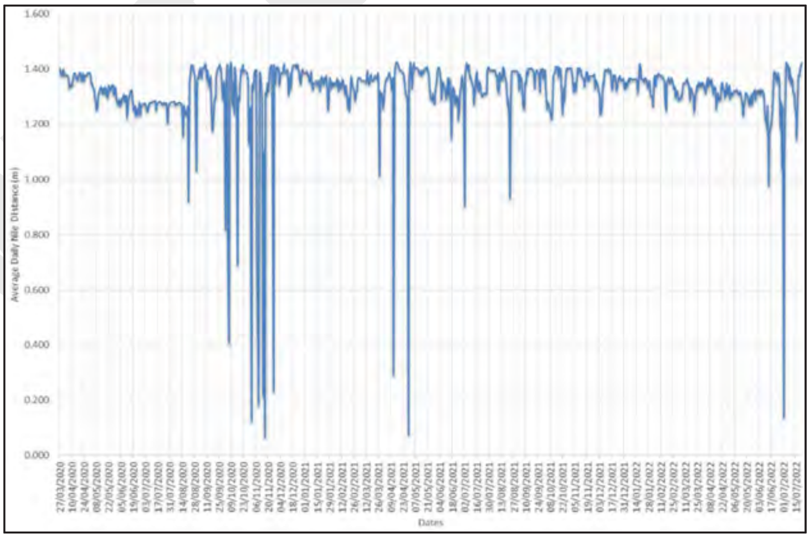


Fig. 9. Nile Distance Measurements Recorded in Trou a L'Eau Up to 22-Jul-2022

As is the case with any data collection effort in earth science, anomalous values are common phenomena and can skew average or maximum/minimum trends in the data. For e.g., strong surging winds in coastal areas, where they are very common, are primary concerns for rainfall under catch in Tipping Bucket Rain Gauges (Sieck, Burges and Stiener, 2007), but more so in Weighing Rain Gauges which are elevated higher off the ground and not used in this venture. Usually, anomalies are easy to detect as they vary significantly from the immediate before and after recordings and are standalone. Methods to detect and remove outliers in time series data are documented by Lu et al (2018) and are to be considered when the data series is extended. To date, no apparent anomalies were detected in the acquired measurements.

In the case of streamflow gauging, uncertainties lie more with the discharges stemming from extrapolated parts of a Rating Curve than from the raw point data itself. Persistent stream gauging is essential to control the validity and uncertainty of Rating Curves, especially during flood events (Coz 2008). This is expanded in more detail below.

6. HYDROLOGIC AND HYDRAULIC DETERMINATIONS FROM DATA COLLECTED

Using the rainfall and streamflow data collected between Mar-2020 to Jul-2022 (exact dates in **Table 5**), the following **catchment-specific information** was generated and can be used for ongoing studies as they relate to watershed management, rainfall analyses, flood mitigation and even yield hydrology for smart agriculture projects in the local community.

6.1 Spatial Correlation of Rainfall

For the Period 27-Mar-2020 to 22-Jul- 2022, the total rainfall at Errard Estate (75m AMSL) was measured as 4668mm, while at the Bois Jolie (100m AMSL), 3200mm was recorded. The latter corresponds to 69% of the total rainfall at Errard. When it comes to major rainstorm events, the measurements are invariably the same for the two days in April 2021, each recorded over a 12-hour duration, as shown in **Table 6**. These overall results are as expected, given the close proximity of these gauges (3km). This enabled us to estimate a correlation factor of 0.85, corroborating the findings of Linacre and Geerts (1998) from ground-based data in mountainous regions.

Table 6. Total Rainfall Recorded at Errard & Bois Joli in Dennery for the Two Storm Events in April 2021

Date	% Diff	Errard (75m AMSL)	Bois Jolie (100m AMSL)
12/04/2021	1.2%	147.574mm	149.35mm
29/04/2021	0.2%	154.178mm	153.92mm

These results support the assumption typically made in hydrology that rainfall data from nearby catchments can be applied to an ungauged watershed due to a high spatial correlation. This also provides some insight to the question of "What is the probability of rain in one location, given that X mm of rainfall was recorded at a neighbouring location?" and can be applied to other catchments across the Island, for which an adjacent catchment is currently being monitored. As documented in Hutchinson (1995), the rate of correlation decline with distance is highly asymmetric. Of course, there are significant factors, other than distance, that impact stochastic events such as rainstorms. The % difference calculated between the two rain gauges is negligible (i.e., <10%) and tells us that spatial distribution is generally uniform for rainstorms of this magnitude in these catchments and the small variations will not have a significant impact on a flood hydrograph.

6.2 High-Intensity Rainfall & Widespread Flooding Events Captured by Alpha's Gauges

The 2020 and 2021 Wet Seasons are illustrated in **Fig. 10** and **Fig. 11**. In 2020 there were no rainfall events that exceeded 100mm in 24 hours. This included all the typically wet months of October and November. The same cannot be said for 2021, in which two significant high intensity rainfall events occurred on April-12 and April-29 that caused widespread riverine flooding throughout the Island of St Lucia and in Dennery Village (News784 2021). Both rainstorm events were nearly 150mm (6 inches) in depth, occurring in 10 to 12 hours (see **Table 6**). This was determined from examination of the hourly measurements recorded by the rainfall gauging stations. Hyetographs are shown below for the two events. Through further analysis of the data and with the injection of the corresponding river stage values by the Streamflow Gauge at Trou a L'Eau, flood hydrographs were developed for the two events. These are shown in **Fig. 12** and **Fig. 13**.

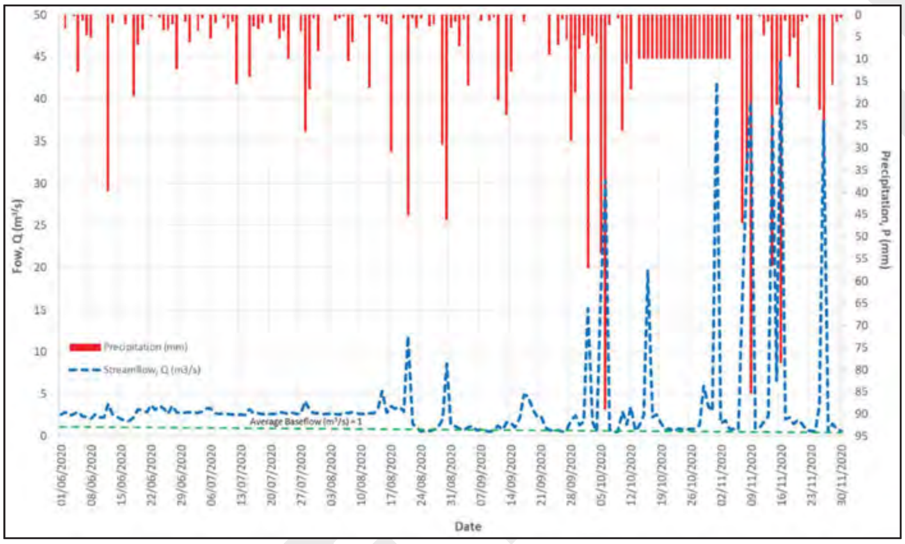


Fig. 10. 2020 Wet Season Graph

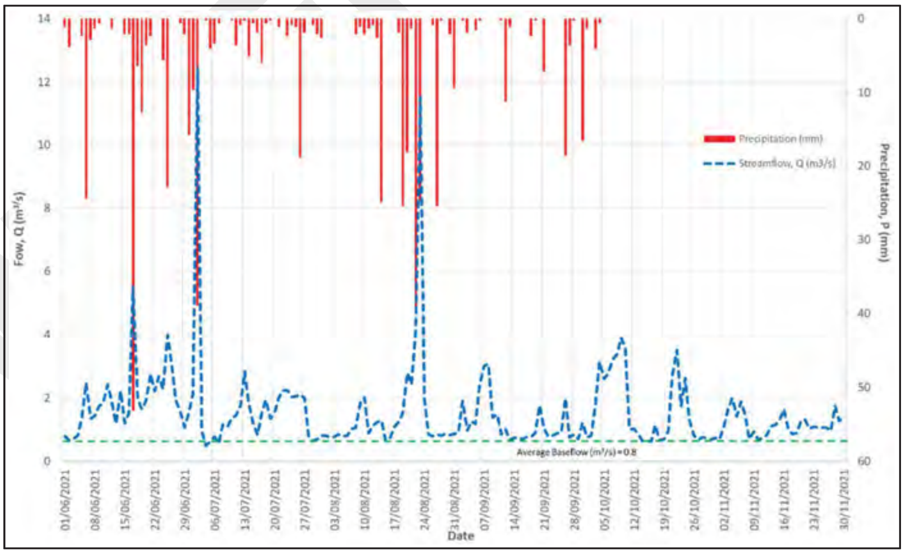


Fig. 11. 2021 Wet Season Graph

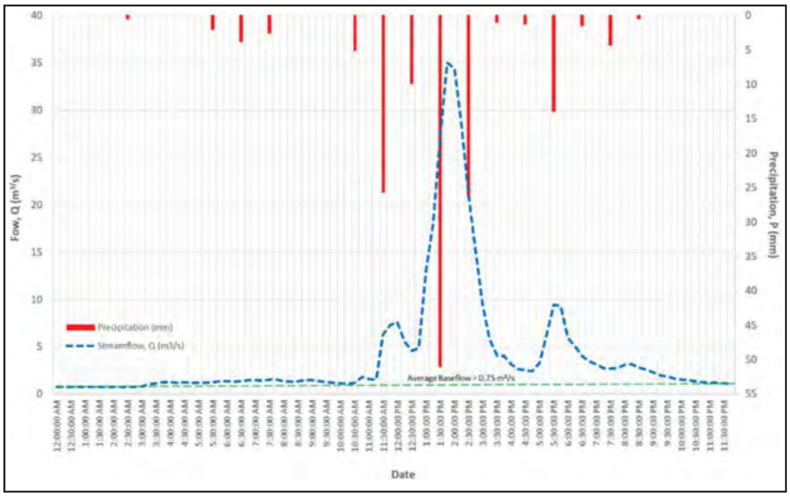


Fig. 12. Flood Hydrograph for the 12-April-2021 Storm Event in Trou a L'Eau

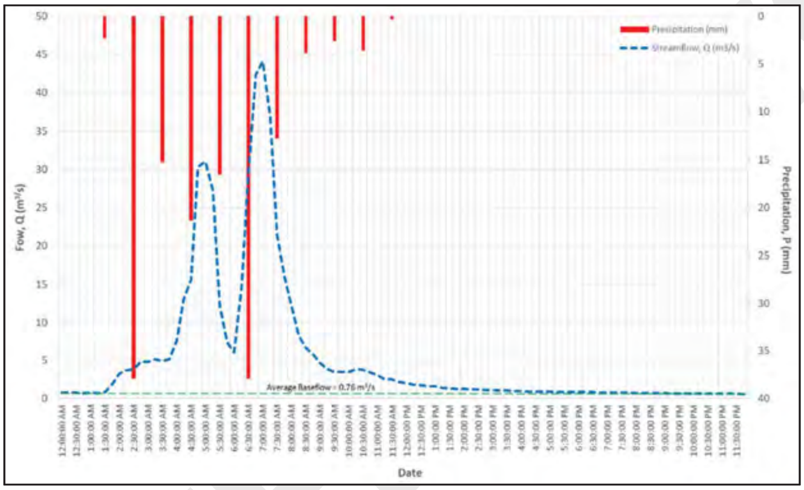


Fig. 13. Flood Hydrograph for the 29-April-2021 Storm Event in Trou a L'Eau

6.3 Classification of the April 2021 Events in Trou a L'Eau

One can categorize these similar rainfall events by plotting them against applicable relevant IDF Curves. Using an average intensity for the 12hr rainfall of 15mm/hr, it appears to be in the order of a 10 Yr. recurrence interval (RI) storm event, as shown on **Fig. 14**.

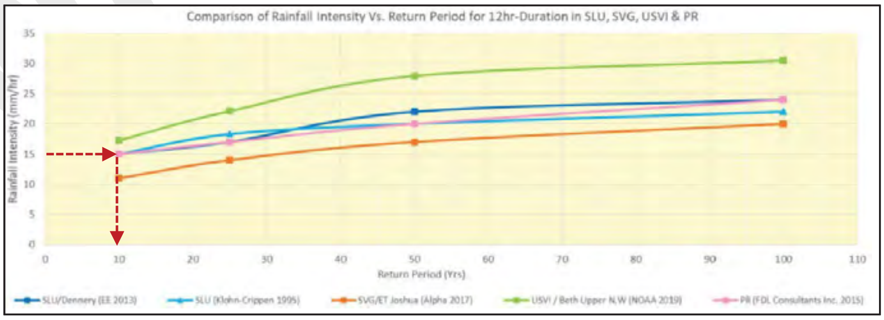


Fig. 14. Classification of the April 2021 Events in Trou a L'Eau Based on Applicable Relevant IDF Curves

6.4 Estimation of Baseflow in the Trou a L'Eau Ravine (L'Eau River)

A review of the Dry Season flows in the L'Eau River can establish the stream's baseflow by looking at the continuous data over the three dry seasons recorded. The dry and wet seasons are generally December – May and June – November respectively. In 2020, measurements started on March 27 and major rainfall commenced after April 19. Since the data collection was continuous thereafter, we observed the dry season conditions experienced in 2021 and 2022 as well. These dry seasons are graphed accordingly in **Fig. 15**, **Fig. 16** and **Fig. 17.**

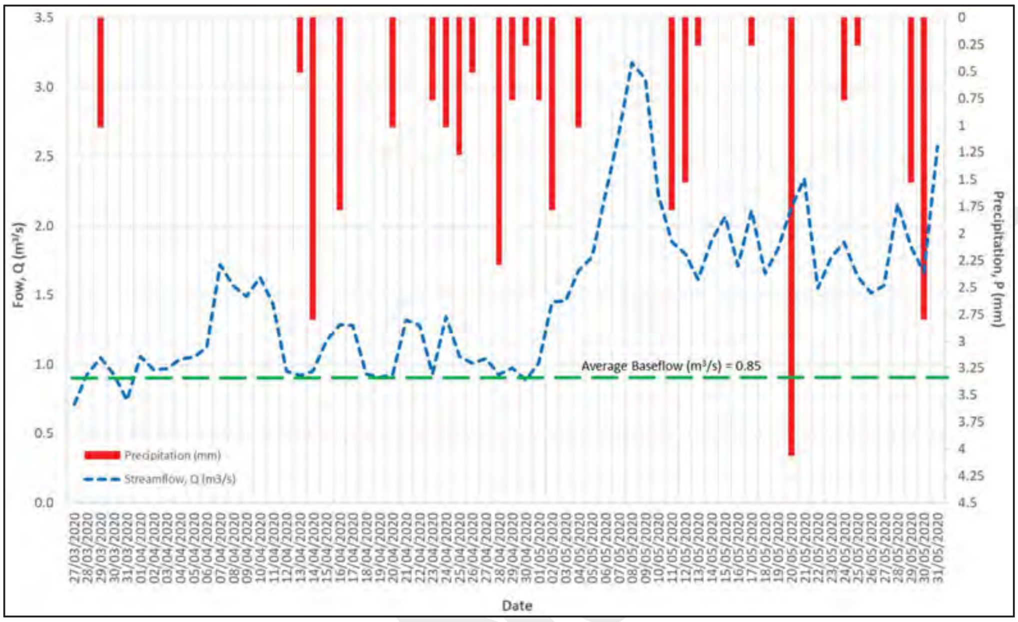


Fig. 15. 2020 Dry Season Graph (Bois Joli / Trou a L'Eau River)

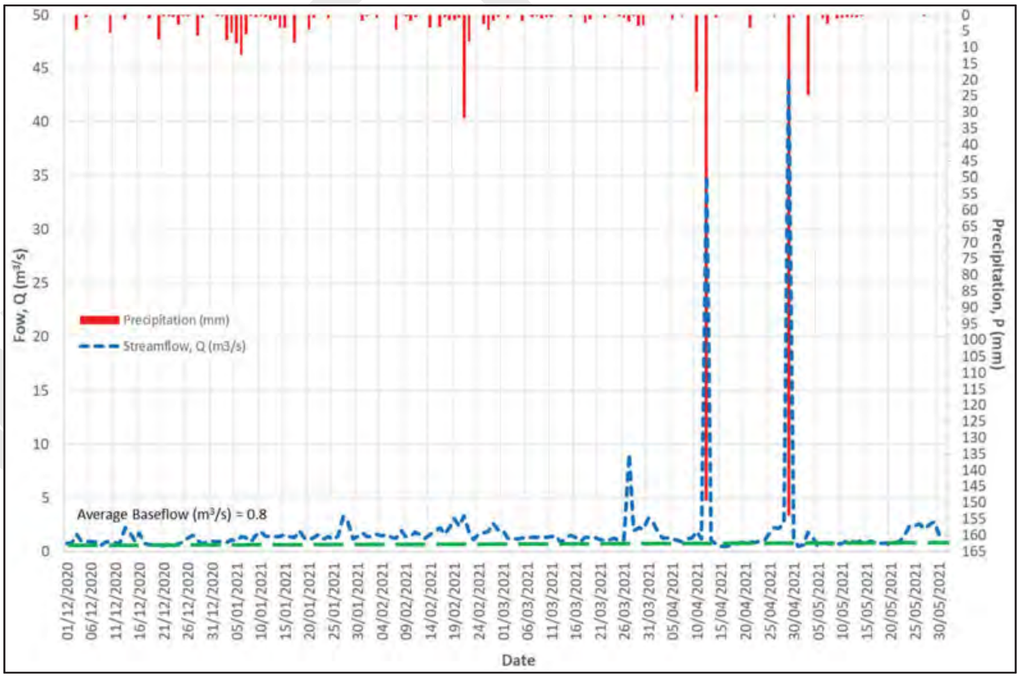


Fig. 16. 2021 Dry Season Graph (Bois Joli / Trou a L'Eau River)

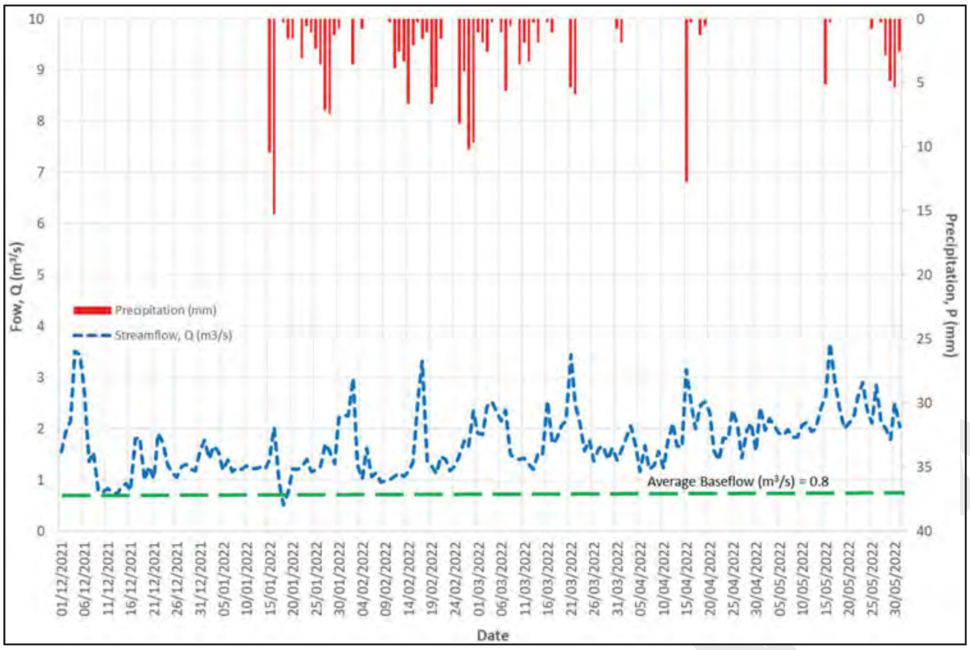


Fig. 17. 2022 Dry Season Graph (Bois Joli / Trou a L'Eau River)

The baseflow can be estimated from the graphical output above as approximately 0.8m³/s at the location of the streamflow gauge in the Trou a L'eau River. This measurement device is approximately 150m upstream of the ultimate outfall of the watercourse to the sea. There is no reported flora or fauna in the downstream area, neither is there an established extraction for anthropogenic use. It is therefore possible to extract at least 20% of this flow or 0.16m³/s (approximately 3.65 MGD) for possible agricultural irrigation use within the village, while maintaining reasonable environmental flows (Richter 2009).

6.5 Rating Curve for Trou a L'Eau River

A Rating Curve represents the relationship between water level (river stage) and discharge (streamflow) in a river (Maidment 1993). The one shown in **Fig. 18** was developed for Trou a L'Eau River with the data collected in **Table 5**. It estimates a bankfull discharge of 43m3/s and a corresponding water depth of 0.18m for the 0.8m3/s baseflow determined from the Dry Season Graphs. Plots of this nature are typically useful for discharge above bankfull stage and to indicate a change in the effective hydraulic area of the channel (e.g., a reduction due to blockage) or in the case of earthen channels, an increase due to excessive scour. It should be noted that the flat portion of a Rating Curve is the indicator of discharge above bankfull stage (Maidment 1993) and as such, the curve in **Fig. 18** requires a significant number of additional stage-discharge pairs to be fully developed. According to Sauer (2002) for the USGS, Rating Curves are developed over time and need to be maintained. They are unique to the site and period of time in which they are developed if they are not continuously updated with new data or reviewed with synthetic Rating Curves (Rojas et al 2020).

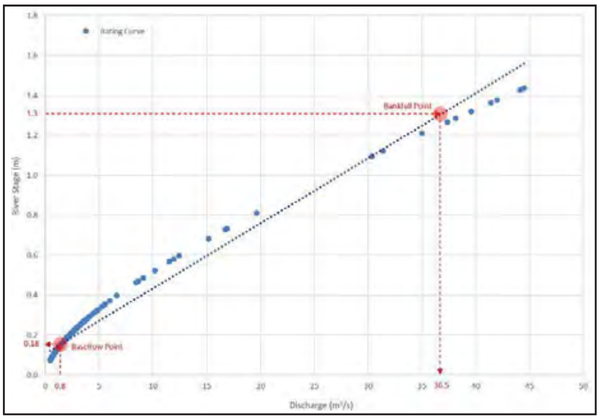


Fig. 18. Rating Curve Developed for Trou a L'Eau Ravine Using Data from March 2020 to July 2022

6.6 Value of the Recorded Data in Future Hydrologic and Hydraulic Determinations

The data collected and analysed to date (presented in the preceding sections) can be used in numerous ways to inform the selections and assumptions made in hydrological and hydraulic modelling to narrow the uncertainty margin and obtain closer orders of magnitude in the output. err

Table 7. Utility of Recorded Data in Hydrological/Hydraulic Modeling

Parameter	Determination for Hydrological/Hydraulic Model	Utility of Recorded Data
Rainfall Distribution	Typically selected from the pre-determined SCS 24-Hr rainfall distributions and disaggregation factors.	Observing the actual location of peak rainfall and distribution across the storm duration to develop disaggregation factors and corroborate the central nesting of the peak of a storm in accordance with NRCS procedures (USDA 2015).
Design Storm Selection	A selection of storm characteristics (in terms of rainfall depth/intensity and duration) based on a specified level of flood risk.	Guides this selection by indicating to designers and stakeholders the flood levels experienced for a measured quantity of rainfall in a known amount of time.
Time of Concentration/ Watershed Lag Time	Determined from pre-developed mathematical approaches outlined in USDA (2010) in which estimations are made for watershed slope and roughness.	The lag time can be measured from the flood response hydrographs developed in the rainfall-runoff event plots and used to corroborate/adjust the values derived in mathematical models.
Antecedent Moisture Content	Typically selected from predetermined conditions by NRCS. E.g., ARC I assumes fully saturated conditions.	A catchment-specific Antecedent Precipitation Index (API) can be calculated using the rainfall recorded from the previous 3 – 6 days of a storm event to aid these assumptions and inform the selection of Curve Numbers in the SCS-CN Model (Ali et al 2008) for predicting the runoff from larger storm events.
Selection of Runoff Coefficients *(CN – Curve Number / C – Runoff Coefficient)*	Based on visual inspection, land use maps and available soil surveys.	The measured lag time and AMC of a catchment can be used in conjunction with the tools already utilized for a more accurate determination of runoff coefficients. Rainfall-runoff models can be calibrated by back-calculating the "measured" CN or C value that would result in the peak flow measured by the streamflow gauge for the input rainfall depth measured by the rain gauge in the same catchment which can then be applied to the Two-CN System Approach by Soilus and Valiantzas (2012) to

		reduce runoff prediction errors related to the use of single composite CN values.
Baseflow Determination	Usually, sufficient data is not available to extract baseflow and hence it is determined by anecdotal evidence and/or a one-time measurement on site in the dry season reasonably assuming there was no preceding rainfall.	The measured catchment response times can be used to execute "baseflow separation" from a flood hydrograph (Maidment 1993) and/or dry season data can be used to extrapolate the flow in channels during periods of no rainfall (as done herein).

Table 8. Who Can Use This Data?

Groups	Application of Data
Engineers/Hydrologists	To inform hydrological/hydraulic modelling applications for flood mitigation across the island
State Agencies[1]	WRMA, WASCO for Use of Water Resources / NEMO for Planning Response Strategies
Farmers	Climate-Smart Agriculture[2] / Managing and optimizing the use of resources
Community Members	To prepare for flood events and increase awareness to reduce flood risk

[1]Perhaps this data is most useful to state agencies who have the national capacity and authority to share and analyse the data, implement similar interventions in other vulnerable areas, apply these findings to other areas, and prepare for future flood events to reduce flood risk and increase resilience to climate change for an Islandwide domain.

[2]According to FAO (2021), Saint Lucia's food production market presents inconsistencies in demand and supply of vegetable crops due to the recurring challenge of the absence of crop scheduling in seasonal conditions. This is exacerbated by a changing climate and the absence of irrigation technologies. Rainfall and streamflow data is of paramount importance in optimizing the use of water resources in the dry season for crop production and protection of the same in the wet season. This data can be applied to similar ungauged catchments where land use is predominantly agriculture to inform crop scheduling.

7. CONCLUSION

In this paper, we explored the value of adding rainfall and streamflow gauging stations in an ungauged flood-prone coastal watershed for the primary purpose of analysing hydrology and hydraulics for flood mitigation through a Case Study in east coast Dennery Watershed in Saint Lucia. While the ongoing venture has posed many challenges and issues to date, from an early collection spanning approximately 28 months, several site-specific findings from the rainfall-runoff relationships obtained have informed/corroborated much of the assumptions/estimates that were made in hydrological and hydraulic modelling of the watershed. The main conclusions drawn from our study are as follows:

i. Rainfall and streamflow gauging in small watersheds and their sub-catchments provide useful information for, inter alia, flood mitigation and climate-smart agriculture planning at relatively cheap costs;
ii. Setting up of rainfall and streamflow gauges require technical competence and also a level of security to protect and maintain the equipment;
iii. Options are available at a reasonable cost to upgrade the existing gauging systems with software to overcome the challenges currently faced with data retrieval, security and processing;
iv. Spatial variation of high intensity rainfall within the small watershed of Dennery is minor;

v. Rainfall events exceeding 150mm in 12 hours cause severe flooding in the lower reaches of the Trou a L'Eau Village;
vi. A 12-hour event with cumulative rainfall depth of 150mm has a recurrence interval of 10 Years and therefore has a 10% chance of occurring in any given year;
vii. The dry season baseflow in the Trou a L'eau River is approximately 0.8m^3/s at an average depth of 0.18m;
viii. Irrigation water is available based on the above of up to 3.65MGD even if one takes 20% of the base flow to allow for environmental flow contribution downstream.

Although our current investigation focused on the case watershed, it provides much insight for conducting hydrological and hydraulic modelling as well as water resource planning for the entire island of Saint Lucia and other Caribbean states. In the future, after more wet seasons are captured in our datasets, we plan to develop various rainfall-runoff models for this catchment and use the findings from event hydrographs of various magnitudes/durations (similar to the two performed herein) to determine input parameters including Curve Numbers (CN) / Runoff Coefficients (C) and assess how CN/C values vary with rainfall intensity across the catchment to bridge the gap between hydrological parameterization and catchment hydrological response in similar Caribbean watersheds. This work will also explore the possible reasons for any differences over time and provide the basis for a catchment-specific direct relationship between rainfall depth and runoff to inform future flood forecasting in the catchment.

LIST OF ACRONYMS

AEDL – Alpha Engineering & Design (2012) Ltd (Alpha)
AMC – Antecedent Moisture Condition
AMSL- Above Mean Sea level
API - Antecedent Precipitation Index
ARC - Antecedent Runoff Condition
CN - Curve Number
CSA - Climate-Smart Agriculture
GOSL - Government of Saint Lucia
IDF - Intensity-Duration Frequency
MGD – Million Gallons per Day
NEH - National Engineering Handbook
NRCS - Natural Resources Conservation Service
OCF - Open Channel Flow
SCS - Soil Conservation Service

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Considerations for Use of Permeable Pavement Systems within Urban Settings across Caribbean Small Island Developing States

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ABSTRACT

Increasing imperviousness caused largely from urban development coupled with global warming, sea-level rise and change in weather patterns contribute immensely to frequent flooding events across numerous urban municipalities across Caribbean Small Island Developing States (SIDS). Existing conventional drainage systems fail to meet stormwater runoff peak flow and volume demands generated by today's changing environment. Land or service constraints often restrict expansion of these drainage systems. Despite those challenges, Caribbean SIDS authorities and drainage engineers continue to recommend and use conventional drainage systems as the dominant infrastructure for the collection and conveyance of stormwater away from urban areas. Sustainable Urban Drainage Systems (SUDS) or Low Impact Development (LID) practices such as porous or Permeable Pavement Systems (PPS) are designed to effectively manage stormwater runoff at the source as opposed to conventional drainage systems. PPS reduce urban runoff and peak flows via development of on-site temporary storage measures for potential water reuse and minimisation of impervious areas. Water quality benefits of PPS include thermal mitigation and reduced pollutant loadings of suspended solids, heavy metals, hydrocarbons, and some nutrients to receiving natural waters. It is recommended that SUDS such as PPS be incorporated within urban drainage systems across Caribbean SIDS to help mitigate the frequent flooding events being experienced annually. PPS installations must be fit for purpose and this paper discusses key considerations for use of PPS within urban settings across Caribbean SIDS.

Keywords: Permeable Pavements, Stormwater Management, Sustainable Urban Drainage Systems (SUDS), Small Island Developing States (SIDS), Surface runoff

1. INTRODUCTION

Conventional Drainage Systems (CDS) designed to focus primarily on stormwater quantity control continue to dominate the urban watersheds across numerous municipalities within Caribbean Small Island Developing States. CDS do not focus on environmental concerns relating to water quality, visual amenity, biodiversity and ecological protection [1]. Additionally, CDS's limited capacity and flexibility to adapt to future climatic variability and urbanisation has been continuously criticized [2, 3]. These urban watersheds are often dominated by large percentages of impervious surfaces from rooftops, parking lots, roadways and driveways that contribute immensely to flooding. Further to increasing urbanisation, poor land use practices, improper utilisation of drainage infrastructure (littering) and faulty designs often exacerbate drainage issues.

SUDS such as PPS can offer an added flood risk mitigation benefit to stormwater management authorities across Caribbean SIDS [4]. PPS are engineered to perform as hybrid pavements with structural requirements typically designed to satisfy lightly trafficked surfaces such as parking lots and pedestrian access whilst promoting infiltration and stormwater runoff mitigation [5]. Permeable pavements supersede conventional paving with an at-source control to prevent or significantly delay stormwater runoff generation [6].

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The typical structure (Figure 1) consists of a permeable paving surface and layers of coarse aggregate materials that function as a storage reservoir during rainfall events. Aggregates such as crushed stone are the most dominant component in permeable pavement systems (PPS). For improved hydraulic and structural performance these aggregates are typically clean, single-sized, or open-graded and angular.
This paper discusses some key considerations for applications of permeable pavement systems within urban municipalities across Caribbean SIDS.

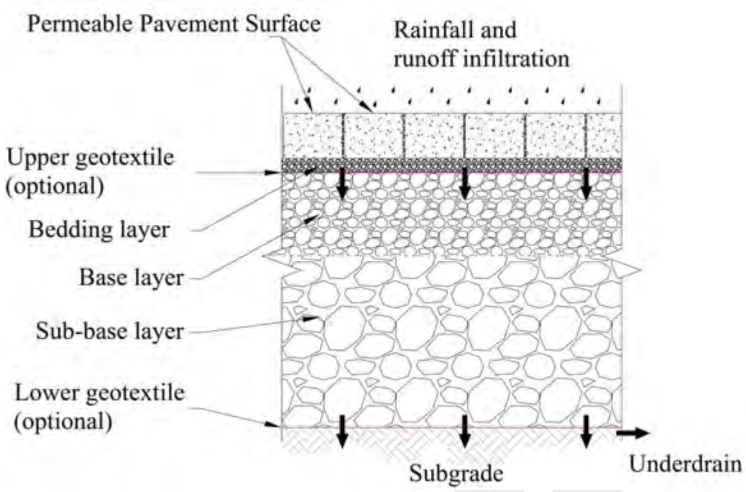

Figure 1 Cross section schematic of a permeable pavement system with geotextiles

[Reprint with permission from [5]]

2. DISCUSSION

2.1 Considerations for use of Permeable Pavements in SIDS

Stormwater management in urban watersheds across Caribbean SIDS are impacted by numerous factors including political instability, economic fragility, climate change vulnerability and infrastructural and maintenance challenges. Despite these challenges, the goal of maintaining efficient urban drainage systems still applies. Discussed below are some key considerations for use of permeable pavements in SIDS.

2.1.1 Physical

2.1.1.1 Structural Integrity and loading applications

The anticipated traffic loadings supported by the permeable pavement can be characterised according to AASHTO [7] as Equivalent 80 KN (18,000 lb) Single Axle Loads (ESALs) [8]. Permeable pavements in North America have typically been designed for applications not exceeding approximately 1 million ESALs [9]. Traffic loadings across Caribbean SIDS are expected be to significantly less than 1 million ESALs due to the lower traffic volumes, smaller parking lots and fewer heavy trucks.

2.1.1.2 Design Storms

It is important that the appropriate design storm is selected for the design of PPS because this establishes the volume or depth of rainfall which should be considered in the design [10]. Permeable pavements need to be able to effectively capture the design storm event and discharge it in a controlled manner to the appropriate discharge mechanism [11]. A permeable pavement system should be able to infiltrate most, or all of the 2-year, 24-hour storm and that the performance of the system should be checked for at least the 10-year, 24-hour storm [10]. Caribbean SIDS are challenged by a scarcity of Intensity Duration Frequency (IDF) curves either because of the limited quantity of short-duration rainfall data being available or because the few IDF curves that have been developed are generally not freely available to the public [12]. Moreover, Caribbean SIDS typically consist of small watersheds that are sensitive to high intensity, short duration storms that often result in flash floods. Cloudbursts (defined as rainfall intensity more than 100 mm/h over a short duration for example 15 minutes) along with spatial and temporal variations in rainfall often present further challenges.

2.1.1.3 Aggregate selection and availability

Given the geological confinement of most SIDS, suitable aggregates may not be locally available for incorporation in PPS. Further, there is a growing demand for construction aggregates in SIDS as demand for housing and other public infrastructure increases with urbanisation. In Trinidad and Tobago, W.I. for instance, the demand for construction has seen a drastic increase during the past decade due to various infrastructural developments. Sustainable development requires the preservation of the environment and conserving the rapidly diminishing natural resources [13]. SIDS nations should, therefore, consider the use of alternative materials in PPS such as Construction and Demolition Waste (CDW), Crushed Brick (CB), Recycled Aggregates (RA), Crushed Concrete Aggregates (CCA), Recycled Concrete Aggregates (RCA) and Carbon Negative Aggregates [14].

2.1.1.4 Subgrade conditions

Permeable pavements are designed for full, partial or no infiltration based on the in-situ soil infiltration rates or Hydrologic Soil Groups (HSG) [15]. Thicker pavements are generally required for pavements constructed over fine-grained soils (silts and clays) as opposed to those constructed over coarse-grained soils (sands and gravels) [16]. This is mainly because fine-grained soils typically have lower bearing capacities than coarse-grained soils that are capable of withstanding greater stresses without excessive deformation. In instances of expansive clay subgrades, such as those present in the southern regions of Trinidad and Tobago and several other Caribbean SIDS, infiltration of stormwater into these expansive clays is not recommended. In such instances, designs may recommend a subgrade replacement layer such as sand or the addition of stabilisation additives such as cement or lime to the existing clay subgrade [16].

2.1.1.5 Water Table

Guidelines recommend that permeable pavements be installed between 600 and 1000 mm above the maximum groundwater level to provide a depth of unsaturated soil that helps to ensure the infiltration performance of the permeable pavement and to protect any groundwater present from possible contamination [11]. Some Caribbean urban municipalities are located on coastal lowlands, which by nature, have high water table levels. Permeable pavements may not function well in these locations.

2.1.1.6 Groundwater contamination

Permeable pavement installations include partial or full infiltration, therefore the potential for groundwater contamination is a concern [17]. These concerns have been minimised since according to Wilson, Newman [18], the incorporation of an adequately designed and constructed impermeable geotextile at the base of the permeable pavement structure, can most likely protect against any possible pollutant migration.

2.1.1.7 Pavement slope

Several Caribbean SIDS have steep urban catchments which play a deciding factor in the location of permeable pavement installations. The majority of laboratory studies on the infiltration performance of permeable pavements on slopes have suggested grades of 5% [19, 20] up to 10% [21, 22]. Few field studies have assessed the performance of permeable pavements on slopes exceeding 5% [6, 23]. Nevertheless, should sloping grounds be inevitable, internal check dams or berms can be incorporated into the subgrade to encourage infiltration. Underdrains may be placed at each dam location if required [24].

2.1.1.8 Clogging as a maintenance issue

The importance of permeable pavement maintenance and its relevance to the integrated stormwater management agenda of urban SIDS authorities cannot be stressed enough. The culture of poor maintenance practices of valuable infrastructure is still widespread throughout most Caribbean SIDS. The unavoidable consequences of significant under-investment and neglect of stormwater management systems over many years are increasingly visible and is always a subject of considerable public concern.

Periodic maintenance of PPS to ensure that they continue functioning as desired, requires monitoring for signs of distress which could alter the structural integrity and hydrologic/hydraulic function of the pavement. Some of these distresses include clogging, depression, rutting, edge restraint damage, ravelling, cracking, excessive joint width, joint filler loss and horizontal creep [16]. Numerous researchers have cited periodic maintenance as being fundamental to limiting clogging of PPS [25-27]. Examples of maintenance techniques include manual removal of the upper 20 mm of fill material, mechanical street sweeping, regenerative air street sweeping, vacuum street sweeping, hand-held vacuuming, high pressure washing, and milling of porous asphalt [28, 29].

Vacuum and street sweeping trucks are not readily available in SIDS and would attract significant importation costs. Consequently, this minimises the use of porous asphalt and porous concrete pavements given that vacuuming is the most effective maintenance option for these pavement types. Permeable pavements utilising interlocking concrete pavers is the preferred option in this regard, because maintenance options such as removal and replacement of the infill material [30] are more in line with Caribbean SIDS economics, having low start-up costs and can provide employment opportunities. Other relatively cheap options maintenance are through the utilisation of the hand-held power brush and the pressure washer [31].

2.1.2 Economic feasibility

2.1.2.1 Cost

The financing and cost-recovery of urban drainage systems remain a challenge to many Caribbean SIDS [4]. Initial costs of PPS usually exceed those of conventional pavements, primarily due to PPS having thicker aggregate layers necessary to maximise stormwater storage and to provide enough structural support to accommodate vehicle loading. However, a life-cycle cost analysis may realise actual cost savings with PPS when compared to conventional pavements if a holistic approach is considered. Savings and benefits include reduced need for conventional downstream stormwater infrastructure such as detention ponds and concrete ditches, less developable land consumed for stormwater ponds, improved aesthetics and reduced urban heat island effect [32].

2.1.3 Political

2.1.3.1 Institutional and legislative framework

Urban stormwater management is typically not prioritised and is often dealt reactively by policy makers and administrators. Moreover, institutions charged with enforcing existing stormwater management policies are often relaxed in their approach as evident in countries such as Trinidad and Tobago and St Lucia by the vast number of properties constructed on drain reserves and flood plains. Unplanned development near urban cities is widespread and often exacerbates flooding problems [33]. There is an increasing demand for improved drainage in society. Simultaneously, a desire for a clean environment, preservation of nature and concern for the welfare of future generations is also progressively salient. Policy makers and politicians across the Caribbean must be cognisant of these conflicting desires along with the added benefits of adequately managed SUDS as well as the various issues which may be confronted. This knowledge would seek to reduce the need for reactive spending and promote long-term integrated planning instead [34].

4. CONCLUSION

PPS reduce pollutants from infiltrating stormwater runoff, provide vital reservoir storage for potential reuse of stormwater and improve the hydrologic functions of various locations. Unlike most territorial states, the geographically and geologically confined nature of most Caribbean SIDS present unique parameters for consideration when designing permeable pavements. Some of the most important considerations include traffic loads, design storms, cost, choice and availability of construction aggregates, permeability of existing soil at the intended location, depth of water table, potential for groundwater contamination, slope of pavements, clogging, infrastructure maintenance and support from policy makers. Reluctance to implement PPS in SIDS may include technical uncertainty in performance, lack of reliable data as well as social perceptions. The implementation of successful PPS in SIDS depends heavily on aggressive enforcement of policies relating specifically to PPS and SUDS in general.

COMPETING INTERESTS

The authors have declared no conflict of interest.

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Analysing climate gentrification in coastal neighbourhoods: A case study of Lagos, Nigeria

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ABSTRACT

The concept of climate gentrification emerged to redefine our understanding of how climate change impacts (sea-level rise, flooding, water storms, tsunami) and adaptations drive inequality in human settlement and probable displacement of low-income households through changes in housing property value. However, the concept of climate gentrification lacks adequate parameters for application in diverse coastal locations. In response, this qualitative case study proposes a climate resilience integrated approach (framework) for identifying the parameters to analyze climate gentrification in the coastal neighborhoods of Lagos (Nigeria). In doing so, a pilot investigation was conducted using naturalistic observation to explore events and lived experiences of residents in Lagos coastal neighborhoods. Findings indicate a preference for built/engineered resilience infrastructures and higher return on physical asset investments as core variables driving climate gentrification patterns in Lagos coastal neighbourhoods.

Keywords: *Climate Change, Sea Level Rise, Flooding, Climate Gentrification, Housing Displacement.*

1. INTRODUCTION

Climate change poses a significant risk to human settlement and urban housing in coastal cities. Because coastal cities have low-lying geographical elevation which makes them vulnerable to the impacts of rising sea levels: flooding, water storm, tsunamis, etc. [1][2][3][4]. Hence, urban housing in vulnerable neighborhoods in coastal cities would require adequate adaptation to be habitable. Especially, in developing climes like Lagos (Nigeria); where adequate flooding adaptation is unconvincing due to socio-economic constraints [4]. However, recent findings have shown that the direct impact of flooding or the cost burden of pursuing adequate adaptation could result in unintended maladaptation for vulnerable low-income socio-economic groups (households). This usually manifests as skyrocketing housing property prices, rents, and cost of living, which may be unbearable for the low-income socio-economic groups who could eventually be displaced from their original settlements [2][4][5][6]. To this end, climate gentrification emerged as a contemporary paradigm to redefine our understanding of how climate change impacts drive low-income housing displacement in coastal neighbourhoods [2][4]. But the problem with climate gentrification is that the concept lacks sufficient and robust parameter(s) for operationalization [2][4][7]) especially, in coastal locations with a distinct character like Lagos, Nigeria. This limitation is the most highlighted among other issues identified in the systematic literature review of this study (see **table 1**). Specifically, scholarly submissions have underpinned the need to integrate the built/engineered resilience of physical infrastructures as an alternative axis for analysing climate gentrification in coastal locations [2][4]. Hence, this qualitative case study proposes a climate resilience integrated approach to explore the experiences of relevant stakeholders in coastal neighbourhoods as a precursor to finding the parameter(s) for analysing the manifestation of climate gentrification.

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Table 1. Research gap in climate gentrification studies*

Type	Study Direction to Address Gaps/Issues**	Literature Source
Approach/Concept Gap	Define more parameters for operationalizing the concept of climate gentrification in diverse coastal locations	*Keenan et al (2018); Wiggins, (2018); Aune et al (2020); Wang et al (2021); Mosgen et al (2019); Bond and Browder (2019); Moretti, (2012), Kjaer (2015).*
	Integrate geographical elevation and built (engineered) resilience in Analysing the manifestation of climate gentrification in coastal locations.	
Knowledge Gap	Extend the concept of climate gentrification to housing studies.	
	Deepen the understanding of human heuristics driving climate gentrification in coastal communities.	

* Research gaps were identified through a systematic literature review of existing climate gentrification studies.

** This study focused on addressing the approach/concept and knowledge gaps.

This study should be viewed from the contextual lens of flooding impacts, urban housing, and the recent global financial constraints. Also, this study focused on Lagos coastal city because it possesses distinct elevational character i.e., moderate-low exposure to sea-level rise [8]; yet experiencing climate gentrification patterns like extreme flooding impacts, increasing influx of affluent households, skyrocketing housing prices/rents, expensive cost of living, socio-spatial changes, etc [9][10][11]. Also, current trends suggest that the coastal neighbourhoods in Lagos are targets for rent-seeking, capital real estate investments, and government revitalization and upgrading projects. But by abduction, the rising preference and influx of affluent households into Lagos coastal neighbourhoods contradicts the common observation of an increasing preference for housing properties on higher grounds in some other coastal locations [2][4][12]. This study is prompt at a time when Nigeria is struggling to address a huge housing deficit (>8 million) and a projected cost (6.17 – 12.34 billion USD) for climate change impact adaptation at the end of the 21st century [10][13][14]. Findings will expand the knowledge of the climate gentrification phenomenon in Nigeria and Africa; shape future studies and redefine housing intervention policies that would address low-income housing displacements in coastal neighbourhoods.

2. METHODOLOGY

The concept of climate gentrification falls within the tenets of humanistic/naturalistic studies [2][15][16][17]. Also, the purpose of this study is exploratory and based on the interpretive belief (philosophy) of analysing climate gentrification from lived experiences. Beyond philosophical justifications, a review on the methods applied in existing climate gentrification studies shows that different methods abound. To this end, a conceptual framework (see **figure 1**) is proposed to guide the practical procedures for data collection, analysis, and finding the parameter(s) for analysing climate gentrification in coastal neighbourhoods.

The proposed conceptual framework is based on the following propositions:

First, *The manifestation of climate gentrification in urban coastal locations is moderated/mediated by climate resilience [2][4].*

Second, *The manifestation of climate gentrification is a non-linear process of maladaptation i.e., interactions between exposure and adaptative capacity [2][4][5][12].*

Third, *Geographical elevation (natural) and built/engineered infrastructures are climate resilience variables that moderate/mediate the manifestation of climate gentrification in a coastal location [2][4].*

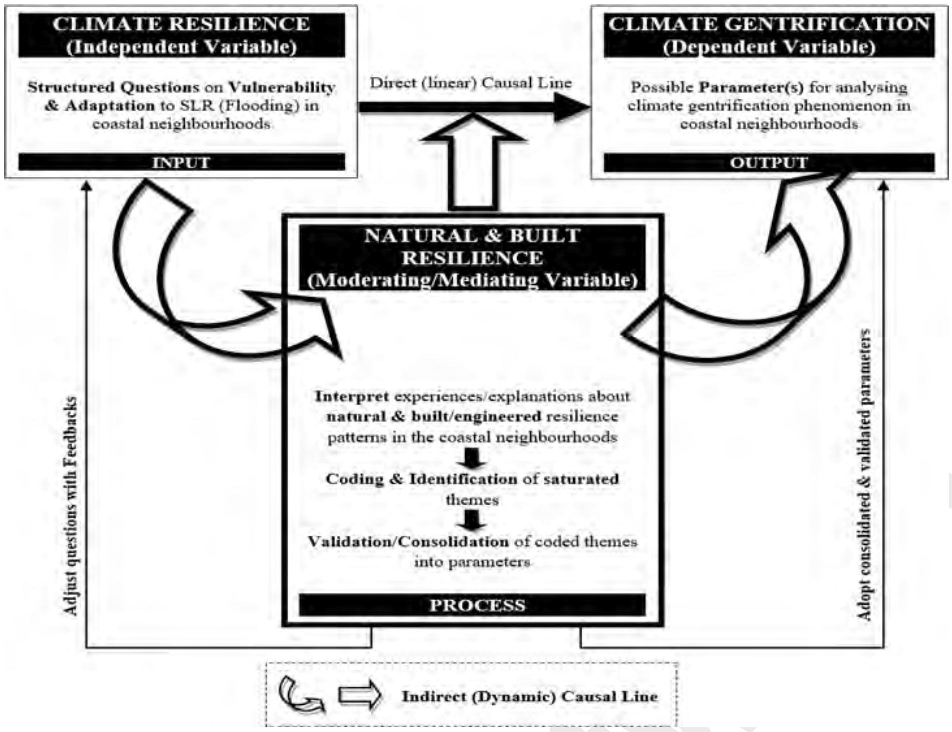

Figure 1. Proposed conceptual framework for identifying the parameters to analyse climate gentrification in Lagos coastal neighbourhoods.

2.1 Data Collection and Analysis

Observation techniques were applied in recording relevant events in the coastal neighbourhoods of Lagos. Images (photographic and cartographic) were gathered using high resolution digital cameras and Quantum Geographic Information System (QGIS). Also, conversations with residents were recorded with field notes. The observation served as a reconnaissance of the coastal neighbourhoods in Lagos. Also, as a pilot to evaluate the data collection instruments for subsequent investigations (focus group discussions and in-depth interviews). The observations focused on six (6) neighbourhoods (Ikoyi, Victoria Island, Maroko, Apapa, and Makoko) drawn from two (2) strategic locational demographics: Lagos Island and mainland (see **figure 2**). This was to capture the diverse character of coastal neighbourhoods in Lagos.

Images and field notes were subjected to content analysis to identify common patterns. The identified patterns were coded as variables driving the manifestation of climate gentrification in Lagos coastal neighbourhoods.

3. RESULTS AND DISCUSSION

- Responses from direct conversations with residents across the sampled neighbourhoods converged significantly on two codes: *physical infrastructures* and *physical asset investments*. Respondents narrated that there is a preference for upgraded housing (built) environment and associated physical infrastructures; better drainage systems for flood management; and high return on physical asset investments. Specifically, respondents recounted how the outlined (codes) variables are driving the influx of high-income households into the coastal neighbourhoods; and whose presence skyrocket housing property prices and living cost. For instance, a respondent in Ikoyi stated: *'it is celebrities, politicians, and rich businessmen that live in this area; and that is why it is very beautiful, and the accommodation is very expensive'*.
- Digital Elevation Model (DEM) images (see **figure 2**) of the coastal neighbourhoods in Lagos shows that coastal neighbourhoods in the lowest elevation are experiencing increase in human settlement, despite their high exposure to flooding impacts. Also, Photographic image records (see **figure 3-right**) and open street maps from ArcGIS (see **figure 3-left**) shows that the coastal neighbourhoods (Ikoyi, Victoria Island, Maroko, and Lekki) in the lowest elevation possess a more upgraded housing (built) environment,

physical infrastructures, and drainage systems for flood management than those nearest to higher grounds (Makoko and Apapa). These patterns explains the reason why the coastal neighbourhoods within the lowest elevation are experiencing more influx of high-income households. Hence, the manifestation of climate gentrification patterns increased towards the coastal neighbourhoods in the lowest geographical elevation.

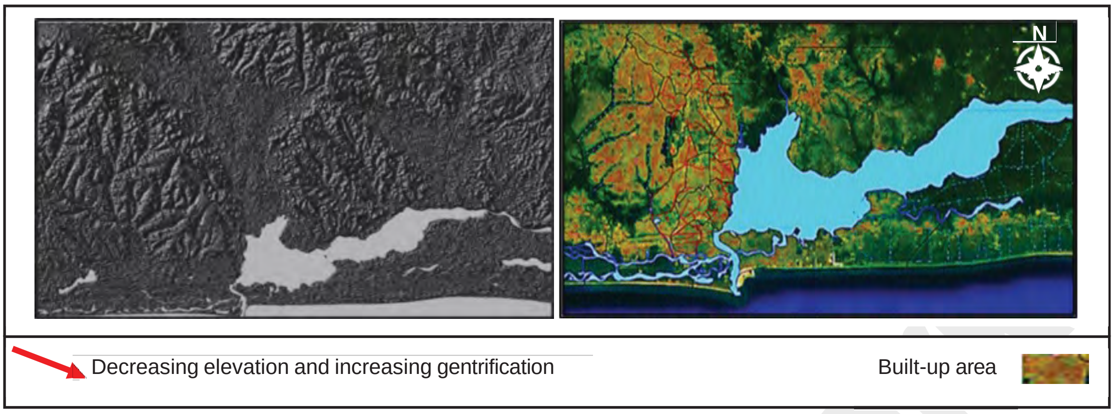

Figure 2. Digital Elevation Map (SRTM-DEM) overlayed with built-up areas across the coastal neighbourhoods in Lagos, Nigeria.

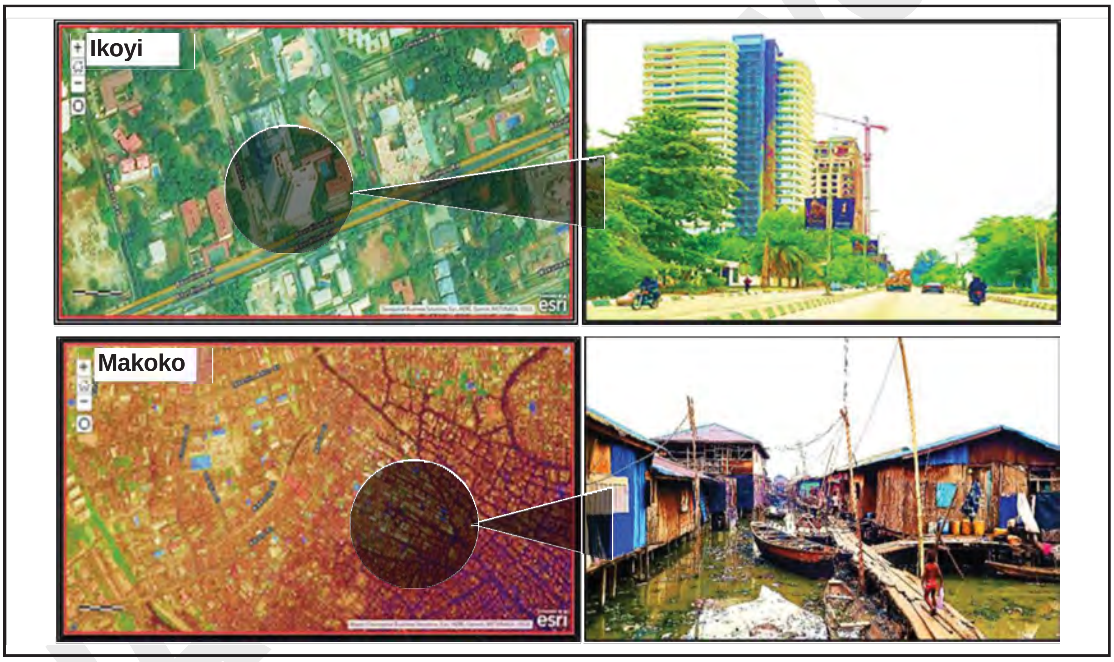

Figure 3. Photographic images (right) and ArcGIS open street maps (left) of a coastal neighbourhood in Lagos Island (top) and Lagos mainland (bottom).

4. CONCLUSION

This study proposes a conceptual framework for identifying the parameters to analyse climate gentrification in Lagos coastal neighbourhoods. Findings show that the manifestation of climate gentrification is relative to preferences for built/engineered resilience infrastructures and higher return on physical asset investment. Also, the manifestation of climate gentrification patterns increased towards neighbourhoods (Lagos Island) with lower elevations. The findings support the proposition to analyse climate gentrification based on built/engineered resilience infrastructure.

ACKNOWLEDGEMENTS

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COMPETING INTERESTS

All authors declare no conflict of competing interests.

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Multi-Step Flood Forecasting in Urban Drainage Systems Using Time-series Data Mining Techniques

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ABSTRACT
While early warning systems are recognised as the most cost-effective solution in urban flood risk management, highly accurate flood forecasting is limited to short-term timesteps, usually less than a few hours especially for prediction of overflowing in urban drainage systems . This study aims to provide a framework for more accurate overflow predictions for longer lead times by using data mining models applied to time series data for multi-step flood forecasting. The framework including event identification, feature analysis and developing models is demonstrated by its application to a pilot study in London. All numerical rainfall data and water levels in urban drainage systems are first turned to the categorical events on which 6 common weak learner models are developed. Then, three new time-series models, including overflowing-based, non-overflowing-based, and accuracy-based, are developed based on these models to predict overflow states among all identified events. Three weak learner models, i.e. discriminant analysis, naive Bayes, and decision tree are considered as the best models based on accuracy, total overflowing detection and total non-overflowing detection. Furthermore, while the accuracy of these models is changed between 95 to 85% from 1 to 12-step ahead of prediction, these models can detect the non-overflow conditions better than overflow detection. To cover this gap, new time series developed models could significantly reduce the overestimation and underestimation of water levels, including correct predicting of 50% of the total events after 12-step ahead by overflow-based model. This result shows the potential of using time-series data-demanding models for effective and highly accurate predictions of overflow events.

Keywords: Data mining; Drainage system; Flooding classification; Multistep prediction Overflow prediction

1. INTRODUCTION
Flooding is recognised as a worldwide natural hazard, which is responsible alone for over 30% of global economic loss and 60% of the total affected people by all types of natural hazards [1]. According to UNDRR [2], the number of flood occurrences increased in the recent 50 years (Figure 1a), in which more than 3.5 billion people have been affected and near 1,750 billion pounds loss is estimated so far (Figure 1b). Therefore, early warning systems can now commerce to a reliable and practical solution for predicting floods' overflowing of drainage systems by using weak learner data mining models (WLDM). However, they are unable to forecast flood for long time steps due to complicated non-periodic and chaotic mechanism of rainfall occurrence and weak correlation between flooding and drainage systems' water level [3]. Previous research works applies WLDM such as support vector machine (SVM), k-nearest neighbourhood (KNN), discriminant analysis (DA), decision tree (DT), Gaussian process regression (GPR), naive Bayes (NB), and neural network pattern recognition (NNPR) to forecasting water level in urban drainage systems (UDS) [4,5]. They were also used to determine overflow conditions in which the flow in UDS exceeds the full capacity of UDS and spills into urban areas and causes flooding. However, other data mining models (DMs), particularly feedback forward and recurrent neural network (RNN) models show more potential due to representing a significant forecasting accuracy, handling big data and high-speed computation [6]. Despite the good performance of these DMs, the accuracy of forecasting overflow in these models for periods longer than 60 minutes is reduced significantly [1]. To tackle this, hybrid models have been developed recently in which WLDM are mixed with RNN

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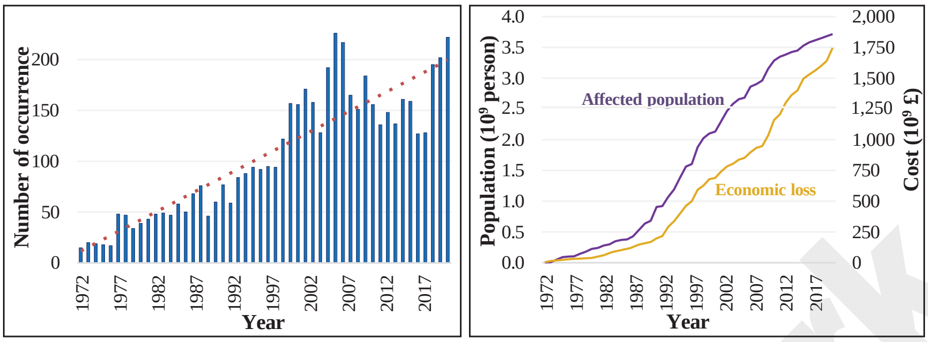
[7]. While these models could increase the accuracy of overflow prediction in near lead time, they are still unable to provide reliable and accurate estimation for lead time longer than two hours. [8,9]. Additionally applied WLDM focus heavily on increasing model accuracy in only one specific time step [10] instead of following the concept of time-series DMs for multi-step prediction [11,12].



(a) (b)

Figure 1. Recent 50-year recorded flood events all around the world: a) number of flood occurrences, b) associated cumulative social and economic loss

Hence, the present study aims to propose flexible time-series WLDM models to fill the above gaps, including (1) providing more accurate overflow predictions for longer lead time, and (2) using the concept of time series DMs to find the best method for multi-step prediction, (3) investigating model performance on not only different lead times, but also multistep overflow detection of flood events. This model can enable the development of a high-speed and outperformed real-time overflow classification model that can be trained based on a limited temporal set of data, i.e. only rainfall and water level in UDS. The proposed framework along with the case study in the UK will be described in the next section followed by presenting research findings, critical discussion, and finally highlighting key findings and final remarks in conclusions.

2. METHODOLOGY

The proposed framework as shown in Figure 2 comprises two main parts: (1) data collection and preparation, and (2) model development and performance assessment. The time-series of rainfall and water level data of UDS (described in section 2.1) collected from a public domain online database and their missing data are infilled by linear regression, are used to identify both flood and non-flood events [13]. Identified numerical events are then turned into categorial order, named hereafter featured events, through the method proposed in Section 2.2. Several widely used WLDMs are developed based on these featured events which are introduced in Section 2.3. The time-series data mining models for predicting overflow conditions in floods and non-floods events are developed and evaluated based on the model performance criteria introduced in section 2.4.

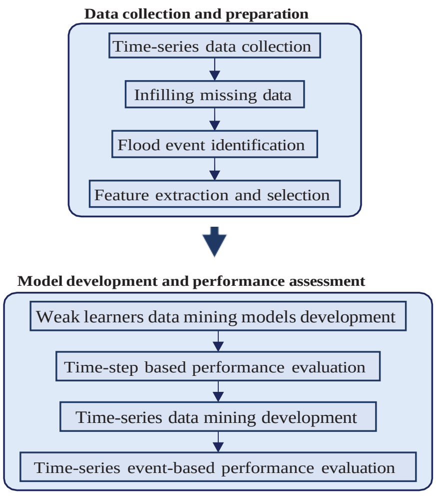


Figure 2. The proposed framework for flood overflow detection

2.1 Data collection and study area

The proposed framework is demonstrated here through its application for forecasting flood overflow in a real-world UDS pilot study in the UK. Figure 2 shows the entire catchment area located in the London Borough of Hillingdon including the Ruislip urban catchment area analysed in this pilot study. The Ruislip UDS drives the Colne catchment surface runoff from south Hertfordshire to a tributary of the River Thames in England. The UDS located in the northwest of London collects the surface runoff through the river Pinn from a catchment area of 13 km^2. The pilot study was selected due to its vulnerability to frequent fluvial flooding over the Ruislip urban neighbourhoods. Ruislip gauging station in the river Pinn located at the outlet of the Ruislip UDS is one of the 55 gauging stations installed in the Colne catchment area and is responsible for measuring and recording the water level. An ultrasonic depth monitor system is used to record the time-series of water level every 15 min at the station since 2009 [14]. Furthermore, the rainfall observed every 15 minutes at RAF Northolt rain gauge station, shown in Figure 2, is also used here [14]. The entire database includes 365,233 data for both rainfall and water level, 15-minute time intervals and a continuous duration of 11 years (2009-2020) are accessible through the application programming interface (API) of the UK Environment Agency [14].

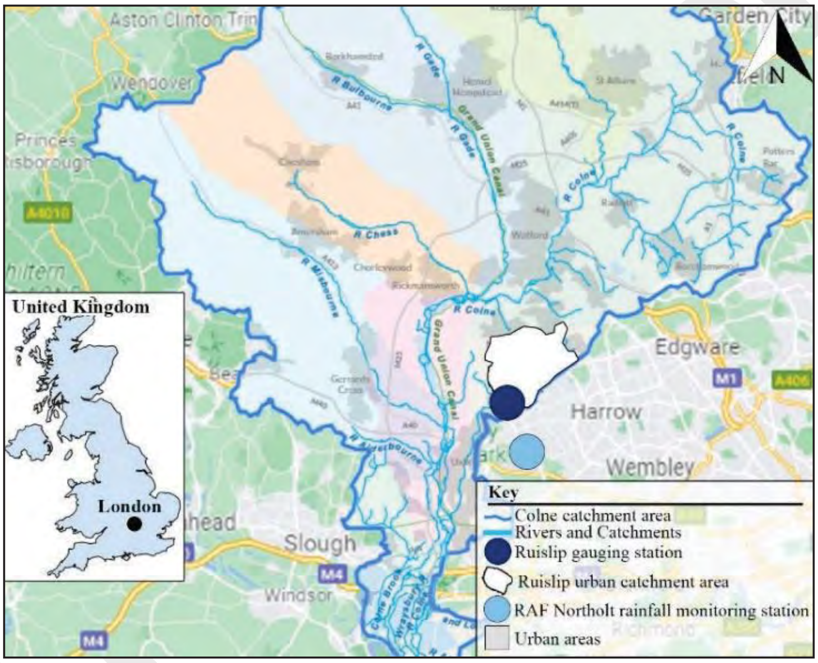


Figure 2. The layout of the case study and location of rainfall and gauging stations

2.2 Feature extraction and selection

Based on rainfall and water level records, data are firstly converted to (1) overflow events i.e., rainfall occurrence causes water level rising in the UDS and overflowing, (2) non-overflowing events, i.e. despite rainfall, there is no water level rise as outlined in the event identifications [13]. Then all identified events are converted into several features of rainfall events listed in Table 1. Principle component analysis [15] is used as criteria (See Figure 3) to determine the final parameters including (1) rainfall duration, (2) rainfall intensity, (3) intensity of previously occurred rainfall, (4) season of the event and (5) overflowing state.

Table 1. Extracted and final selected features for turning identified events to featured events

Group feature	Extracted feature	Description	Type of used data	Transformation
Rainfall data	Duration (F_1)	Duration of rainfall occurrence in the area of interest	Actual	Timestep (Every 15 mins)
	Total depth (F_2)	Liquid precipitation covering a horizontal surface area of interest	Actual	mm
	Intensity (F_3)	The average rainfall rate for a specific duration	Actual	mm/hr.
	Peak depth (F_4)	The maximum amount of rainfall Intensity	Actual	mm
Previous rainfall	Occurrence (F_5)	Evidence showing rainfall occurred before the current time	Categorised	0 (No) 1 (Yes)
	Intensity (F_6)	The average rainfall intensity of previously recorded rainfall	Actual	mm/hr.
Date of the year	Season (F_7)	Time of the year	Categorised	1(Dry) 2 (Mild) 3 (Rainy)
	Long-term history (F_8)	Average rainfall intensity of this date for the past 10 years	Actual	mm/hr.
Overflow	Existence	State of water level in comparison to full capacity of UDS	Categorised	Class 1 (No overflowing) Class 2 (Overflowing)

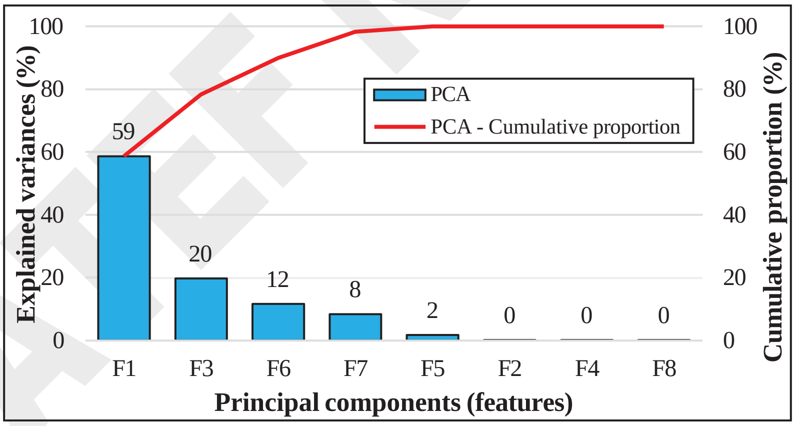


Figure 3. Principal components analysis (PCA) on extracted features

2.3 Developed data mining models

Based on the literature, 7 WLDMs are selected to develop here, including: (1) DA, (2) DT, (3) GPR, (4) KNN, (5) NB, (6) SVM, and (7) NNPR. Models are developed based on 2 classes (1) non-overflow state, and (2) overflow detection. Time-series models are also developed based on the process shown in Figure 4, in which 12 timesteps of each event are predicted by a specified developed model. Here, WLDMs are used to build three time-series models based on the best overall accuracy (called "ACC" model), the best overflow detection (called "TPR" model) and the best non-overflow detection (called "TNR" model) of previous developed WLDM. For this purpose, among all developed WLDM, the best model is used for the prediction of the event's class in first time step ahead, and then this process goes continued iteratively for further time steps.

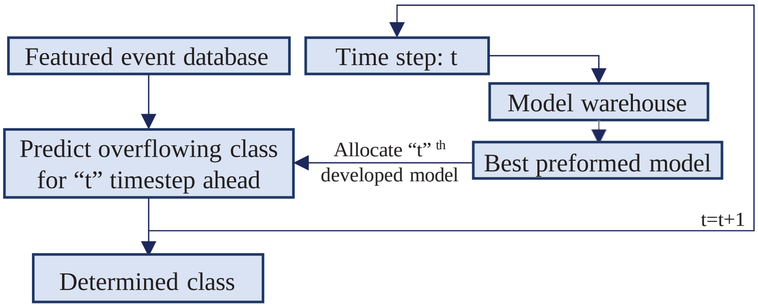


Figure 4. Iterative process of developing time-series data mining models

All models are built for prediction from 1-step to 12-step ahead using MATLAB 2021a and then individually optimised based on 30 different iterations. 75% and 25% of total events are used for building WLDMs and time-series models, respectively. All WLDM models are developed based on 70% (53% of the total database) for model training, 15% (11% of the total database) for validation and 15% of data for test. In each timestep, training, validation and test dataset are generated randomly based on characteristics of the entire database, meaning 70% of non-overflow events and 30% of overflow events. All developed WLDM models are stored in a model warehouse (library) used then for developing time-series models.

2.4 Key performance indicators

Performance of WLDM models is evaluated by using three main indicators listed in Table 2, including (1) accuracy, (2) total correct detection of overflow events, and (3) Total correct detection of non-overflow events. Besides, developed time-series models evaluated based on the confusion matrix shown in Figure 5, consisting of (1) hit rate, predicting correct event's class in correct timestep, (2) miss rate, underestimated prediction in both event class and timestep, (3) Over rate, overestimated prediction in event class, (4) acceptable rate, predicting correct event class in correct or earlier timestep.

Table 2. Key performance indicators used for performance assessment of WLDM models

Metric	Covered concern	Equation	Range
Accuracy (ACC)	Probability in that the model prediction is correct, i.e. interested in predicting the right classes without caring about the type of the class or class distribution.	$\frac{TP + TN}{n}$	[0,1]
Total positive rate (TPR)	Sensitivity of model in recalling actual overflow condition, i.e. accuracy of overflow class	$\frac{TP}{TP + FN}$	[0,1]
Total negative rate (TNR)	Specificity of the model in selecting actual non-overflow condition, i.e. accuracy of non-overflow class	$\frac{TN}{TN + FP}$	[0,1]

TP: True overflow detection TN: True non-overflow detection n: total number of events
FP: Non-overflow event is detected as an overflow condition
FN: Overflow event is detected as a non-overflow condition

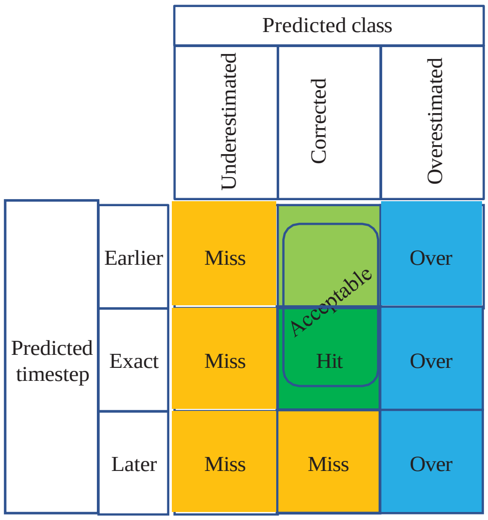


Figure 5. Structure of time-series event-based performance assessment

3. RESULTS AND DISCUSSION

Performance of developed WLDM models for different prediction timesteps (1-12 timesteps ahead) are demonstrated in Figure 6 and the best model performance is indicated in Table 3. Overall, ACC and TNR reduced from near 95% to 80% from 1 timestep to 12 timesteps for all models (Figure 6a and 6C), whereas TPR dropped up to 50% for the longest lead time (Figure 6b). While the number of the observed non-overflow event is 3 times more than overflow events, high accuracy of TNR shows that developed WLDM models are more capable to detect non-overflow events than detecting correct overflow conditions. Furthermore, Table 3 indicates that no absolute and unique WLDM model can perfectly show the best performance in comparison to other models. For example, although DA is recognised as the best ACC model, positioning the first rank in 6 out of total 12 timesteps with an average of 88.65%, it could not obtain the best TPR or TNR score, whereas NB and DT models have the best performance in these metrics, respectively.

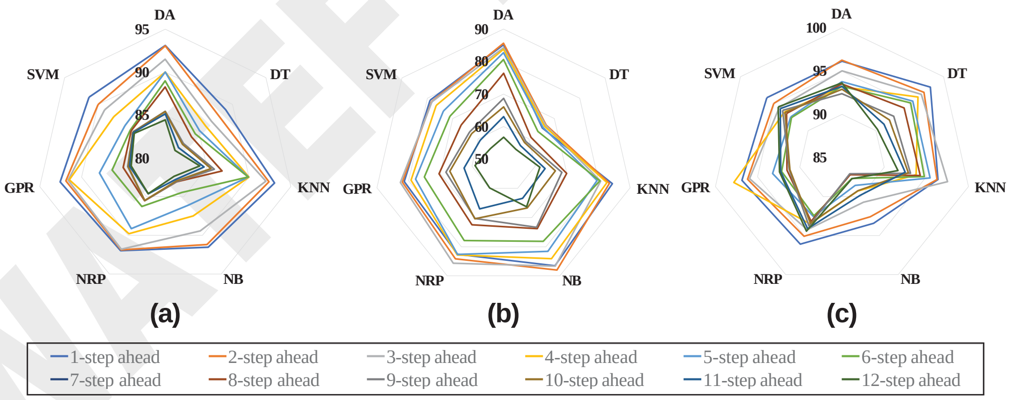


Figure 6. Performance of WLDMs in different prediction timesteps: (a) ACC, (b) TPR, (c) TNR

Based on the recognised best WLDM models for all metrics, the performance of these models is investigated in time-series modelling, which is shown in Figure 7. While the accuracy of exact prediction, i.e. correct detection of events, is generally reduced from around 85% (1 step ahead) to near 80% (12 steps ahead) for all models, the TPR-based model shows slightly better performance in which the accuracy is about 80% for all time steps. However, these models are distinguished from each other in the rate and trend of underestimation and overestimation accuracy. The ACC-based model for longer lead times tends to overestimate flood forecasting while the TNR-based model has more underestimated flood forecasting. Although the result of the TNR-based model was expected because of the ability of WLDM models to better prediction of non-overflow events in comparison to overflow events, it was expected that the TPR-based model has more underestimated forecasting because of using WLDM with the lower range of TPR rather than TNR and ACC score. However, flexible use of WLDM in time-series models could overcome this gap and shows better performance on the low range of both overestimated and underestimated predictions.

Table 3. Best WLDMs based on the key performance indicators

Time step	Best developed model		
	ACC	TPR	TNR
1	**DA** (93.10%)	**NB** (86.56%)	**DT** (98.07%)
2	**DA** (93.06%)	**NB** (87.93%)	**DT** (97.13%)
3	KNN (92.03%)	**NB** (86.68%)	KNN (97.51%)
4	GPR (91.47%)	**NB** (84.07%)	GPR (97.81%)
5	**DA** (90.01%)	DA (82.74%)	**DT** (95.49%)
6	KNN (89.92%)	DA (80.61%)	**DT** (95.16%)
7	**DA** (88.42%)	**NB** (74.63%)	DA (95.39%)
8	**DA** (88.27%)	DA (76.42%)	KNN (94.29%)
9	KNN (85.86%)	**NB** (73.35%)	SVM (93.77%)
10	NRP (85.47%)	NRP (70.44%)	SVM (93.41%)
11	**DA** (85.15%)	NRP (67.04%)	SVM (94.10%)
12	SVM (84.64%)	**NB** (66.29%)	SVM (94.42%)
Best model[1*]	**DA (6[2*], 88.65%[3*])**	**NB (7[2*], 76.94%[3*])**	**DT (4[2*], 94.47%[3*])**

1*: Best model is selected based on the Friedman test for all 12 timesteps
2*: Frequency of best model among total 12 timesteps
3*: Average value for all 12 timesteps

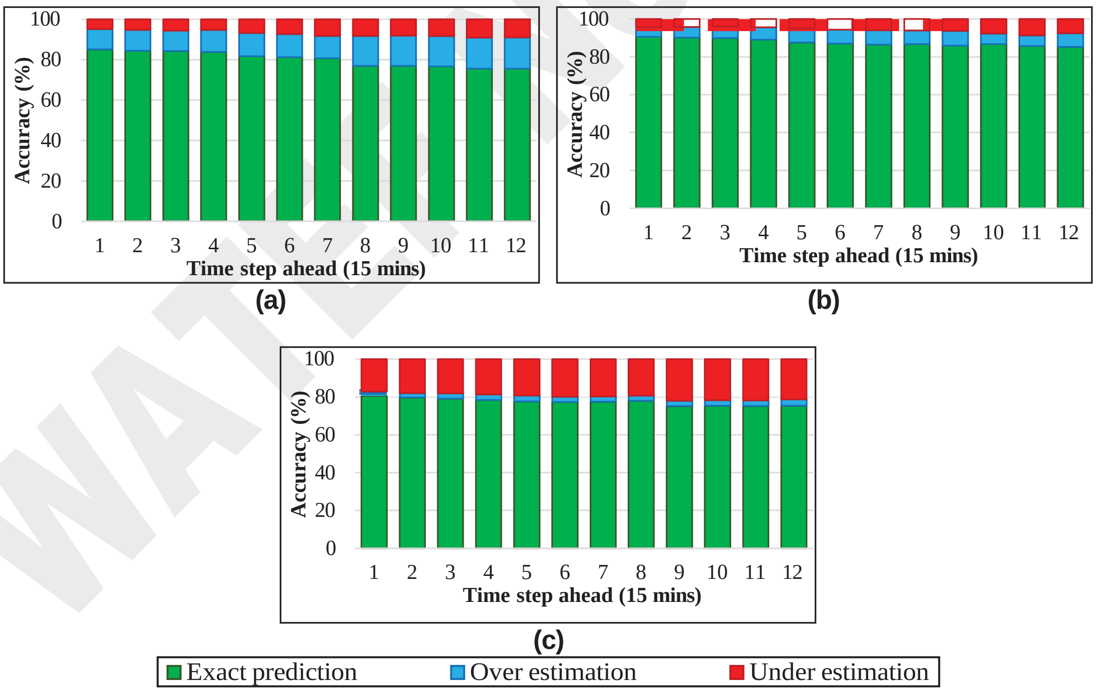


Figure 7. Performance assessment of developed time-series models in each time step: (a) ACC-based, (b) TPR-based, (C) TNR-based

Model performance should be also investigated for all duration of events, as shown in Figure 8. Results show that the miss rate of prediction is still low for both ACC-based and TPR-based models (less than 3% in Figure 8a). However, this rate suddenly increases to 25% for the TNR-based model. On the other hand, as can be seen in Figure 8b, 13% of total events have overestimation forecasting in the ACC-based model, whereas this rate is reduced to 5.68% and 2.44% for TPR-based and TNR-based, respectively. Finally, while hit rate is quite low for all models (42%, 22% and 11% for TPR, ACC, and TNR models, shown in Figure 8c, respectively), acceptable rate illustrated in Figure 8d shows these models can satisfactorily predict the correct class, meaning overflowing or not overflowing, in correct timestep or slightly earlier. More specifically, the TPR-based model could show a 91% acceptable rate with just an average of 1.13 timestep lag (average of all lagged time between actual timestep and predicted timestep of true predicted class). These promising results can show time series models, particularly the TPR-based model, can be simply but effectively applied for early warning overflow detection systems.

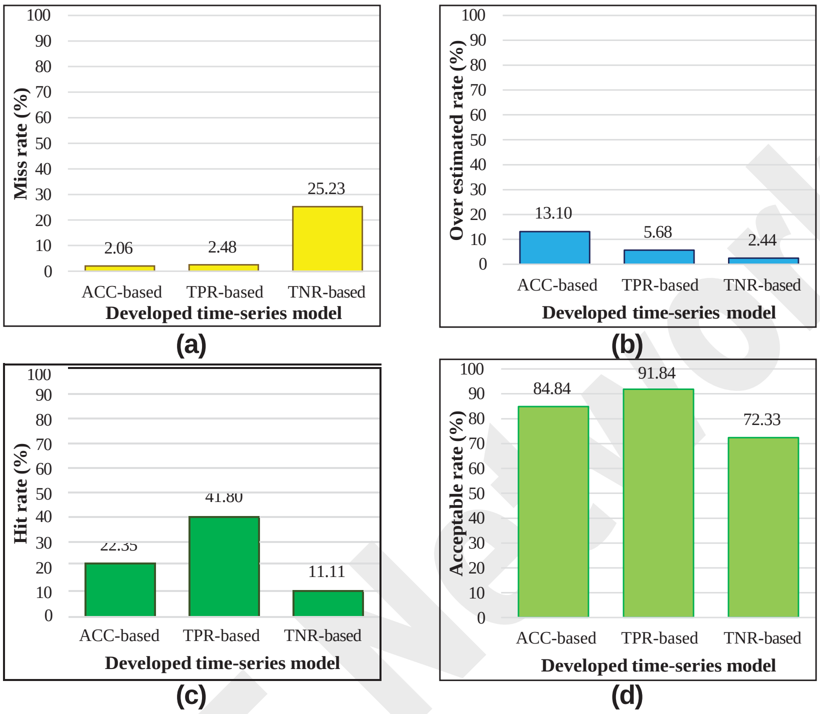


Figure 8. Event-based performance assessment of developed time-series models: (a) Miss rate, (b) Over estimated rate, (c) Hit rate, (d) Acceptable rate

4. CONCLUSIONS

The present study provides a framework for developing WLDMs followed by time-series models based on ACC, TPR and TNR metrics, for early warning overflow detection systems. Analysis of WLDMs, applied for the real case study, shows that none of the selected models could outperform each other for all metrics or for all prediction timesteps. While models indicating better ACC rather than TPR are more capable in detecting non-overflow events than overflow conditions (80-95% vs 60-90% accuracy of TNR and TPR). However, time-series models, especially the TPR-based model, could cover this accuracy by choosing the best WLDM in each time step, which result in reducing overestimation and underestimations for different timesteps of prediction as well as reflecting more than 90% of the acceptable rate. Hence, the application of time-series data mining models can enable the development of a high-speed and real-time overflow classification model that can be trained based on limited features obtained from only rainfall and water level in UDS. However, the applied concept requires further studies such as using advantages of ensemble modelling and involving more input decision variables, especially temperature, soil moisture and wind characteristics.

COMPETING INTERESTS

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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#2. Resources, environment, and climate change

Suitability of the SCS Type Temporal Distributions for Local Rainfall in Trinidad and Tobago, West Indies.

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ABSTRACT

Stormwater management facilities are intended to convey the peak flows generated by some critical storm. A major feature of that storm controlling the resulting peak flow is its temporal distribution, that is, how the rainfall depth is distributed in time. The chosen distribution should be representative of the rainfall observed in the local vicinity of the facilities, and this is obtained by Analysing fine temporal resolution records (between 5 minutes to 1-hour intervals) collected from local rainfall stations. Often, the only records available are ones collected on a daily scale, which are too coarse for the small watersheds that typify small island states. For estimating design peak flows, designers frequently refer to the temporal distributions published within the SCS peak flow estimation procedure. The problem is these temporal distributions were developed for the United States and they may be markedly different from local distributions. This study analyzed fine resolution data from a few stations in Trinidad. It found that the representative temporal distributions were bi-modal, unlike the strong uni-modal distributions in the SCS procedure. A comparison of peak flows derived from the various distributions found that the NRCS distributions over-estimated peak flows by more than 100%. Although this may suggest the possibility of oversizing infrastructure for drainage, caution is required in realizing that while not frequent, from time to time, recorded storms have mimicked the SCS curves. Clearly the work needs to be extended to consider longer rainfall series, from a larger number of rainfall stations across the country.

Keywords: Rainfall temporal distribution; SCS hydrologic procedure; oversizing infrastructure; HEC-HMS; Trinidad and Tobago

1. INTRODUCTION

It is imperative that engineers and hydrologists evaluate the design rainfall for designing hydraulic structures. This requires estimation of the design flood. Good estimation can only be done by a sound representation of the time distribution for the rainfall event to produce the design storm that is subsequently used to create the runoff hydrograph [1]. Temporal distribution of rainfall can be defined as the distribution of the rainfall depth with respect to time during a given rainfall event [2]. Typically, the Soil Conservation Service (SCS) Type II curve is used for the rainfall distribution of many regions. The Soil Conservation Service is an organization in the United States that focuses on agricultural practices where one of its specializations is drainage and irrigation work. This organization developed four (4) cumulative curves: Type I, Type IA, Type II and Type III, valid for the various climates of the United States and based on the derivation of hypothetical storms for the rainfall patterns. The curves were

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developed for the 24-hour duration events with a conversion for 6-hour events; but they are dimensionless curves that can also be scaled to other durations [1].
The use of data that was not developed for the climate of interest poses questions on the representativeness of the rainfall and might be problematic for use for the hydraulic design of structures. Furthermore, researchers have questioned the validity of the Type II curve (most popular among the four distributions) and proposed developing their own rainfall distribution pattern based on actual rainfall events in their local environment. [1] theorizes that patterns developed based on actual rainfall events would be able to produce a better representation of the temporal distribution of rainfall as opposed to the SCS temporal curves. To begin, the SCS curves were developed based on rainfall bursts during a storm event and not the actual rainfall event ([1], [3]). Moreover, authors have recognized that these design curves are being used with no consideration on how they had been created [1].

[3] states that the various Soil Conservation Services (SCS) curves, especially the Type II, is used by many practitioners in arid regions for hydraulic modelling. Similarly, the SCS Type II curve is the recognized distribution of the arid region of Saudi Arabia, according to [4], despite these curves being designed specifically for the US and similar temperate regions [3]. These researchers further stated that there is no justification for using the SCS Type II curve in other regions of the world. Yet still most practitioners do. A critically important question is whether the SCS Type II curve is an accurate and valid representation of the temporal distribution of rainfall in the place of interest.

Researchers ([1], [3], [4], [5]) conducted studies and investigated this conventional use of the SCS Type II compared to their respective design curves. [1] discovered that the designed Short Pattern Curve and Long Pattern Curve had an earlier and later peak intensity respectively when compared with the SCS Type II curve and that studies conducted in other parts of the US displayed similar trends to each other, however, and they all differed from the SCS Type II curve. Similarly, [4] discovered that the curves developed were steeper in comparison to the SCS Type II curve and [5] developed upper, middle and lower cluster curves where each differed from the SCS Type II curve. The runoff hydrograph highlighted by [5] noted that the SCS Type II curve produced a peak discharge 1.8 times greater than the middle cluster curve where 61 % of the observed storm distributions were included. Likewise, [3] highlighted that there was an overestimation in the hydrographs using the SCS Type II curve. The series of studies have unanimously concluded that the designed RDPs differed from the SCS Type II curve and that the SCS Type II curve was not very suitable as it did not accurately represent the rainfall of the area.

Design Storms are the basis for sizing drainage channels and the development of effective hydraulic structures conveying flows from catchments [1]. This is largely dependent on the determination of the design rainfall, which is the distribution over the storm duration of the rainfall depth for the chosen recurrence interval ([1], [2], [3],[4], [5]). As stated previously, these regions employ the use of the SCS Type II curve as the rainfall distribution. However, it was highlighted that this curve tends to overestimate the peak discharge of the hydrograph [3] resulting in the capacity of the hydraulic structure being overestimated ([1], [5]), increasing the cost and making it uneconomical [3]. By creating the temporal distribution from local daily rainfall records, a more accurate pattern of the rainfall distribution on a daily basis would be obtained and allow for better sizing of hydraulic structures.

[5] advanced studies on rainfall distributions by evaluating the trends of rainfall extremes so as to better understand the effects of climate change. This was achieved by using statistical trends namely the Mann-Kendall and Sen’s Slope Test to analyse the changes of the amplitude in addition to the frequency of said rainfall extreme events. The results of these tests indicated that there was insignificant trends or variation with respect to climate change. Additional studies to understand more completely the key aspects of climate change and the daily temporal distribution are required in the field [2].

According to [3], before designing hydraulic and hydrologic projects, the foundation of any project should involve establishing the Rainfall Distribution Patterns (RDPs). Understanding the methods employed by various researchers in creating RDPs would effectively aid in developing a method for the temporal distribution of daily rainfall in Trinidad. Several researchers around the world ([1], [2], [3], [4], [5]) conducted studies on temporal distributions using discrete rainfall records collected within their study areas.

This study seeks to develop the temporal distribution of daily rainfall of Trinidad and compare it with the SCS Type II curve, which is used where only daily rainfall might be available. The consequences of using a locally developed temporal distribution on the resulting hydrograph will be illustrated using the Hydrologic Engineering Center-Hydrologic Modelling System (HEC-HMS) rainfall-runoff model applied to a particular catchment.

The aim is to determine the most suitable way of accurately representing the temporal distribution of daily rainfall in Trinidad, the objectives being:

- Collection of long series of continuous rainfall data on a fine time resolution (5 minute to 1 hour time intervals) from rainfall stations in Trinidad;
- Analysis of the data to derive the temporal distribution of the daily rainfall for the rainfall station in Trinidad;
- Comparison of the peak flows generated by the derived temporal distribution with that for the SCS Type II distribution;
- Statements on how work should continue in establishing the most appropriate distribution for the country of Trinidad and Tobago.

The last objective is a recognition that the work needs to be continued using a larger sample of rainfall stations across the country. The results point to the importance of doing so, because it can have serious implications on the level of over-expenditure for providing drainage infrastructure.

2. MATERIAL AND METHODS

2.1 Derivation of the Temporal Distributions at the Local Stations

Rainfall records were obtained from two stations in the St. Joseph River Catchment (SJRC) (see Figure 1). This catchment has been widely studied by The UWI as its lower reaches have been subject to frequent flooding over the last twenty years. One station was in the lower reach, at The UWI Field Station (UFS), the other, called Upper Loango Station (ULS) was in the upper reach. The length of records ranged from 2019 to 2022 from the UFS and 2017 to 2022 from the ULS. The records were on an hourly basis from both stations. Data processing to separate the rainfall records into a series of independent 24-hour storms was done aided by coding in MATLAB and based on the suggestion by [1] for delineating records into separate events.

The method employed by [6] for delineation of the records into non-dimensionalized cumulative curves over a 24-hour period and eventually developing the single representative curve. was applied on the extracted storms from each station This was the same procedure used in developing the SCS temporal distribution curves.

2.1 Effect of Temporal Distribution on Peak Flow Estimation

A comparison was made between the runoff hydrographs generated from design storms derived from the various temporal distributions. The study area was the SJRC, which has an area of 50 km^2. The catchment is steep, is covered by secondary and primary forest in its upper reaches, and by moderate to dense urban areas in its lower floodplains. This catchment has been well studied and so a tight range of parameters values for the application of the SCS hydrologic procedure had already been established from calibration to observed flood events.

The public-domain hydrologic model, HEC-HMS was used to generate the runoff hydrographs, and evidently, the SCS hydrologic procedure was chosen among its several other available runoff hydrographs procedures. The daily rainfall depth at the 100-year return period was obtained from a statistical analysis of long-term records from a rainfall station within the catchment. Various combination of rain storms, shown below, were inputted into HEC-HMS for generating the runoff hydrographs used to compare the effects of temporal distributions, but only the first three being discussed in this paper:

- SCS Type II, evenly distributed over the entire catchment;
- UFS temporal distribution, evenly distributed over the entire catchment;
- ULS temporal distribution, evenly distributed over the entire catchment;
- UFS and ULS temporal distributions, applied to the lower and upper sections of the catchment, respectively.

In particular, the peak and volume of the resulting hydrographs were considered.

Figure 1: The location of the St. Joseph Catchment, Trinidad, West Indies

3. RESULTS AND DISCUSSION

The rainfall distribution patterns for the UWI and Upper Loango Stations were developed and are highlighted in Figures 2 and 3 respectively. They show a few of the actual storms and the derived representative distribution patters (RDPs) using the procedure in [6]. The RDPs display a marked similarity in their shape, but there are also some differences. They both display a bi-modal distribution meaning that there is an initial period of significant rainfall intensity that then ceases for some time unit a more intense, longer period of rainfall ensues. There is then a subsequent tapering of intensity to the end of the 24-hour period. The noticeable difference is in the initial rainfall pattern, better seen in Figure 4. It shows that rainfall although the rainfall intensity (determined by the slope of the curve) is about equal, this extends for a longer time at the UFS and hence rainfall at UFS has a greater fraction of the 24-hour rainfall occurs in this initial period. The resumption of rainfall occurs, more or less at the same time and at the end of it, about 90% of the rainfall has fallen. Since the second burst starts around the same time, the period of relatively no rainfall is longer at the ULS than at the UFS. The pattern for the last 10% of rainfall is identical between the two stations.

Figure 4 also shows the SCS Type II curve. The bi-modal distributions at the two rainfall stations contrast with that of the SCS Type II, which has one. The two peaks of the UFS and ULS, with the period of little rainfall have already been described. This is very different than for the SCS Type II. For that curve, there is a precipitous increase in cumulative rainfall with about 70% of the rainfall occurring over a five-hour period (between time period 0.2 and 0.4) Within that time step, there is a period of about 1 hour long during which 40% of the rainfall occurs. No where within the derived temporal distributions from the stations was such an intense period of rainfall observed. The second peak in the distribution of the two local stations is of an intense rainfall sustained over a period of about three hours. About 55% of the rainfall occurs during this peak.

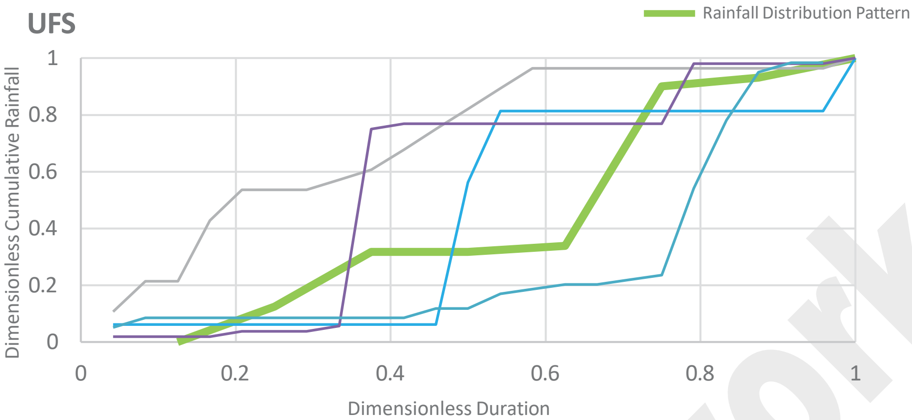

Figure 2: Dimensionless Rainfall Distribution Pattern, UWI station (University of the West Indies, St. Augustine Campus)

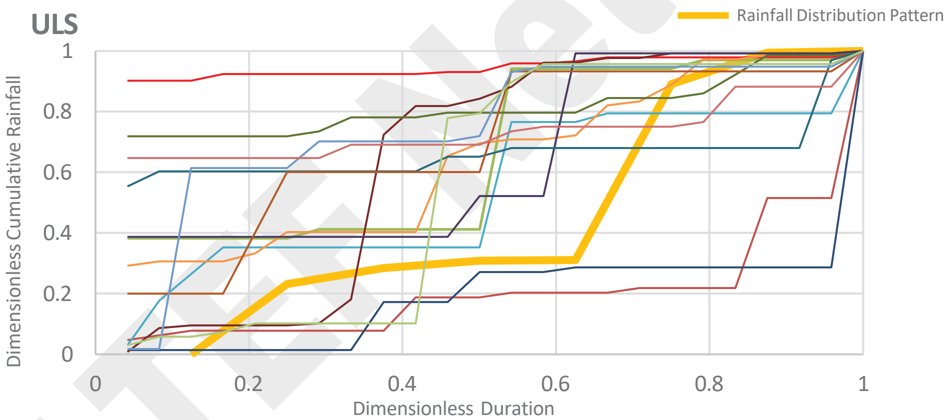

Figure 3: Dimensionless Rainfall Distribution Pattern Upper Loango Station

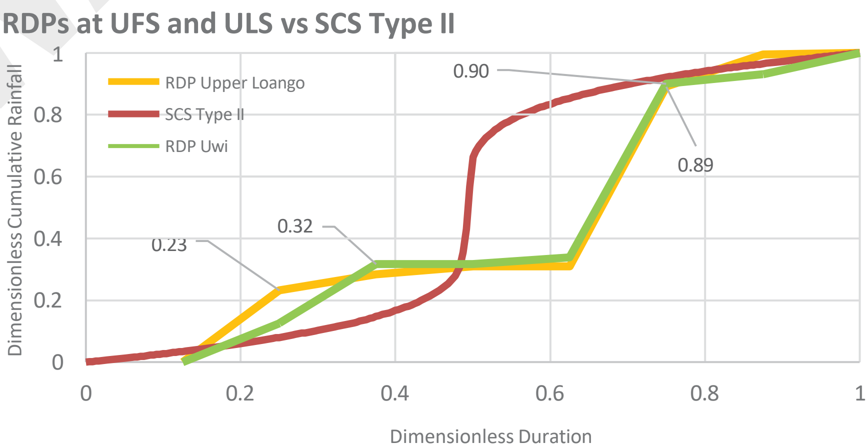

Figure 4: RDPs of Upper Loango and UWI vs SCS Type II

The results from this study show a similar trend to that of [1], [3], [4], [5] where the RDPs developed were different from the SCS Type II curve comprising of developed pattern curves with an earlier and later peak intensity when compared with the SCS Type II. Likewise, they noted that there was a slow rise to peak and a similar pattern was observed for rainfall from the UFS and ULS. These all contrast with the SCS Type II, which is sharper in nature.

Four scenarios of the RDP as well as the SCS Type II were used for the simulation run of HEC-HMS. As seen in Figures 5 and 6 the SCS Type II has a peak flow of 631.7 m^3/s for the 100-year return period. This peak flow occurs at an earlier time and the duration of the hydrograph is shorter when compared with the design RDPs. Furthermore, this value is up to 2.6 times larger than the design RDPs for the two rainfall stations shown in the two figures. [5] also reported a runoff hydrograph that produced a peak discharge 1.8 times larger than the design RDP of the middle cluster curve where the majority of the observed storm distributions were included. When only looking at the peak flow, the SCS Type II produced peaks significantly larger than the design RDP. [5] along with the other researchers, suggest that this may lead to an overestimation in the design of the hydraulic structures.

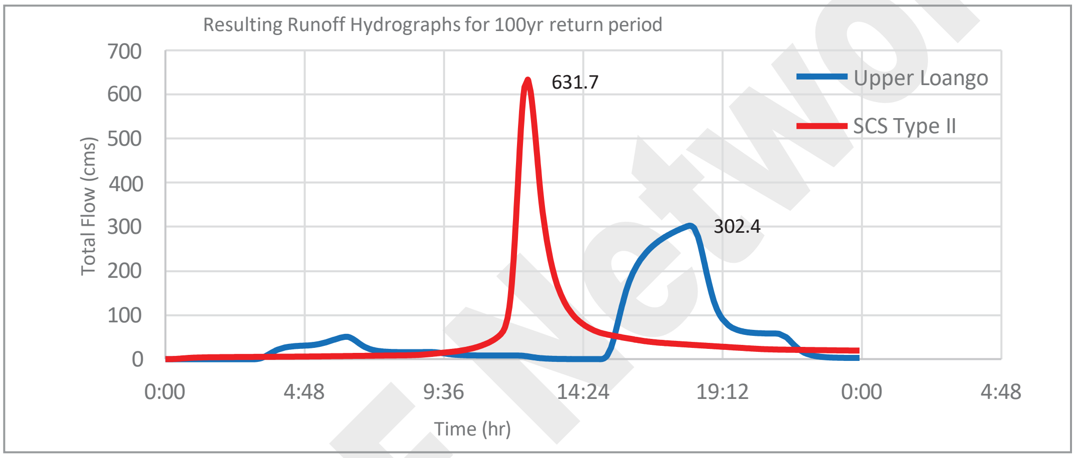

Figure 5: Resulting Hydrograph Upper Loango vs SCS Type II for the 100yr return period

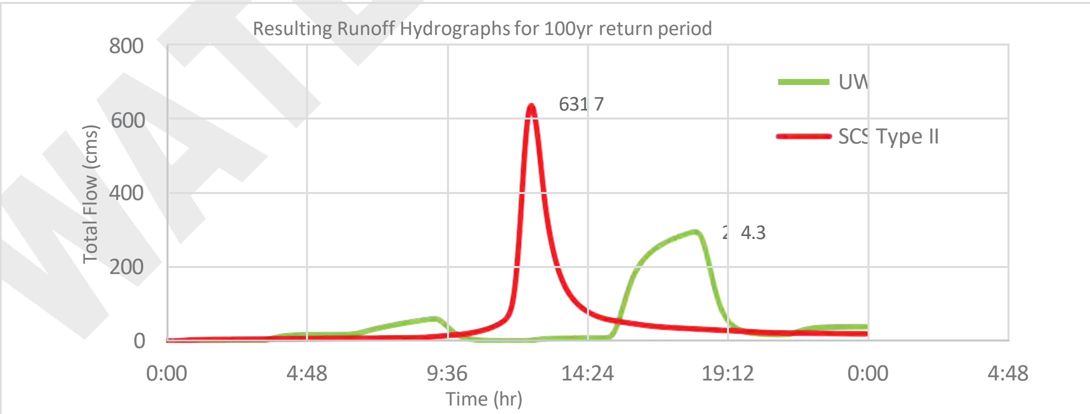

Figure 6:Resulting Hydrograph UWI vs SCS Type II for the 100yr return period

While peak flows are important for designing conveyance structures, such as river channels, the runoff volume also is required for designing storage facilities such as for detention basins. On comparison of the runoff volume estimated from the various temporal distributions, From the results shown in Table 1 of runoff volumes for the various runoff hydrographs, there is very little difference among them. The distribution of rainfall may still have some bearing on the design of the inlet and outlet works of these facilities, though it is not likely to have any major difference on the storage volume to be provided.

Table 1: Volume of runoff from the different scenarios for the 100 yr return period

Rainfall depths (mm) for the runoff hydrographs generated using the temporal distribution:		
SCS	UFS	ULS
87.35	88.26	86.24

Before concluding, it is perhaps worthwhile to re-examine Figures 2 and 3—but mainly the latter because more events are shown—to see the actual shapes of the events from which the RDPs were developed and compare them with the SCS Type II. As can be seen, there is a wide array of patterns recorded, but among them, there are several events displaying the uni-modal distribution found in the SCS Type II. This means to say that although such patterns may not be predominant, they do indeed occur. Among them, there is an event in which about 65% of the rainfall depth occurred over a one-hour period. This does not mean that because such does occur that the decision should continue to be the use of the SCS Type II curve. For one, there is no information on whether patterns are related to rainfall depth. Also, the records are relatively short and so a much more extensive database is required before making a decision.

4. CONCLUSION

Based on the study conducted on the temporal distribution of daily rainfall in Trinidad, the following were concluded:

- The designed RDP for both The UWI and Upper Loango stations are similar in shape.
- These designed RDPs are both different from the SCS Type II curve including the shape and number of peaks.
- For the runoff hydrographs, the SCS Type II had a significantly greater peak in comparison to the RDPs.
- The runoff volumes for both the RDP and SCS Type II hydrographs were the same but would have different effects on the hydraulic structure.

The findings suggest that the SCS Type II may not be suitable as it departs markedly from the representative temporal distributions of daily rainfall observed at these two stations in Trinidad.

There are some recommendations that can be useful to help advance this study and to finally conclude on the RDPs for hydraulic designs in the country. To begin, the dataset used spanned 3-5 years but typically a rainfall record of over 10 years is required. Therefore, a longer database should be adopted. This would allow for a better depiction of the patterns and more accurate results. Furthermore, how the temporal distribution varies spatially across Trinidad and even Tobago should also be investigated. This would allow other factors to be considered and hence get an even better understanding of the temporal distribution. Investigations should also be conducted on whether rainfall patterns are linked with the extremity of rainfall depth. Other durations of daily rainfall should also be explored. Lastly, investigations on whether there are any effects of climate change and the temporal distribution of daily rainfall should be conducted.

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COMPETING INTERESTS

I declare that each author has no competing interest.

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Examining the feasibility of GeoAI and IoT for Smart Flood Early Warning Systems for Local Communities in Caribbean Urban Spaces

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ABSTRACT

Over the last few decades, flooding has resulted in many problems that significantly impact countries in the Caribbean. This has been especially challenging in urban areas where widespread damage has occurred. In addition, given that over 50% of the world's population lives in urban areas, these locations are deemed to be vulnerable to climate-related disaster events that would further exacerbate the challenges in the region. These urban spaces in the Caribbean have limited access to real-time flood monitoring data for formulating and supporting policies for disaster practitioners to coordinate timely preparedness and mitigation efforts.

While flooding is complex, with a series of negative impacts on social and economic sectors, it is essential to provide a basis to support decision-making information on vulnerability and resilience through early warning systems (EWS). However, the main obstacle in creating early warnings in the Caribbean is the suitability and availability of data for real-time flood prediction (Aliasgar, 2010).

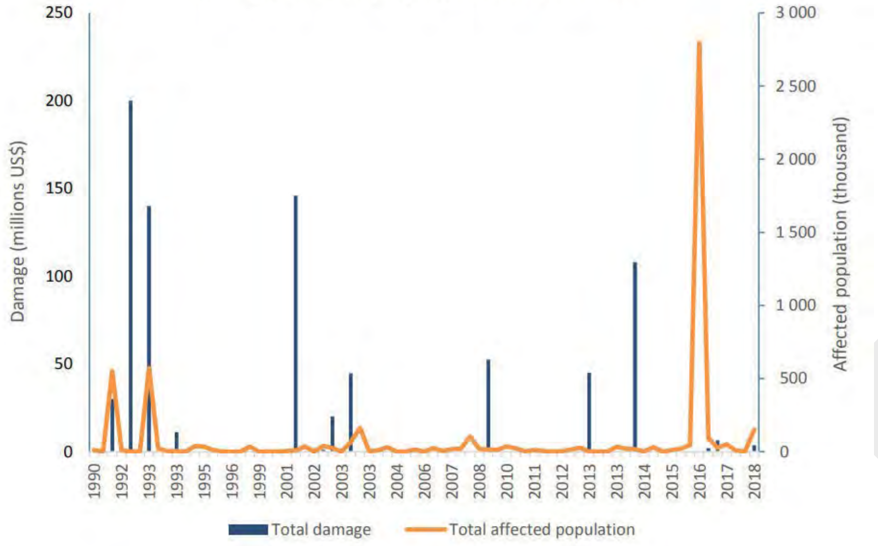
Consequently, there is a research gap from the perspective of short-term forecasting for sudden rainfall events in urban spaces in the Caribbean. Given the early warning system culture in the Caribbean has an environment of real-time data of scarce resources, it is necessary to forge an approach for real-time forecasting flooding impact in urban spaces. The paper will provide a preliminary analysis of the feasibility of machine learning and IoT use in supporting EWS in Caribbean urban spaces.

Keywords: GeoAI, Machine Learning, Internet of Things

1. INTRODUCTION

The Caribbean is susceptible to a range of natural hazards including floods, hurricanes, landslides, earthquakes, volcanoes, and tsunamis, making it the second most hazard-prone region in the world (UNDP, 2012). In the Caribbean region, flooding events cause a substantial impact on the socio-economic sector (ECLAC, 2019). Throughout the past decades, many SIDS including Antigua and Barbuda (2010); Barbados (2010); the British Virgin Islands (2010, 2017); Trinidad and Tobago (2016, 2017, and 2018); Dominica (2013, 2017); Saint Lucia (2013, 2016); and Saint Vincent and the Grenadines (2013) across the region have reported high occurrence of flooding events (ECLAC, 2019). See Figure 1.

Figure 1
Damage and affected people by disasters in the Caribbean



Source: EM-DAT: The Emergency Events Database - Universite catholique de Louvain (UCL) - CRED, D. Guha-Sapir - www.emdat.be, Brussels, Belgium, 2019.

The increase in the frequency of flooding in the Caribbean over the decades presents a major threat to the socio-economic sector of the region (Fontes & Phillips, 2019). Poor drainage infrastructure in urban spaces further intensify flooding events in many Caribbean countries; Urban areas often consist of planned and unplanned built environment, in combination with deprived preservation of drainage networks (Fontes & Phillips, 2019).

These conditions are recognized as primary triggers for the heightened impacts of disastrous flooding events in the Caribbean States (Fontes & Phillips, 2019). The occurrence of high-intensity rainfall contributes significantly to many flood events and increases with major tropical storms (Fontes & Phillips, 2019). Urban communities are deemed to be vulnerable to climate-related disaster events. Climate change presents even more significant challenges with sea-level rise (WMO,2018). The prevalence of climate change poses a problem for urban flooding given the unplanned built environment which exists in the Caribbean (Fontes & Phillips, 2019). The importance of evaluating and measuring the level impact of hydro-metrological events on local communities in urban areas for the Caribbean States has created a significant demand for increasing the preparedness and mitigation initiatives.

While early notification of flooding generates the benefits of preventive measures, present-day flood events tend to increase and become severe with climate change, rapid urbanization, and aging infrastructure (Fontes & Phillips, 2019). Collymore (2016), in a study on the Early Warning System (EWS) in the Caribbean, argued that while many noteworthy changes have been made in the developments of EWS, specifically for tropical cyclonic events, there is a need for improvement and focus essentially to pursue EWS at the community level.

Given the complexity of inundation in the Caribbean states, to support the decision-making within this space, it is crucial to understand the concept of urban flooding and how it can be detected, measured, and monitored in urban communities. The issue that arises is the problem of data scarcity of timely and relevant weather-related sensor datasets in urban spaces. Consequently, there is a research gap from the perspective of short-term forecasting for sudden rainfall events in urban spaces in the Caribbean. Considering this, (Aliasgar, 2010) found that the key hindrance in creating early warnings in developing countries is the accessibility and suitability of data for flood prediction.

Chang & Chang (2020) argued that flood inundation forecasting in urban spaces is critical for effective flood management. However, it remains a substantial challenge in urban areas, given the multi-layered connections related to exceedingly ambiguous parameters for hydro-meteorological and limited access to high-resolution hydro-geomorphological data.

Chang & Chang (2020) recommended an online flood forecasting models through the distribution of flood data to advance early warning systems and disaster prevention. This supports the need for monitoring flood depth data in urban areas for flood prediction with the integration of geospatial technologies for real-time observed inundation depths (Chang & Chang, 2020). The need to adopt a comprehensive EWS in a data scarce environment will give strength to develop an approach for forecasting flooding impact in urban spaces (Collymore, 2016). Concerning the impacts of climate-induced events, flooding results in social and economic losses, especially for populations that reside in flood-prone regions (Fontes & Phillips, 2019). These pose challenges for local urban communities, therefore it is essential to detect, monitor, and assess the using the best available solutions. We argue that machine learning, IoT, and Geospatial technologies are most appropriate for supporting these objectives.

The study intends to understand, and review IoT, Geospatial technologies, and Machine Learning (ML) techniques that can be leveraged to address the need for improving EWS in the Caribbean region as expressed by Collymore (2016), through the integration of IoT sensor data to evaluate the feasibility of a GeoAI machine learning-based system that will provide timely prediction information in urban areas to support and decision making and mitigation planning.

2. METHODOLOGY

This research was conducted through a comprehensive literature review of different journal articles and reports over the decade. The terms used in searching for related articles are “Machine Learning flood forecasting” “IoT real-time flood prediction” and “real-time urban flood prediction. As a result, a total of 72 articles were reviewed with 38 of the 72 considered relevant articles.

The review examines the following areas: background on the data existence of flood sensed datasets, IoT Flood Early Warning Systems, IoT Flood Prediction and Data Processing, IoT and GIS Integration. Based on the findings of the review, we propose a strategy for supporting efforts to mitigate the impacts of flooding on urban communities in the Caribbean.

2.1 Background: Real-Time Flood Forecasting

In a series of research undertaken over the past years, it may be noted that real-time flood forecasting is becoming more predominant in the future for urban areas (Rene et al., 2018). There is a growing need for better data-driven analysis to address the problem of flooding. In an example of research completed in the Caribbean, Rene et al. (2018) outlined an approach to monitor and predict flood levels in an urban context for Castries, St. Lucia for a real-time pluvial flood forecasting system developed using high-resolution hydrodynamic modeling and hydrological forcing. The approach utilized Global Forecast System (GFS) rainfall data in an area where data is low resolution for urban areas. This was useable enough to support flood forecasting from rainfall data (Rene et al., 2018).

Given the complexity of an urban environment to model different features, a wide range of highly accurate resolution datasets are required in a manner to model flooding (Zahura, 2020). Rene et al. (2018) proposed an improvement to the flood forecasting system using radar and other feature datasets (drainage network) to strengthen the predictions. Rene et al. (2018) expressed confidence in the flood forecast system. However, given the significant amount of skill and post-processed required for the useability this may be challenging for easy adoption by urban community managers.

In addition, while Rene et al. (2018) proposed a real-time flood forecasting system that provides fast lead time, this solution requires post-processing initially, thereafter rainfall forecast is utilized for flood prediction. However, this could be significantly enhanced with ML (Mosavi et al., 2018). As seen over the last two decades, ML has advanced better performance to support real-time flood management in urban communities (Zahura, 2020).

2.2 IoT Flood Early Warning Systems

The use of IoT is becoming a common practice around the world. In research conducted by Gartner, 6.4 billion connected things existed in 2016, this figure is predicted to be 20.8 billion by 2020 (Khalaf et al., 2020). IoTs comprise a wide range of sensors that are beneficial for enhancing the collection of data to make better decisions (Khalaf et al., 2020). IoT allows for direct control devices to precisely measure and sense remotely data in different environment settings (Khalaf et al., 2020).

Most environmental monitoring structures have implemented IoTs to support early flood warning systems. Flooding is considered the highest problem of natural disasters globally (WMO, 2018). In many cases, heavy rainfall occurs in a short duration, however, this results in landslides, property damage, and the possible loss of lives (WMO, 2018). Often intensive rainfall over a short duration occurs without adequate warning and poses a significant threat to the physical environment (Šakrak et al., 2020). Many studies in IoT for flood monitoring have been done, applying different approaches for flood early warning systems. Basha et al. (2007) argued that developing countries often times are severely impacted and less prepared to deal with these disasters than developed nations. This research discusses the management of the early warning system for river flooding.

Khaire et al. (2019) researched the strengths and weaknesses of five (5) flood management systems in the following countries/regions; Honduras, Xicheng District, Beijing, Rambla del Albujon Watershed, Korchar Haor, Bangladesh and Iowa, USA. The key strength identified includes real-time data logging, use of multiple types of sensors, alerts and notifications, data security, internet communication, and data storage. Mousa et al. (2019) presented on flood detection using ultrasonic and infrared sensors for flash floods. Research done by Udo & Isong (2013) shows how flood monitoring and detection can be conducted using wireless sensor networks. In the research, the parameters for flood detection were water level, rainfall, humidity and temperature. The water level will regulate the extent of the inundation.

Ancona et al. (2015) conducted research on flood detection by using IoT for rain and river gauging. Ancona et al. (2015) recommended a gauge shielding architecture and software requirements for IoT implementation. Leon et al. (2018) presented an EWS for river overflows. The sensor network comprises a river-level sensor that measures the water depth using a precision ultrasonic sensor. Salunke & Korade (2017) outlined the use of the IoTs in urban flood management system. The system was divided into three components; calculation of drainage flow, rainfall, and water level. The research investigated the use of various sensor types to measure water level, velocity of rainfall and volume flow.

Šakrak et al. (2020) found that most flood detection systems have been aimed at predicting water levels in the river basin. Yuliandoko et al. (2018) illustrated that the flood forecasting systems consist of several hydrometeorological and geospatial datasets collected from monitoring stations. Importantly, data from IoT flood forecasting sensors supports the reduction in the effects of floods (Yuliandoko et al., 2018). Critically the output from flood prediction will be the input for an early warning system (Yuliandoko et al., 2018).

2.3 IoT Flood Prediction and Data Processing

Most commonly, IoT provides a platform for real-time data collection and, provides data analysis and visualization (Abdullahi et al., 2019). Data collected is continuously transmitted via the communication infrastructure (Abdullahi et al., 2019). Hirabayashi et al. (2013) recommended the use of IoT and machine learning to forecast the likelihood of floods in a river basin. Abdullahi et al. (2019) suggested that it is imperative to have a platform that stipulates how hardware and software components fit within a flood data management system. Zahir et al. (2019) designed a system that displays the data of the water level measured on a webserver. The system allows users to monitor the water level through a computer or mobile phone via an internet connection. Khaire et al. (2019) illustrate how a user can get real-time information by monitoring flooded roads over SMS-based service. The research used flood limiting water level (FLWL) approach to protect the public from flood impacts. Ancona et al. (2015) stated that flashes flood alarm systems require a compact network of rain gauges for monitoring local rainfall events.

Bande et al. (2017) evaluated the physical parameters for flood prediction to mathematical modeling-based flood prediction schemes, and around machine learning algorithmic approaches. Artificial Neural Networks ANN is known to be one of the algorithms that can be useful in flood prediction (Abdullahi et. al, 2019). Machine learning presents the opportunity to effectively model the non-linear features being used for hydrological problems (Abdullahi et. al, 2019). Abdullahi et al. (2019) recommended using machine learning model and IoT to predict the average flood inundation depth for Erren River basin in south Taiwan.

Chang & Chang (2020) stated that for physical-based models short-term forecasting have limitation because of the non-linear features of a rain event. This in turn leads to a lack in accuracy and robustness of statistical methods (Chang & Chang, 2020). Abdullahi et al. (2019) proposed a real-time monitoring suing Microsoft's Azure Machine Learning (AzureML) with a built-in 2-class neural network which was used to forecast flood depth. Khalaf et. al, (2020) demonstrated recent growth in the IoT and machine learning for the automated flood analysis. The objective was to derive a machine-learning algorithm to analyze flood sensor datasets, with non-linearities characteristics. In this research, Khalaf et al. (2020) proposed a global flood monitoring sensor system to measure rivers' water levels.

Khalaf et al. (2020) presented the following algorithms used in flood prediction; Artificial Neural Networks (ANN), Random Forest (RF), K-Nearest Neighbour classifier (KNN), Long-Short Term Memory (LSTM), and Support Vector Machine (SVM). Panchal et al. (2019) used the machine learning algorithms, support vector machine and random forest to measure flood level.

Sun et al. (2019) applied machine learning deep learning algorithms to vastly improve early flood warnings. Based on the machine learning algorithms, the systems can create the necessary early warnings, notifications, and alarms to locations where the disaster is going to occur.

2.4 IoT and GIS Integration

The use of geographical concepts in (IoT) is developing rapidly, (van der Zee & Scholten, 2013). The United Nations Global Geospatial Information Management Committee of Experts (UN-GGIM) report on future trends in geospatial information management (United Nations, 2020) highlights the role of geospatial technology with IoT. IoT contributes to the delivery of geospatial data to facilitate real-time processing.

Many developed countries like Germany, China, and the USA have taken an integral step to fuse advanced ML technologies alongside IoT and big data analytics. Yusoff et al. (2021) suggest that geography is key feature ingredient for IoT. IoT datasets have the following features; position, dimension, and orientation in space and time. These features have interconnected spatial relationships (van der Zee & Scholten, 2013). Satria et al. (2020) investigated this further with a prototype of a flood monitoring system based on Google Maps was developed using ultrasonic sensors.

Kang & Lee (2018) proposed sensor devices to detect river flooding in areas with a high risk of flooding. In this study, the system developed collect disaster data and overlay with an insurance map. The system provides updated information to support the decision-making related to flood monitoring.

Given the dynamics of a changing urban landscape, the application of GeoAI provides the basis for advanced geo-enabled decision-making. GIS provides the spatial variation of different features and properties in particular topographical datasets. In addition to the emergence of Big Data, cloud computing, and IoT, GIS allows for connectivity in urban spaces alongside processing and ease of visualization for decision making Yusoff et al. (2021). Similarly, Tehrany et al. (2019) developed a GIS-based SVM support vector machine (SVM) model which demonstrates its use for assessing flood susceptibility areas in Brisbane, Queensland, Australia.

The integration of AI and geospatial data delivers real-time information flood prediction and 3D GIS-based structure. Yusoff et al. (2021) emphasize that the features of sensing, identification, processing, communication, and networking capabilities are key catalysts to delivering quality products associated with real-time flood prediction.

3. RESULTS AND DISCUSSION

Overall, we found that the current literature examines river-based monitoring approaches to flood detection, especially in Latin America and the Caribbean. There has been very limited study conducted in the region for using machine learning for urban flooding monitoring and detection. Because of the limited study, there is a need for reliable data that can accurately measure and visualize predicted flood levels, and to determine how GeoAI can be leveraged to support flood levels in urban spaces. Flooding is the most significant problem of natural disasters globally, which is often exacerbated by heavy torrential rainfall over a short duration, causes property damages and loss of human lives (Šakrak et al., 2020).

Understanding the current trends in ML techniques for flood forecasting lays the foundation for effective algorithms to carry out the best-suited prediction. The results of the literature review provide the guidance to identify appropriate components in terms of the applicability of ML techniques and the consideration of input parameters needed in supporting flood prediction. This in turn can be leveraged to develop mitigation measures that are context specific for Caribbean urban communities. Mosavi et al. (2018) extensively cover the latest machine learning techniques used for flood prediction on two bases of short- and long-term prediction, whereby the drawbacks of limited data and non-linearity pose an adequate mechanism to support modeling with minimal inputs to quickly develop flood predictions. Mosavi et al. (2018) describe the flood prediction model as data-intensive that requires specific techniques to deliver a short lead time for a prediction. While physical models provide a mechanism for prediction, there is a major gap in short-term prediction. Additionally, it requires in-depth knowledge and expertise to compute, which is similarly mentioned in the paper from Rene et al. (2018).

Mosavi et al. (2018) supports the advantages of ML, presenting a faster development of the system with minimal inputs for real-time flood forecasting. This reduces easier implementation with low computation cost, which delivers high-performance models to the convention methods (Mosavi et al., 2018). Rene et al. 2018, indicated that there is a shortfall in physically based statistical models. The advancement of data-driven models not having previous knowledge, through machine learning can support real-time flood forecasting. Consequently, this presents an opportunity for data-driven systems flood monitoring inundation levels in Caribbean urban communities. According to Maspo et al. (2019), ANN is one of the most popular used machine learning algorithms for real-time flood prediction, in particular hybrid models. Deep Learning Neural networks produce a high accuracy rate in flood prediction.

Table 1: List of common techniques and key input parameters for consideration in ML urban flood prediction models

Authors and Title	ML Algorithm	Parameters	Region
Chen, G., Zhou, Z., Zheng, R., & Qi, T. (2018). **A Smart Urban Flood Control And Warning System Based On Big Data.**	Neural Network	Rainfall, hydrodynamics	China

Eini, M., Kaboli, H. S., Rashidian, M., & Hedayat, H. (2020). **Hazard and vulnerability in urban flood risk mapping: Machine learning techniques and considering the role of urban districts.**	Maximum entropy approach	Population, urban texture and distance to channels	Iran
Rahmati, O., Darabi, H., Panahi, M., Kalantari, Z., Naghibi, S. A., Ferreira, C. S. S., ... & Haghighi, A. T. (2020). **Development of novel hybridized models for urban flood susceptibility mapping.**	Hybrid Model Wavelet	Elevation, distance from channel, precipitation, slope percentage	Iran
Motta, M., de Castro Neto, M., & Sarmento, P. (2021). **A mixed approach for urban flood prediction using Machine Learning and GIS**.	Random forest	Temperature precipitation	Libson, Portgual
Hou, J., Zhou, N., Chen, G., Huang, M., & Bai, G. (2021). **Rapid forecasting of urban flood inundation using multiple machine learning models.**	Random forest (RF) and K-nearest neighbor (KNN)	Rainfall	China
Löwe, R., Böhm, J., Jensen, D. G., Leandro, J., & Rasmussen, S. H. (2021). U-FLOOD– **Topographic deep learning for predicting urban pluvial flood water depth.**	Neural network	Topography, rainfall	Denmark
Ke, Q., Tian, X., Bricker, J., Tian, Z., Guan, G., Cai, H., ... & Liu, J. (2020). **Urban pluvial flooding prediction by machine learning approaches–a case study of Shenzhen city, China**.	Binary Classification Decision Tree, Support vector machine	Rainfall	China
Darabi, H., Choubin, B., Rahmati, O., Haghighi, A. T., Pradhan, B., & Kløve, B. (2019). **Urban flood risk mapping using the GARP and QUEST models: A comparative study of machine learning techniques.**	Genetic Algorithm Rule-Set Production (GARP) and Quick Unbiased Efficient Statistical Tree (QUEST).	Rainfall, precipitation, slope, curve number, distance to river, distance to channel, depth to groundwater, land use, and elevation.	China
Bouramtane, T., Kacimi, I., Bouramtane, K., Aziz, M., Abraham, S., Omari, K., ... & Barbiero, L. (2021). **Multivariate Analysis and Machine Learning Approach for Mapping the Variability and Vulnerability of Urban Flooding: The Case of Tangier City**	Classifications and Regression Tree and Support Vector Machine	Topographic properties and urban characteristics (population density and building density) the drainage density and distance to channels)	Morocco
Zahura, F. T., Goodall, J. L., Sadler, J. M., Shen, Y., Morsy, M. M., & Behl, M. (2020). **Training machine learning surrogate models from a high-fidelity physics-based model: Application for real-time street-scale flood prediction in an urban coastal community.**	Random Forest (RF)	Rainfall, Water drainage	USA

The second component, input parameters for flood prediction highlights the characteristics of the urban spaces. The Caribbean urban areas consist of such characteristics whereby urban drainage, soils, temperature, and streamflow datasets form a great part of the decision-making output. The ML models outlined in Table 1, include such parameters that would be beneficial determine flood forecasting. Maspo et al. (2019) presents additional parameters to incorporate datasets which include radar remote sensing, drone, and in-situ-observation. Considering these parameters provide the nature on which reflects of the complexity of the urban context in the Caribbean.

The growth in urbanisation across the Caribbean is on an upward trend in recent years. Bahamas, Trinidad and Tobago and Suriname are the most urbanised countries in the Caribbean region (Mycoo and Donavan, 2017). These urban spaces consist of the critical infrastructure located within these zones impacted by sea-level rise. For example, 54 percent of Jamaica's population lives in urban areas, similarly in Barbados 66 percent of the urban population resides in coastal urban centres. Such dense population centres are also prone to urban flooding (Mycoo and Donavan, 2017).

The causes of flooding in urban spaces are the experiences of high above normal rainfall, often time-triggered through the form of encroachment in the flood plains of river/water bodies and proximity to the coastal zone. These urban systems are often constructed without adequate drainage systems being built (Udika, 2010). Sometimes the construction materials are not eco-friendly, and the surface is not permeable. The real issue of urban flooding being faced in Caribbean states emanates from the high forms of informal urbanization, gaps in limited data on flood occurrences and improper waste management, in areas vulnerable to flooding (Udika, ,2010).

IoT networks can provide real-time geospatial data with location, accompanied by other attribute information. For instance, in an urban environment with frequent land-use changes, building density, and drainage management, it is important to understand flood prediction using such input parameters. Data collection in a changing environment and with time-sensitive variables make it even more important for the integration of geospatial technologies and AI for predicting flooding events in real-time (Yusoff et al., 2021).

Importantly what we need to achieve is the mechanism to capture, process, and analyze flood-related information in real-time. As a result, the geospatial technologies and IoT are key inputs in managing this process (Yusoff et al., 2021). The introduction of GeoAI and IoT - data science, deep learning approaches will allow for much faster delivery for decision making. The introduction of GeoAI in Caribbean cities must encompass several layers of a system of systems where various components are connected to the social, economic, environmental, and physical infrastructure.

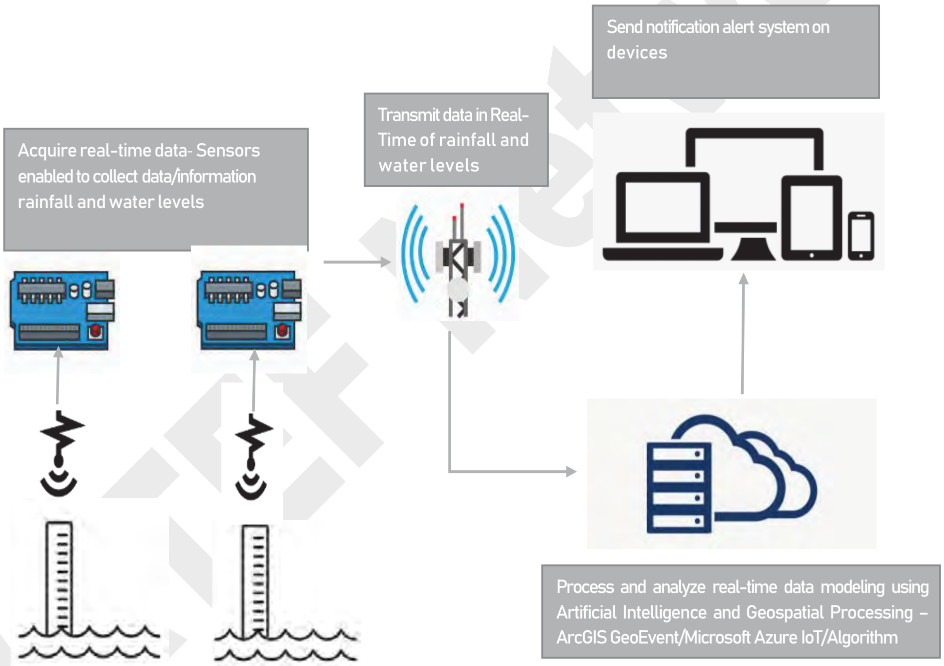


Figure 2: Conceptual Design for smart flood early warning detection system

4. CONCLUSION

This study presents a wide range of academic research published on flood early warning systems. There is a need to build upon the foundations of the existing ML techniques presented by in the literature, so that these strategies may be adopted for Caribbean states, given the present lack of the use of IoT technology to facilitate the faster exchange of flood data for urban spaces in the region.

The existing gap in the landscape for urban spaces for Caribbean states can be closed to foster better modelling, evaluating and visualizing the natural and built environment, using IoT as an intelligent sensing infrastructure with the combination of GIS and IoT technologies, together with geospatial analysis (Kamilaris, 2018).

Arguably, GeoAI offers significant potential for a significant contribution to the knowledge that this study will make, is the infusion of geospatial technologies with ML and IoT. There is an opportunity for Caribbean states to overcome the technical challenges that exist with flood early warning systems presently being utilized. As demonstrated in Figure 2, the conceptual design model shows that the integration of Machine Learning, IoT with spatial technologies provides the mechanism for an effective early warning system, which in turn will enable a better understanding of the interaction of urban communities with natural hazards (Mousa et al., 2016).

COMPETING INTERESTS

We wish to declare that we do not have any conflicts of interest with respect to this work. This work was done solely by the authors.

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Advancing Solar Energy driven Heterogeneous Photo-Fenton processes for River Water Remediation

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ABSTRACT

Water is essential for life. Many of the countries across the globe that have poor management of potable water resources and lack critical infrastructure for managing wastewater. As the population of the Earth grows exponentially, the demand for water increases. More than 1 billion people across the world do not have access to potable water and they are struggling with epidemic level disease outbreaks, limited water supply among other large-scale public health risks. Large-scale critical infrastructure chains are required in order to produce potable water from raw water sources, and developing countries continue to struggle economically and cannot afford the same treatment chains as the developed world. This research project evaluated the feasibility of two photocatalytic and photo-Fenton solar reactors on their capabilities to breakdown water contaminants present in natural hydrosystems and freshwater resources namely rivers. The approached adopted to achieve this was the photo-Fenton reaction and solar-photochemical reactors designed constructed and tested for the removal efficiencies of Chemical Oxygen Demand (COD), ammonia, nitrates, nitrites and phosphates via LCK curvette tests, in addition to turbidity and colour. The water quality analysis and results showed that oversaturation of the photo-Fenton reagents reduces the effectiveness of the reaction, and that finding the correct chemical balance has a greater impact on the removal efficiencies of the five pollutants than the use of UV light catalyst. The difference in the reactor builds was their diameter, and the results showed that the reactor of smaller diameter achieved the best removal efficiency across all five pollutants.

Keywords: Photo-Fenton, Heterogeneous Fenton, Photochemical treatment, Solar Energy, River Water, Detoxification, *Drinking water,* Solar *radiation*

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1. INTRODUCTION

Across the United Kingdom (UK), wastewater and stormwater network systems are often linked to the same sewers and hence go through the same treatment process [1]. The wastewater treatment works chain implemented in the UK are typically a four-stage process: preliminary, primary, secondary, and tertiary [2]. Each stage involves various infrastructure in place to treat the wastewater. Preliminary screening involves removing the insoluble solids in the wastewater through filtering machines. Primary treatment use sediment basins that settle the wastewater [2,3]. This has compound utility in that sludge (liquid waste) settles at the bottom of the basin where it is pumped out to a treatment facility while fats and oils rise to the surface due to the differing densities where a skimmer removes and segregates the oils for use as glycerides in processes like saponification. The secondary process specifically focuses on decreasing the biological content of the wastewater, generally by aeration and the addition of a controlled "good" bacteria that consumes the pathogens in water, converting it into carbon dioxide, water, and energy for the "good" bacteria's own reproduction [4,5] . The tertiary stage involves processes that are energy intensive, and often involve the addition of oxidising chemicals to disinfect the water of microorganisms. According to the World Bank Group [6] 99% of all impurities in water are removed by tertiary stage processes.

The St Mary's Island development in Chatham, Kent, UK began development in the late 1990s, built on an island in the River Medway and entered into additional phases of renovation into a small village that are still ongoing [7]. Currently there are more than 2000 domestic homes and dwellings on the edge of River Medway. The initial research ideas of the project were developing a concept of a hypothetical water treatment facility for the St Mary's Island development, drawing from the River Medway to produce potable water. A full-scale water treatment plant would not be feasible to construct, although the river water quality would not require extensive treatment processes and infrastructure. Upon further independent research it was discovered that the team of hydrologists had already designed the intended storm water drainage systems for the new development but had not considered the prospect of drawing water from the River Medway, treating it to drinking standard in an underground laboratory and distributing the potable water around the island. This would save energy, expense, and encourage recycling of water, albeit on a small scale. River water and wastewater are not of the same quality. This is primarily down to the pre-treatment of river water from wastewater treatment plants (WWTPs) to reduce the pollutants in the water to specific standards of quality before being allowed to run into the channel. Even with the pre-treatment, river water is not fit for human consumption, though. Producing potable water from a municipal source can be very expensive and may not be the ideal option for a developing country to implement. Considering that the pollutant levels of river water are already lower than that of a municipal source, it would therefore cost less to treat river water to drinking standards than it would to treat municipal water. Based on the fact that approximately 90-99% of all impurities in water are removed by tertiary stage processes [6,8] an efficient tertiary treatment process alone may be enough to bring the physical, chemical and biochemical parameters up to drinkable standards. Interest was taken for the Solar driven photo-Fenton reaction and treatment process. The aim of this project was to build and test two photo-Fenton reactors, compare their pollutant removal efficiencies, and deduce whether the St Mary's Island housing development would benefit from implementing a similar reactor to produce potable water locally. In order to achieve these aims, the following objectives of the study were set about:

- Analyse constructed reactors and use the knowledge to design two original models that explore the strengths of disinfection through the Photo-Fenton Reaction.
- Construct the final designs with the resources available.
- Test the effluent water from the reactors with appropriate apparatus to study what chemical changes the water has experienced whilst in the reactors.
- Apply variables to the testing of each experiment.
- Critically analyse the data of these tests and judge whether the effluent water is of satisfactory quality to be considered potable.
- Decide whether the St Mary's Island development will benefit from the use of such a water treatment process.

2. EXPERIMENTAL DETAILS / METHODOLOGY

The three mainly documented reactor designs are non-concentrating, medium-concentrating and high-concentrating reactors. Non-concentrating reactors are typically designed without an emphasis on redirecting the light that enters the solar collector, whilst the medium and high categories are generally dictated by the temperature range that the reactor reaches under the concentration of sunlight (150-400˚C for medium and over 400˚C for high) [9].

2.1. The Design and Fabrication for the Solar Photoreactors

Strips of aluminium film were applied to a plywood base. Thereafter, Borosilicate glass tubing with 32mm and 44mm diameter alongside flexible tubing were used to connect the solar-photoreactor together, alongside aluminium reflective surfaces. Silicone was used to seal various ends and connectors of the glass tubing to ensure no leaks occurred. Plastic clips and horseshoe clamps were placed at the end sections of the glass tubes and slides along the edges of the photoreactor to stabilise the U-bends. Two Pipestock Ltd. (Romsey, England, UK) digital flow meters with a range of 1-40 litres per minute were placed at the entry and exit points of the solar reactors. Using the design from Figures 1 (a) and (b) a materials list was created in order to begin procurement of the materials. The materials list for both reactors are in Table 1 below:

Table 1: Materials List for Solar Photochemical Reactors

Function	Materials	Specifications
Effluent water container	Steel container	Inert, volume capacity between 40-60L
Raw water container	Steel container	Inert, volume capacity between 40-60L
Water distribution through pipe network	Peristaltic pump	1-40Litres/hour flow rate
Reflective surface	Aluminium sheet	Low malleability, highly reflective of UV-A radiation, 1.7m width 0.5m height
Piping – main reactor frame	Straight borosilicate tubing	44mmm diameter, 1.6mm wall thickness, 4.05m required
Piping – adjoining pieces to link reactor chamber lengths	U-shaped bends	Inert material, 44mm diameter, compact, transparent- Solar Reactor 1 Inert material, 32mm diameter, compact, transparent- Solar Reactor 2
Piping – flexible tubing connecting all apparatus together	Flexible inert tubing, exact material not yet found	Must be able to fit onto varying diameters, opaque is fine, ability to withstand water pressure, corrosion resistant
Insulation from light	Fibreglass	Easily attached to materials
Photo-Fenton reagents	Chemicals	N/A
Apparatus stand	Steel trolley	Attached castor wheels to allow for mobility
UV-A wavelength	Low pressure mercury UV lamp (315-400nm)	Quantifiable UV-A dosage

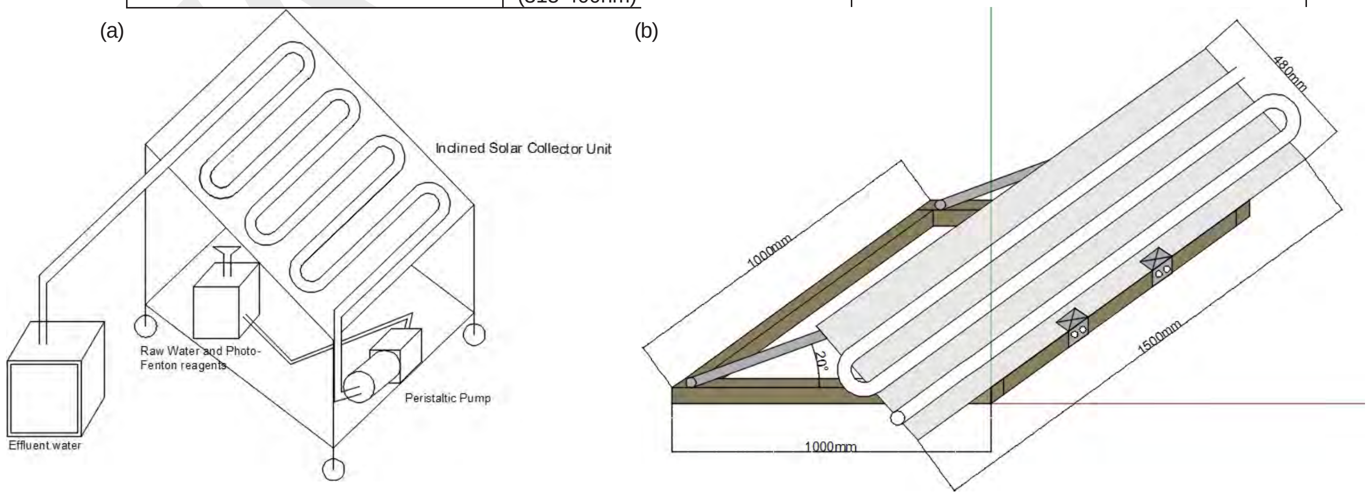

Figure1 (a) Isometric Serpentine Array of Solar Photoreactor (b) Schematic of the Solar Photochemical Reactor Designs Adapted from (Tota-Maharaj [8] and [9]).

(a)

(b)

Figure 2: (a) 44mm Diameter reactor, referenced as Solar Reactor 1 (b) 32mm Diameter reactor, referenced as Solar Reactor 2

2.2 The Photo-Fenton Reaction

The photo-Fenton reaction involves the use of highly oxidising chemicals that are sensitive to light, and balance of the reaction requires precision in the volume of reagents allowed to mix. Since the sensitivity to light only takes effect once the photo-Fenton reactants are mixed and added to the raw water supply, the pre-test phase (which involves a few processes in the water and environmental engineering laboratory to make the water testable) does not have the complication of being light sensitive. Essentially, the raw river water was split into two separate containers, one had the required Fe (II) concentration and the other had the required H_2O_2 concentration. This separation was to prevent the reaction occurring before entering the reactor. There is the issue of chemical balance for the photo-Fenton reaction. If the concentration of photo-Fenton reagents used was too high, it would lead to rapid oxidisation of the Fe (II) to form iron (II) hydroxide or iron (III) hydroxide which are both turbid solutions. This would spoil not only the data received, but it could also potentially stain and ruin the apparatus. The concentrations of Fenton reagents needed is based on the quality and volume of the raw water being treated. As such, careful research and appropriate testing of the raw water once collected is the way to ensure the chemicals are correctly mixed. In terms of achieving the accuracy of reagents being added to solution, it is possible to use laboratory processes such as titrations to achieve molar accuracy.

2.3 River Water Collection and Sample preparation

River water was collected twice weekly from the River Medway Kent, in order to begin the pre-testing phase of the water quality. Once the river water was collected, it had to be treated chemically prior to being used in the reactors. The reason for this was to add the photo-Fenton reagents and to bring the river water to an appropriate pH to facilitate the photo-Fenton reaction. Based on previous study, the photo-Fenton reaction has the highest inactivation rate of bacteria with a pH between 3 and 3.5 [10]. Previous studies also reveal that the correct proportions of Fe^{2+} and H_2O_2 must be found depending on how polluted the water sample being tested is. With insufficient H_2O_2, Fe^{3+} may precipitate and stain the apparatus, as well as pollute the samples. Similarly, too high quantities of H_2O_2 and Fe^{2+} will cause a heavy oxidation reaction and cause the water sample to become far too turbid and polluted to be tested. The pre-test phase in the links lab involved experimentation with the collected river water to find three ideal concentrations of Fe^{2+} and H_2O_2 as well as to acidify the water to a pH of between 3 and 3.5 [11]. Previous study conducted by Rabelo [12] showed that the ideal ratio of reactants for a Kraft Pulp Mill was 500mg/l of H_2O_2 to 50mg/l of Fe^{2+}. Although in a general case, a ratio of 100:1 yields the greatest efficiency of Chemical Oxygen Demand (COD) removal. This is because the concentration of H_2O_2 in a Fenton reaction gradually decreases over time, contrasted with a very sharp decrease within the first 30 minutes of concentrations less than 100:1. With a proportion of 100:1, the H_2O_2 is homogenous enough within the solution to last the entire reaction length, decreasing in a more linear fashion, rather than a steep decrease and plateau. As such, samples were prepared with 100:1, 10:1 and 2:1 ratios so that concentration can become another variable to test, with the expected trend of efficiency to go from high proportions to low.

2.3.1 Experimental Methods for Testing proportions of photo-Fenton Reagents

After the volume of river water was selected, the pH was measured and adjusted to 3-3.5 using the addition of sulphuric acid. The molecular weight of $FeSO_4.7H_2O$ was calculated and the desired proportion measured with the selected volumetric loading of river water. The calculated molecular weight of H_2O_2 was needed to establish the desired proportion by using chosen volume of water. A zero weight of the weighing dish was set prior to any photochemical additions; the acidified river water was placed in a beaker and onto a metallic stirrer and a measured mass of Fe^{2+}, followed by H_2O_2. This procedure was carried out multiple times in order to observe the results of the reactions over time. The first experiment involved measuring out quantities for a proportion of 2000mg/l and 200mg/l of H_2O_2 and Fe^{2+} respectively (a 10:1 ratio). These experiments in the environmental engineering laboratory were using river water from the first sample of water collection, and so small quantities of water were used in order to preserve as much of the water as possible for the actual lab testing. 400ml of water was measured out approximately by using a volumetric beaker. The pH was then adjusted by adding 1.25mol sulphuric acid to the sample to raise the pH to between 3 and 3.5.

2.3.2 Water Quality Analysis

Water quality sample tests were carried out for ammonia, nitrites, nitrates, phosphates, and COD. Using a continuous process means extracting the river water from each reactor at regular time intervals during the test. Before the test began, 100ml of each chemically pre-treated river water sample was mixed and the LCK water quality sample tests were run for time = 0s. From there, samples were taken every 15 minutes up to 1 hour, with the LCK tests being conducted in the time spaces between samplings. Previous studies suggest that the reaction peaks at around time = 30 mins and from that point, the reaction begins to follow a predictive trend. As such, it is not required to conduct the experiment for times above one hour, as the trend will most likely be very similar from 30 minutes onward and will also give a good extrapolation of how the trend behaves above 1 hour.

2.4. THE CHICK-WATSON DISINFECTION MODEL

Experimental data from tests undertaken in the past have been used to develop mathematical models to predict the bacterial survival result of water disinfection, and one simple model is the Chick-Watson model, as shown in Equation 4.2.1 and in the report by Cho [13]. This model is acceptable for use in this project since flow is assumed to be uniform. The Chick-Watson first order differential equation expresses both microorganism and inorganic inactivation rates. Corrections to the Chick-Watson model take more considerations for the test environment into account. The Chick-Watson model gives us a value for the bacterial survival rate, quantifying the effectiveness of disinfection in the form of a ratio.

$$[illegible] \frac{N}{N_0} [illegible] \frac{N}{N_0} [illegible] N [illegible]$$ Equation 2.4.1

where N is the bacterial density after exposure in colony forming units (CFU), N_0 is the initial bacterial density exposure (CFU/ml), N/N_0 is the bacterial survival ratio, k is the inactivation rate constant (cm^2/μWattmin), i is the intensity of incoming UV-A radiation (μWatt/cm^2), and T is residence time (minutes). The residence time is the time taken for influent water to pass through the reactor, and as such can be quantified by equation:

$$T\ (Residence\ Reactor\ Time) = \frac{V}{Q} \qquad \text{Equation 2.4.2}$$

The Resident Time in the reactor (T) is calculated with V, the total volume of the reactor, and Q, the flow rate of water passing through the reactor (litres/minute). The use of the Chick-Watson equation has a few requirements involved. Firstly, the need to prepare bacteria cultures in petri dishes enabling a count of N_0 and N. Secondly is the need for a peristaltic pump in order to set a flow rate and determine the residence time, T.

3. RESULTS AND DISCUSSION

Blank sampling testing to compare the pollutant levels over time in distilled water, river water and finally pre-treated water occurred in this study. This gave a good benchmark trendline for the water pollutant values and results showed the effect of both solar reactors and how they treated or remediation the river water in addition to the length of time photochemical treatment was needed in order to reach the approach water quality standards to that of clean potable water. The general trends of the curves presented in Figures 3 (a)-(d) illustrates that the concentration of river water pollutants is decreasing with time, indicating that the photochemical reaction is occurring at a relatively good rate. The performance of Solar Photoreactor 1 versus Solar Photoreactor 2 are presented in Figures 3 (a)-(d) with respect to effluent concentrations of nitrites, nitrates, phosphates, and COD. The experimental performance of Solar Photoreactor 1 demonstrated a higher kinetic reaction rate when compared with that of reactor 2, with both, however, being effective at reducing the water pollutant concentrations. The steeper gradients demonstrated a faster remediation or disintegration reaction time. There were no significant differences or drastic occurrences between the Chick-Watson disinfection model curves for the measured inflow and outflow concentrations, with a few minor anomalies with extremely low concentrations of nitrites and nitrates. The fact that these pilot-scaled experiments provided similar curve shapes for the two photoreactors suggests that the photo-Fenton reaction is occurring successfully in both experiments and confirms the reliability of the results. The data measured and generated from this study showed the efficacy of the Solar energy driven Photo-Fenton Processes and can be used when comparing the efficiency of alternative surface water treatment processes and water infrastructure.

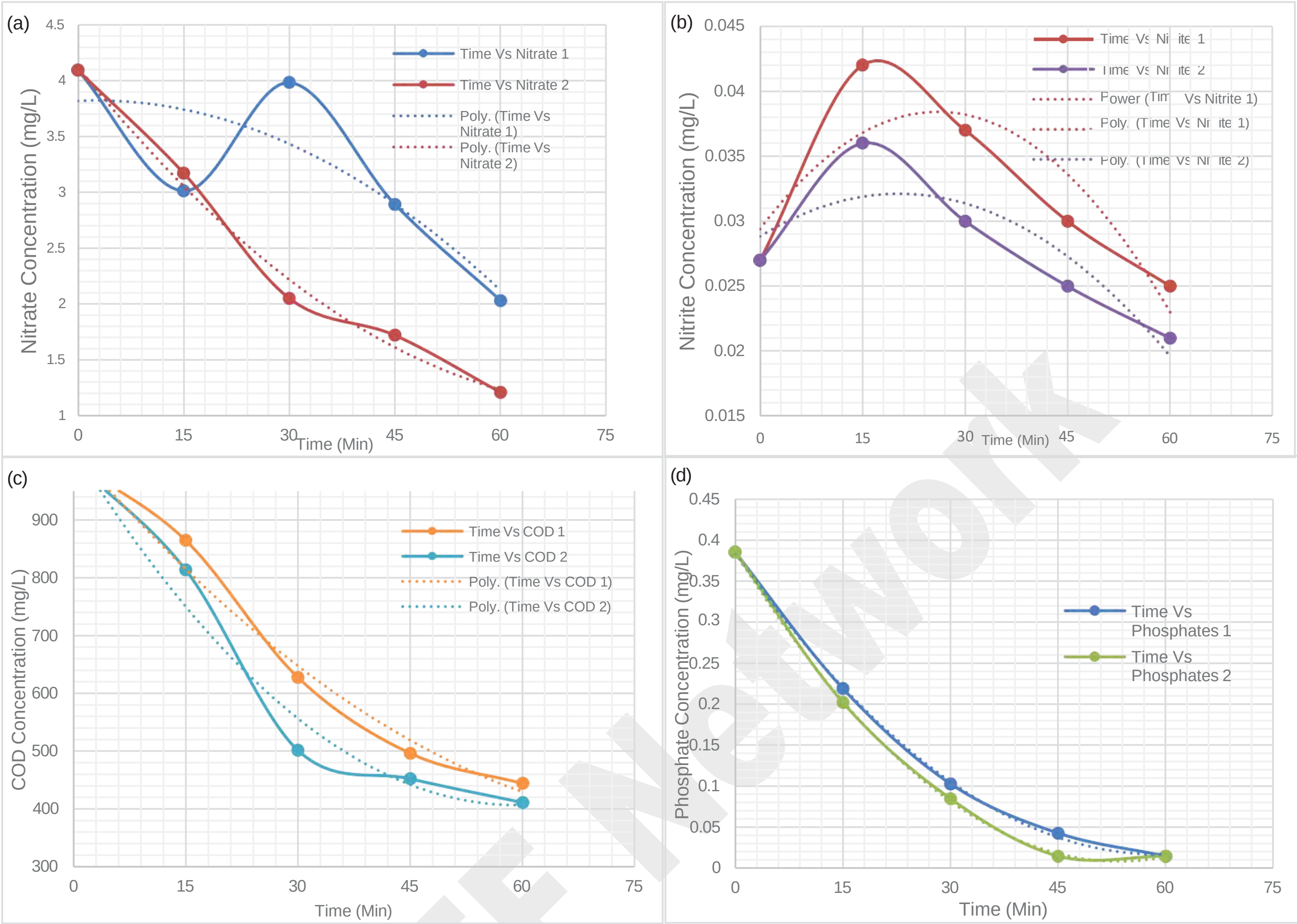

Figure 3 (a) Effluent Nitrate concentrations and polynomial best fit curves, (b) Nitrite Concentrations (mg/L) and polynomial best fit curves, (c) Chemical Oxygen Demand (COD) outflow measurements (mg/L) and (d) Phosphates outflow concentrations and polynomial curves.

The table below takes sources from various reputable names for water standards, such as the World Health Organisation (WHO) the United States Environment Protection Agency (USEPA) and the National Health and Medical Research Council Australia (NHMRC) [14-16] to find the acceptable standards of the above pollutants in drinking water. This information is contrasted against the pollutant levels achieved after this photo-Fenton treatment process in order to deduce where this effluent source lies in terms of how potable the water is.

Table 2: Comparing the Effluent Remediated River Water Quality parameters against that of Drinking Water

Pollutant	Drinking Water Standard * (mg/L)	Achieved Effluent Standard (mg/L)
Ammonia	0.5	<0.015
Nitrite	<3.0	0.0098
Nitrate	50-100	0.48
COD	0	388
Phosphate	<0.1	0.015
pH	6.5-8.5	4.3

*Data adapted from National Health and Medical Research Council and WHO [14-16].

The data collected on the pollutants tested for show acceptable levels of ammonia, nitrites, nitrates, and phosphate levels, although the COD and pH values were out of range at times and fluctuated. Other physical factors such as total suspended solids, total dissolved solids and turbidity were not tested for, although based on the visible colour of the effluent source it can be deduced that the physical water quality was not quite up to the required standards for potable water consumption. In order to make this process feasible for the St Mary's Island development, the photo-Fenton test would have to be preceded by screening and aeration systems in order to reduce the physical parameters before being treated by the photo-Fenton reaction to treat the chemical impurities, followed by the addition of lime and shock chlorination. The chain of processes as described is nothing short of a typical wastewater chain, and as such, this reactor design as the sole treatment may not be able to produce potable water from a river source. The experimental study showed that the chemical balance has more of an impact on effectiveness of treatment than any sub-variables such as the build of the reactor or the UV influx. This research project also demonstrated that the reaction is more successful at higher concentrations of H_2O_2. As such, a more successful reactor may use computer automation to regulate the quantities of chemicals being added to a batch of water. On top of this, the reactions are kept at optimal removal efficiency rates since the concentration of chemicals can be changed in very fine increments, compared with human input. Considering this, and based on the results from the laboratory and pilot-scale study, the photo-Fenton process can successfully, and efficiently, reduce the concentration of positively charged chemical ions in river water, stormwater, or wastewater if the water sample has a low turbidity and clearer colour. It is a cost-efficient process, when compared to energy-intensive processes such as Reverse Osmosis or other tertiary treatments that require expensive membranes and high pressures. It is also efficient at removing protozoa such as Cryptosporidium, and as such, the developing world countries may see the rise of wastewater treatment works that primarily feature a computer-automated photo-Fenton reaction combined with simple pre-screening, aeration and post-treatment with lime and shock chlorine to provide very cheap potable water.

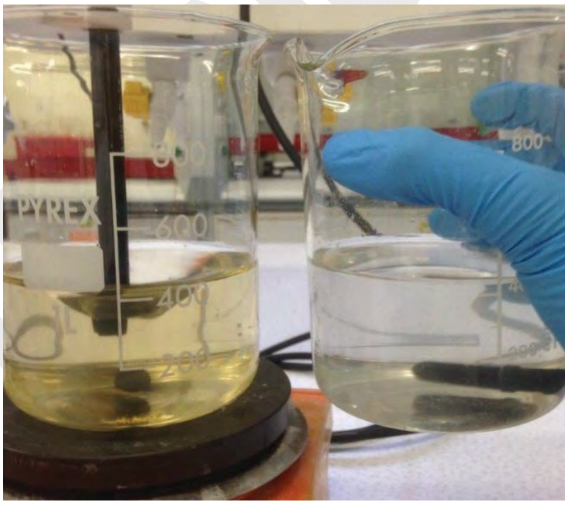

Figure. 4. Untreated Medway River Water (left) compared to Treated River Water (right)

4. CONCLUSION AND OUTLOOK

The borosilicate serpentine glass pipeline connections with self-tightening sealing bends provided very good air and water pressure sealing during this study. The initial trial and error assessment found the optimal chemical dosage and concentrations for the best removal efficiency taking a 100:1 H_2O_2: Fe^{2+} ratio as the starting point. The study demonstrated that the ratio of photochemicals is very influential to the success of other variables and thus paramount importance falls on finding the ideal concentrations and conditions. The project demonstrated ways of reducing the physio-chemical parameters in satisfying drinking water standards of the USEPA, WHO, DWI and NHMRC. The conclusions found from this research project are as follows:

- Under the conditions set by this research project, a smaller diameter of piping yields the best removal efficiencies of pollutants.
- An accurate chemical balance of H_2O_2: Fe^{2+} has more of an impact on the effectiveness of treatment than the use of UV radiation.
- The ideal H_2O_2: Fe^{2+} ratio for the treatment of river water is close to 100:1 mg/L.
- The impact of semi-variables such as diameter of pipe and UV influx on the effectiveness of treatment were not noticeable compared to the effect of changing the photo-Fenton chemical ratio.
- The photo-Fenton processes' removal efficiency is directly proportional to the remaining H_2O_2 concentration, as the reaction proceeds.

Combining automation and sufficient reduction of physical, chemical, and biochemical pollution parameters through the use of a good photo-Fenton reactor can reduce the need for extra infrastructure, whilst still providing potable water which may be a significant step forward for providing developing countries. The next generation of water and environmental engineers have the opportunity to contribute to topic of ongoing global research, by exploring into means of producing potable water in developing countries.

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COMPETING INTERESTS

The Authors declare that there is no competing interest.

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Potential Impact of Oil Spills in Coastal Waters on Water Supply

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ABSTRACT

It is apparent that oil exploration poses an inherent risk to water resources and water quality. This is exemplified by oil spills resulting from broken pipelines, underwater blowouts and oil transport vessel accidents. In these instances, water is usually the first casualty, as some form of water body is vulnerable to spilled oil, resulting in oil contaminated water. The freshwater resources of Small Island Developing States (SIDS) are often said to be stressed from anthropogenic pollution, which can lead to freshwater scarcity. In an effort to ensure the sustainability of freshwater resources resilience in SIDS, desalination is increasingly being used to provide potable water. Hence the quality of seawater deserves serious consideration. It is in this light that oceanic oil spills are of significant relevance to the provision of a guaranteed supply of potable water. For an oil producing small island developing state as Trinidad and Tobago, which has considerable oil and gas activities on land and in shallow waters along its coasts, the island's oceanic water can become increasingly stressed from oil spills, possibly leading to the shutting down of seawater intakes in the desalination process. A real-life seawater surface oil spill in the Gulf of Paria, south-west coast of Trinidad, not far from the largest desalination plant in the Caribbean, is investigated and its behaviour modelled, using numerical mathematical modelling techniques to produce trajectory plots. These plots are analysed to infer the potential impacts of the oil behaviour in the coastal waters on the island's domestic water supply.

Keywords: Small Island developing states (SIDS), oil spills, coastal water quality, freshwater resources, desalination, numerical mathematical modelling, Gulf of Paria, Trinidad

1. INTRODUCTION

Despite the global thrust towards renewable energy, several authors including [5], [8], [11], [17], [24] however, have stated that the increase in renewable energy is less than the overall increase in global energy demand, implying that renewable energy is not keeping abreast with rising energy demand. Therefore, it can be said that within the foreseeable future, this continued growth in global oil demand will continue to require more oil and gas activities such as exploration, production, refining and transportation. This gives more reasons to believe that the associated environmental risks will intensify, particularly, in the coastal marine environment. This is so, as water is often times the first casualty of oil exploration related accidents or faulty operations causing marine pollution such as oil spills which impacts the quality of the coastal waters. Small island developing states (SIDS) face threats to their freshwater resources from different forms of anthropogenic pollution that can lead to freshwater scarcity [6], [9], [10], [25]. One such pollution is oil spills, which is more likely to occur in oil producing SIDS. Trinidad and Tobago is an oil exporting SIDS [21] with considerable oil and gas exploration and production activities along its coasts, especially along the south-west coast in the Gulf of Paria. These industrial activities consume a lot of water, which is usually sourced from groundwater, aquifers, reservoirs or rivers, which are already in high demand in the residential, agricultural and industrial sectors. This competition for freshwater resources can engender water scarcity in the fossil fuel industry, and so in an effort to provide a solution to this problem while relieving the

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pressure of the natural freshwater resources for the island's domestic use, a 50 MIGD desalination plant was installed on the central west coast to provide as well as augment the supply of fresh potable water to the island. Desalination, broadly speaking, is the process of removing salts from seawater to obtain freshwater suitable for human consumption and other uses. However, the frequency and severity of any nearshore and near-surface oil spills can cause stress to the island's coastal seawaters due to oil contamination that adversely impacts its quality. This high oil content in the seawater can have a significant impact on the operational capability of a desalination plant. For example, [23] stated that the oil slick water reduces the intake performance of filters, causing biofouling due to the hydrocarbons and restricting the flow of the main intake manifold by clogging the screens. Hence, it can be seen that oil spills pose a clear risk to equipment failure in the desalination plant. This can disrupt the supply of freshwater to the island's main state-owned public utility water supply as well as other areas of Trinidad. This paper identifies and discusses the potential impacts of a near surface oil spill due to a broken jetty pipeline in the Gulf of Paria on potable water supply in Trinidad.

Almost nine years ago, an oil spill occurred at a loading jetty at Trinidad's largest refinery operations (PETROTRIN), in which over 7000 barrels of oil were released into the Gulf of Paria. The loading pipeline broke at a flange joint falling to an angle of approximately 15 degrees to the water surface, resulting in the injection of heavy bunker C fuel No. 6 oil into the shallow coastal waters of the Gulf of Paria. An aerial reconnaissance survey was conducted, and the oil was observed to be trapped underwater at a shallow water depth and travelling horizontally in the direction of the prevailing current before eventually resurfacing at some distance away from the spill site. Figure 1 encapsulates this entire the oil spill flow behaviour in flow field regions. In the near-field region, the oil is being discharged from the pipeline, the oil is suspended in the water column at a shallow water depth in the intermediate field region, and the far-field region, the oil re-surfaces. Note, the encircled area is the focus of the research.

This oil spill behaviour posed great interest because its behaviour fitted none of the two expected main types of oil spills, according to existing literature. The first type, water surface oil spills occurs when oil is spilled directly on the water surface, forming a surface slick, which is acted upon by a number of oil weathering processes that transform the oil over time and determine its fate. Figure 2 shows the typical configuration of a water surface oil spill and the different oil weathering processes; hence most water surface oil spill models are designed to simulate these processes and determine the transport of the oil [18], [19], [22], [30], [15], [1], [4], [20], [12], [14], [2]. The second type, namely underwater oil spills, occurs due to blowouts and underwater ruptured pipelines. These oil spills usually possess a near-field jet flow which is a flow behaviour driven by momentum and a far-field plume flow which is a flow behaviour driven by buoyancy. Hence underwater oil spill models are developed to simulate both the jet and plume flow behaviours [27], [28], [29]. Figure 3 shows the configuration of the common form of an underwater oil spill. Figure 4 however, illustrates that the configuration of the oil spill under study herein, does not conform to typical water surface oil spills, as the oil was not spilled directly on the water surface, but instead showed a jet and plume behaviour which are characteristic of underwater oil spills, and thus limited the use of existing oil spill models. The interest was further heightened when no oil slick was observed on the water surface at the beginning of the spill, an unexpected occurrence because oil is less dense than the seawater and should have floated. The eventual amount of sheen that appeared near the spill site could not account for the volume of oil spilled, making it necessary to determine, firstly where was the majority of oil, and then, how did the oil became suspended underwater and travelled a significant distance and then seemingly precipitously re-surfaced on the beach. It meant therefore, that a different approach had to be employed to capture the features of this oil spill.

This oil spill behaviour was therefore investigated by developing a novel water surface oil plume model. This was based on integral bubble plume theory and numerical mathematical modelling techniques to produce trajectory plots of the oil movement underwater for different values of three important dimensionless parameters including the entrainment of seawater into the plume, the detrainment of the plume fluid into the seawater and the drag coefficient to determine the force on the lateral boundaries of the plume as it moves through the seawater. This expansion in the range of modelling techniques allows for the determination of the fate and transport of an oil spill exhibiting such features. Having tools to predict such a phenomenon will be an important aid for quantifying the threat to the desalination plant and perhaps help in developing operational procedures to monitoring water quality and modifying plant operation This paper therefore, infers from the results of the model, the potential possible impacts of the suspended underwater oil plume having a somewhat long residence time in shallow water depths of the Gulf of Paria on the island's water supply.

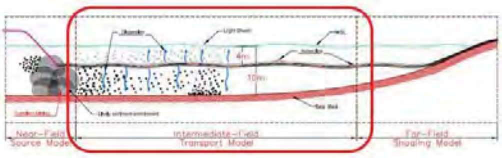

Fig. Error! No text of specified style in document.**:** The entire oil spill behaviour studied herein

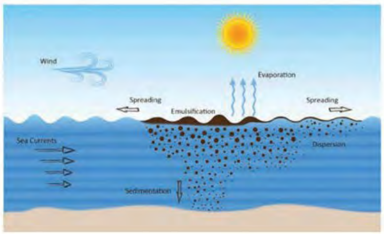

Fig. 2: Typical water surface oil spill flow configuration. *Source*: The sea: The Science of Ocean Prediction, Journal of Marine Research, 75, 923-953, 2017

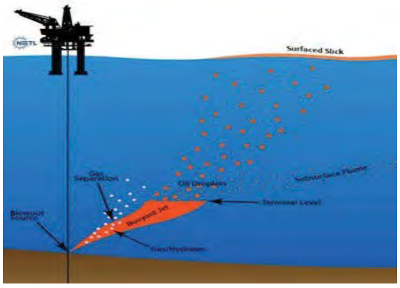

Fig. 3. Typical underwater oil spill flow configuration. *Source*: Offshore Technology (2020)

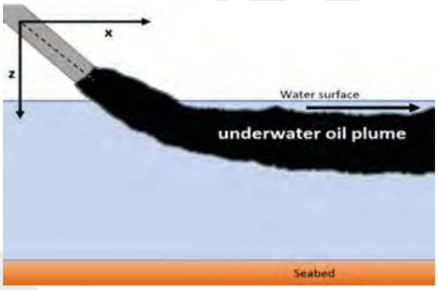

Fig. 4. The unusual water surface oil spill flow configuration studied herein

2. METHODOLOGY

In light of the fact that existing water surface oil spill models are not quite sufficient to handle the water surface oil spill research problem, it was useful to consider the modelling behaviour of underwater/subsurface oil spills, since the oil spill research problem herein, can be conceptualized as an underwater spill turned on its head, and more importantly, it involves oil jet and plume behaviours which are characteristic of underwater oil spills. The mathematical modelling of the oil spill flow behaviour is briefly presented in the next sections.

2.1 MATHEMATICAL MODELLING

The mathematical modelling methodology involved three main modelling steps. The details of the derivation of the equations for steps 1 and 2 are fully and completely discussed in the author's doctoral thesis [5] and so only the final set of derived equations for the step 3 are presented. The first step involved the derivation of the single phase plume flow in a cross-flow current with the effect of the entrainment of the ambient seawater fluid into the buoyant oil jet. Single-phase plume flows involve the dynamics of the same phase, for example, sewage water discharge from outfall discharges or heated water plumes from cooling plants. In single-phase plumes, the buoyancy is well-mixed with the bulk fluid**,** therefore the advection of buoyancy is controlled by the motion of the fluid. The equations derived were the continuity equation, the horizontal and vertical momentum equations and the buoyancy flux equation the spread equation, the x-direction geometric equation and the z-direction geometric equation. These equations form the basic mathematical model in which the two-phase plume flow will be incorporated. For this single-phase plume flow step, the research followed the work of [7], [26].

The second step involved the modification of the equations in step 1 to introduce the two-phase oil plume flow. The two-phase bubble flow is a flow of gas and liquid. Now, the main feature of two-phase bubble plume models is the separation of the dispersed phase from the continuous phase, and this is due to an important variable called the slip velocity. The slip velocity is the rise velocity of the dispersed phase relative to that of the surrounding liquid oil phase. Recall, the research is focused on an oil spill that involved an oil plume in seawater, so the existing two-phase bubble plume model of an oil and gas mixture in seawater by [16] was to guide the derivation of the equations to make the research model equations suitable and novel to handle the research problem. First, in the work done by [16], there was a slip velocity, however, for the research model, the slip velocity was set to zero, thereby making the oil plume fluid a homogeneous mixture. In the [16] model, ambient cross-flow was not included and vertical density variation was in the gas fluid and not in the seawater. However, for this model, vertical density variation was implemented in the seawater and ambient cross-flow current was included. Also, the [16] model did not cater for an inclined jet, so the jet angle of 15 deg. to the water surface was included. Another significant difference of existing two-phase bubble plumes is that they are usually vertical flows since they stem from underwater blowouts, but the research oil spill flow is predominantly a horizontal flow. Also, a mixture of both the water and oil densities were incorporated into the momentum equations, with the water density being a function of depth, thereby making the model an acceptable model for the research problem. The two-phase set of equations that were derived included water mass volume flux equation, oil mass volume flux equation, horizontal momentum equation, vertical momentum equation, buoyancy flux equation, x-horizontal geometric equation, the z-vertical geometric equation and the spread equation.

2.1.2 Two-fluid oil plume flow model equations

Step 3 involve the introduction of the two-fluid oil plume flow model. Interestingly, because of the complexity of the equations derived in step 2, an equivalent setoff equations were further derived with equal rigor and robustness of the original set of equations in step 2. The recent work of [13] was used to guide the derivation of the two-fluid oil plume. Although their work focused on subsurface spills, they used integral theory for the derivation of the governing oil spill flow equations of mass, momentum and buoyancy and treat the oil spill as a plume model under similar key conditions to the flow studied herein. These conditions include: the absence of a slip velocity (i.e. a homogeneous system), it can allow for jet angle inclination and the effects of entrainment of seawater into the plume and detrainment of oil into the seawater are considered and the ambient water density is a function of depth (i.e. vertical ambient stratification). The final equations are shown below in equation 1 through to equation 13:

The mean vertical velocity profile

$$u_{eg}(r, s_{eg}) = U_{eg}(s_{eg})e^{-\frac{r^2}{b^2(s_{eg})}} \quad (1)$$

The local mean oil fraction

$$f(r, s_{eg}) = c(s_{eg})e^{-\frac{r^2}{A^2 b^2(s_{eg})}} \quad (2)$$

$$p(r, s_{eg}) = f(r, s_{eg})p_o + [1 - f(r, s_{eg})]p_w(z) \quad (3)$$

The total mass flux equation

$$<(s_{eg}) = \int_o^{\infty} p(r, s_{eg})u_{eg}(r, s_{eg})2\pi r dr \quad (4)$$

The oil mass flux equation

$$<_o(s_{eg}) = p_o \int_o^{\infty} u_{eg}(r, s_{eg})f(r, s_{eg})2\pi r dr \quad (5)$$

The total momentum flux equation

$$M(s_{eg}) = \int_o^{\infty} p(r, s_{eg}) u_{eg}^2 (r, s_{eg}) 2\pi r dr \quad (6)$$

The total buoyancy flux equation

$$B(s_{eg}) = g \int_o^{\infty} [p_w(z) - p(r, s_{eg})] 2\pi r dr \quad (7)$$

The conservation of total mass in the plume

$$\frac{dcp(s_{eg})}{ds_{eg}} = 2\pi b(s_{eg})[U_{eg}(s_{eg}) - U_{CX} \cos a_r (s_{eg})](ap_w(z) - \Delta p_o) + 2\pi b(s_{eg}) U_{eg}(s_{eg}) U_{CX} \sin a_r (s_{eg}) \quad (8)$$

The conservation of the oil in the plume

$$\frac{dcp_o(s_{eg})}{ds_{eg}} = -2\pi b(s_{eg}) c(s_{eg})[U_{eg}(s_{eg}) - U_{CX} \cos a\, r(s_{eg})] \Delta p_o \quad (9)$$

The horizontal momentum equation in cross-flow

$$\frac{dM(s_{eg})}{ds_{eg}} = -C_d p_w(z) 2\pi b(s_{eg}) U_{eg}^2 (s_{eg}) + B(s_{eg}) \sin a_r (s_{eg}) + U_{CX} \cos a_r (s_{eg}) \frac{dcp(s_{eg})}{ds_{eg}} \quad (10)$$

The vertical momentum equation in cross-flow

$$M(s_{eg}) \frac{da_r(s_{eg})}{ds_{eg}} = B(s_{eg}) \cos a_r (s_{eg}) - U_{CX} \sin a_r (s_{eg}) \frac{dcp(s_{eg})}{ds_{eg}} \quad (11)$$

The spatial x-coordinate

$$\alpha_{seg} z \rho_{\Lambda} s_{eg} \lambda_{\vartheta} = \sin \alpha_r \rho_{\Lambda} s_{eg} \lambda_{\vartheta} \quad (12)$$

The spatial z-coordinate

$$\alpha_{seg} x \rho_{\Lambda} s_{eg} \lambda_{\vartheta} = \cos \alpha_r \rho_{\Lambda} s_{eg} \lambda_{\vartheta} \quad (13)$$

Equations 1 through to 13 represent the movement of the liquid oil in seawater. Therefore, the final model becomes a two-fluid oil plume flow model. This means the model caters for liquid oil (fluid 1) in seawater (fluid 2). This model developed to simulate the underwater behaviour of oil spills located at the sea surface that particularly behave as a jet due to a protruding oil source (e.g. a broken pipe) into shallow ocean depth (< 10m) with slight vertical density variation and crossflow current. All these equations shown were solved in **MATHEMATICA**.

3. RESULTS AND DISCUSSION

The model developed in this research is a phenomenological model that considers parametric changes to observe a particular phenomenon or flow behaviour. A number of flow cases were set up under certain parametric conditions to observe the different flow behaviours, again, these flow cases are fully explained and presented in the author's doctoral thesis [5]. Therefore, a small appropriate sample of the results is presented here, which takes the form of a set of trajectory graphical plots, in which the blue line represents the oil path. The following oil spill data was incorporated into the model:

- The angle of the discharge source (jet angle) is at 15 degrees to the water surface.
- The oil has an API gravity of 13.6, hence its specific gravity can be calculated to be 0.98.

- The density of the oil is 974.21kg/m^3.
- The discharge velocity of the oil is 1m/s
- while the seawater velocity is 0.2m/s.

Recall, the main parametric values are the drag coefficient, the rate of entrainment and the rate of detrainment. Table 1 shows the conditions that allow the oil to become neutrally buoyant, the trajectory plot associated with this condition is shown in Figure 5. Figures 6 and 7 are very interesting and important plots that show the suspended travelling a horizontal distance of 500m and 300m respectively.

Table 1. Parametric conditions for the oil to become suspended/neutrally buoyant

Jet Angle, α_T	Drag Coefficient C_d	Rate of Entrainment, α	Rate of detrainment, β	Oil Velocity, U_o	Seawater Velocity, U_∞	Oil Density ρ_o
-15^0	0.00008	0.007128	0.00007	1m/s	0.2m/s	974.21kg/m^3

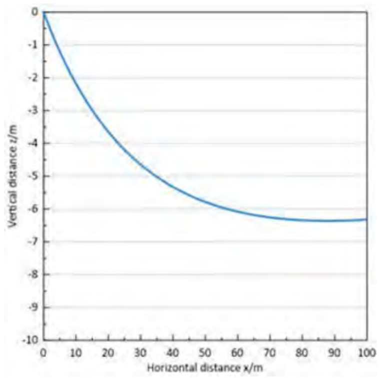

Fig. 5. Trajectory plot for oil-water plume flow under conditions for oil to become suspended or neutrally buoyant

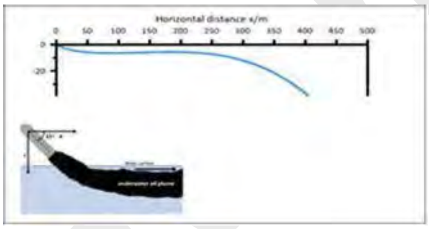

Fig. 6. Trajectory plot for oil-water plume flow under conditions for oil to become suspended or neutrally buoyant at a horizontal distance of 500m

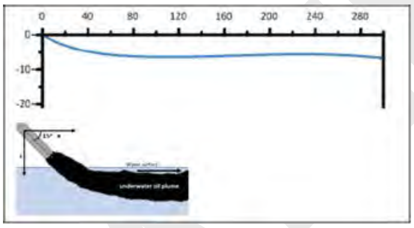

Fig. 7. Trajectory plot for oil-water plume flow under conditions for oil to become suspended or neutrally buoyant at a horizontal distance of about 300m

3.1 Discussion

The water surface oil plume model designed herein allows users to make the most appropriate values of the main parameters. Figure 5 shows the the main model results, as it depicts the oil becoming suspended in the water column. This plot shows that as the oil plume drops below the water surface to a shallow depth between 6 and 7m, it begins to experience neutral buoyancy as the flow appears to be horizontal at a distance of about 70m, and seems to be moving downstream. The explanation for the oil becoming suspended when released from a broken water surface pipeline and injecting heavy oil into the water column, is that due to the low ambient velocity, the entrainment of the water into the plume is greater than the detrainment of the oil into the water thereby forming a water-in-oil emulsion which is very voluminous, viscous and stable over time, which means it can persist for some time after the spill has occurred. Therefore, the formation of the oil plume emulsion along with ambient stratification enable the oil to become trapped in the water column and carried downstream by the ambient current due to its high viscosity. This main model result is verified by the observations of an aerial survey stated in the local unpublished oil report, which states that: "A long, narrow patch of oil appears to be trapped within the water column and hence under the water surface" [3]. Figure 6 is a very interesting trajectory plot as it shows the oil falling underwater and remaining suspended at a shallow water depth for about 300m before moving downwards towards the seabed. This plot indicates that the water-in-oil emulsion form of the plume has travelled a significantly long distance, before eventually destabilizing and falling onto the seabed. This model result is quite significant, as it shows the model's capability to capture this significant flow behaviour of the oil falling onto the seabed, since it was thought that the oil immediately resurfaced after being suspended and moving for a long distance. This result is strongly indicating therefore, that when the oil fell onto the seabed, it shoaled, that is, the oil was acted upon by the ambient sea motion/wave action which eventually pushes the oil onto the beach. Figure 7 is another interesting plot, as it shows in more detail, the full extent of the distance the oil travelled before destabilizing. This plot indicates that the model can show that the oil can be suspended for a fairly long distance before becoming disrupted. Based on the water surface models seen throughout literature, this phenomenon has not been fully captured in this way (which is, the movement of an oil spill travelling at such long horizontal distance underwater) by existing oil spill plume models.

According to [24], oil spill in seawater causes changes in its thermophysical properties such as density, pressure, temperature, thermal conductivity and dynamic viscosity. With the model results indicating that the oil travelled underwater for a significant distance implying its high residency in the water column before destabilizing, it can be deduced that the thermophysical properties of the oil contaminated seawater may have been impacted in such a way that can cause increased difficulty in treating it to an acceptable quality for human consumption. From the model results, it can be inferred that the submerged emulsified oil, which is often referred to in literature as a 'chocolate mousse', because of its thick, dark appearance can destroy the efficiency of the desalination plant by potentially coating the seawater intake equipment. The work of [24] stated that oily seawater intake reduces the thermal performance of the heat exchangers, leading to a decrease in the productivity of distilled water. Another impact of the submerged oil studied herein, is that, it can lead to fouling of the heat transfer tubes and disruption in the heat transfer process, since in general oil adheres to heat transfer surfaces.

4. CONCLUSION

The above discussion confirms that oil spills adversely impact coastal seawater quality and are indeed a threat to desalination plants. Oil present in seawater, makes the desalination process challenging. Since most types of oils do not get mixed with water, due to their lower density, they typically cannot be extracted via the desalination process. Hence, desalinated water containing oil would also affect the quality of the produced freshwater. It is also clear that an oil spill can lead to the shutting down of the desalination plant which in turn can disrupt the production of freshwater supply for the island. The work done in this study shows that analysing the potential impacts of oil spills on desalination plants allows for policy making, in that, it informs the decision for providing policy for suitable site selection for a desalination plant as well as determining acceptable seawater quality to decrease the vulnerability to desalination plants allowing them to provide constant freshwater supply to the island. As mentioned earlier, the spill occurred in the Gulf of Paria, which is a semi-enclosed sea located between Venezuela and Trinidad in the southern Caribbean, known to experience significant current reversal along its coast. Hence, in this oil spill event, the results of the oil spill trajectory confirmed that the desalination plant was not threatened because it was located north of the spill location and the slow-moving prevailing current at the time of the spill carried the oil plume southward. Under these conditions, any desalination plant built south of the spill site or downstream of the prevailing current of the spill site will be under threat and may have interruption of its operations for supplying potable water to the island. However, the situation could have been quite different if at the time of the spill, under certain climatic and ambient conditions, for example rain or wind and strong Orinoco river flow from the Venezuela mainland, the prevailing current was reversed and was instead northward. The application of the model under such circumstances would show movement of the oil plume northwards and in the direction of the desalination plant, such would have adversely threatened the operations of the desalination plant. The research provides insights into the mechanism of underwater transport of oil spills and would be useful tool in developing spatial plans for sighting any future desalination plants in which there is a real threat that accidents from oil facilities may spew oil into the coastal waters.

COMPETING INTERESTS

The author declares no competing interests.

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Water Efficiency Conference 2022

The Impact of Climate Change on the Navet Reservoir, Trinidad

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ABSTRACT

A hydrologic study of the Navet reservoir and its catchment was conducted to investigate and evaluate the potential impacts of climate change on it, using the Soil Moisture Accounting algorithm in HEC-HMS to perform continuous simulations. The catchment is partially gauged, with a single rainfall gauge located within it and with the absence of a stream gauge, stage data from the reservoir was used to evaluate catchment response.

The selection of model parameters was based on previous work done on the nearby Nariva catchment and were improved on by a manual optimization technique. The model was subject to a split-sample test with a calibration period of 24 months (2003, 2004) on a daily time-step followed by validation over a period of 60 months (2005-2009).

Upon successful validation, the model was used to evaluate the system's response to climate change. The meteorological data for this was generated by the PRECIS software for this region. The model was subject to three scenarios based on the SRES A1B scenario. The results of simulations for the period 2030-2096 showed that for successful operation, production rates at the Navet reservoir requires a 40% reduction of present values for two of these scenarios and by 30% for the most optimistic scenario.

Keywords: Continuous Hydrologic Modelling; Soil Moisture Accounting; HEC-HMS; Navet Reservoir

1. INTRODUCTION

The Water and Sewerage Authority (WASA) of Trinidad and Tobago has the mandate to deliver a safe, reliable and efficient water supply to satisfy the demands of all sectors of its population. In recent decades however, it may be perceived that the authority has faced an increasing challenge in meeting this mandate. This becomes particularly evident during the months of May to August each year when the islands' surface water reservoirs approach their minimum pool levels. Water management strategies such as rationing and use restrictions often result in constrained commercial activities, short-term closure of water dependent businesses and a general angst amongst citizens about short-term availability.

Regionally, there are predictions of more frequent and intense storm events and hurricanes. These effects can lead to changes in the total precipitation, its seasonal distribution, frequency and intensity. These conditions together with changes in evapotranspiration may affect the magnitude and timing of runoff, and subsequently, the intensity of floods and droughts [1].

Historically, Caribbean territories have not been as severely impacted by droughts as other parts of the world and when they do occur, reactive responses have resulted in short-term solutions. However, the occurrence of droughts is a normal part of climate variability [2] and since there is evidence that both the duration and frequency of droughts are increasing there is need for study of the long term impacts and the development of strategies to improve resilience to future droughts.

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A meteorological drought analysis performed by Beharry et.al in 2019 using 35 years (1980-2014) of monthly totals from fourteen rainfall stations across Trinidad identified drought variability between stations in the north and central regions of the island. The stations at the islands three major dams Arena, Hollis and Navet shown in Figure 1 were among those analysed. Over the study period, it was found that the frequency of meteorological droughts decreased at the Hollis Reservoir, while at the Navet Reservoir there was an increase, implying a drying pattern and possible adverse effects on the water resources of Trinidad [3].

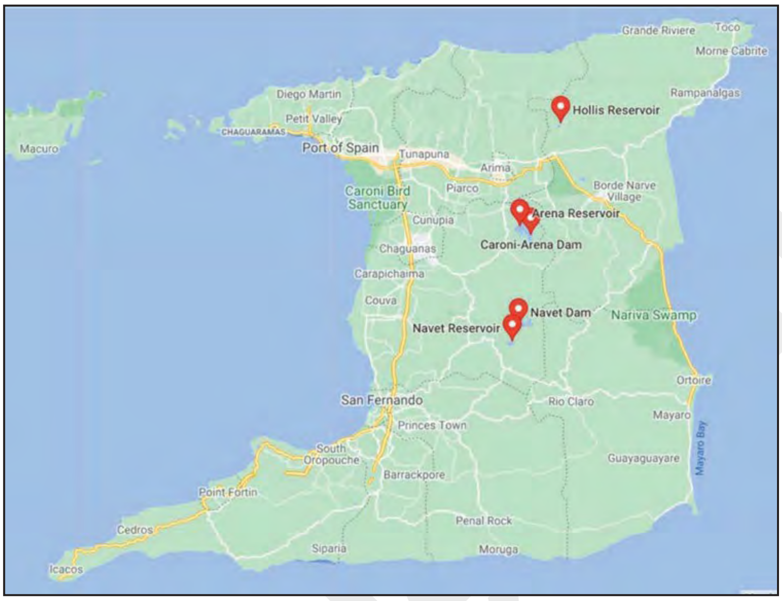


Figure 1 Map of Trinidad showing the location of its three major surface reservoirs, Hollis, Arena and Navet

With multiple authors identifying the need for further specific studies to understand the impact of climate change on our water resources, this research looked at the potential impacts of climate change on the hydrological response of the catchment of the Navet Reservoir. The method of approach involved the use of predicted hydro-meteorological data input to a hydrologic model of the catchment to gauge its future response.

The Navet Reservoir System (NRS) consists of interconnected dual reservoirs, their catchments and the Navet Water Works (NWW), an onsite water treatment plant. The larger reservoir is the smallest of the three major surface water impoundment facilities operated by the WASA. Existing literature sources often refer to the reservoirs as the Navet High Dam and Navet Low Dam, for this article, they will be referred to as the Navet Reservoir Upper (NRU) and the Navet Reservoir Lower (NRL). The NRU is the larger of the two reservoirs, with a storage capacity of approximately 19 million cubic meters and was created by the damming of the Navet River and a major tributary. Its catchment is approximately 17.57 km^2. The NRL is downstream and intercepts the Navet River and a second tributary, it collects water from a lower catchment 9.13 km^2 in size and adds a further 4 million cubic meters of storage capacity to the NRS. Overflow from the NRU continues along the Navet river which eventually enters the NRL.

The hydrologic model used to simulate the response of the NRS is categorised as a conceptual model. This model type uses a number of conceptual, interconnected reservoirs which represent the physical elements in a catchment. These elements are recharged by rainfall, infiltration and percolation, and are emptied by evaporation, runoff, drainage and seepage. Semi-empirical equations relate the reservoirs and model parameters are assessed from field data which are modified during model calibration.

The challenge in using this type of model is that its parameters although related to the physical features within the catchment are not measurable. This can lead to the selection of unrealistic parameter values while attempting to calibrate the model. Traditionally, this has made these types of models undesirable for use on ungauged catchments since calibration may be challenged. However, work by researchers such as Fleming and Neary [4] to relate the parameter values to physical measurable catchment properties has enabled the estimation of the major model parameters from its physical features, and observations of streamflow records.

Meteorological data on climate change were derived from outputs of the PRECIS (Providing Regional Climates for Impact Studies) software. PRECIS is a regional climate model used to downscale global circulation models to a regional level. This consisted of daily rainfall, maximum and minimum temperatures. The hydrological model of the NRS was created within the Hydrologic Engineering Centre Hydrologic Modelling System (HEC-HMS) software. After successful calibration and validation stages, the meteorological data obtained from the regional climate model was used as input to the hydrological model. Model outputs gave an insight to the response of the catchment to possible future conditions. The results of this approach can become an effective tool in guiding water management strategies for the future. Therefore, the objectives of the study were (1) to calibrate and (2) validate a hydrological model of the NRS, then (3) to evaluate the hydrologic response of the NRS to anticipated climate change.

2. METHODOLOGY

The HEC-HMS 4.4 software used to create the conceptual model of the NRS was designed to simulate the complete hydrologic processes of dendritic watershed systems [5] and is a public-domain model which has been in existence since 1992.

Modelling of the NRS was done in four phases. In the initial phase, a basin model of the NRS was configured where hydrologic elements were created and linked. Parameter values were then assigned to these elements and the processes linking them. There are four elements in the basin model shown in Figure 2 below. The Navet Water works element was created as a Sink which draws water from the Navet upper dam while the Navet low Dam was created as a Source since its function is as a water supply to the upper dam. The Upper Navet catchment was created as a sub-basin element which allows its hydrologic processes to be simulated. This element also inputs flows to the upper dam as a catchment output. The Navet Upper dam was created as a reservoir element accepting inflows from both the low dam and upper catchment. Water leaves the element via pumping to the water works and a spillway which is not shown in the basin model.

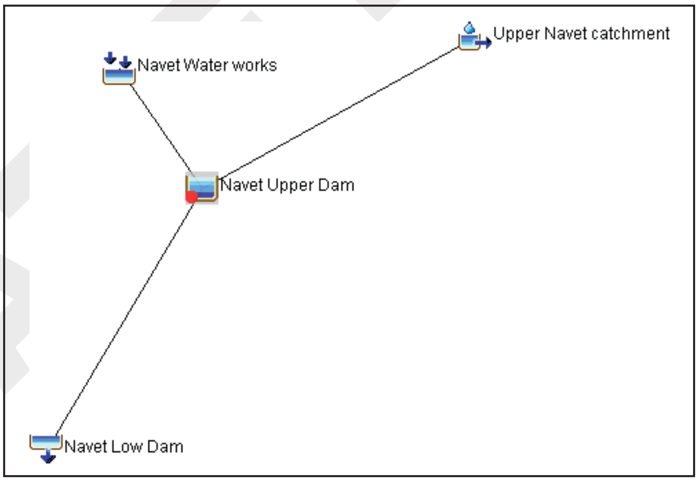


Figure 2 Basin Model of the NRS

The second stage was a calibration process that was performed to determine the suitability of the model for representing the NRS. Observed meteorological data for a select period was input to the model and a comparison made between the modelled output and observed data for the same period. Each model parameter value assigned in the initial stage can vary within a realistic range which is dependent on the physical properties of the catchment. During model calibration these parameters are varied within their ranges until its output is a good fit to observed values. An optimal combination of these parameter values is required for the modelled output to simulate the catchment response. In HEC-HMS this parameter optimization procedure can be automated where an algorithm continuously tries combinations of parameters and compares its output to known streamflow records. This approach could not be used because of missing

streamflow records for the catchment. Instead, catchment response was derived using the daily water level records for the NRU. A manual trial-and-error was done for finding the best set of parameter values, the basis being the goodness of fit between simulated and observed lake levels.
Model validation was done using a split-sample test (SS) [6] which involves the validation of a model based on a period of data other than that used in the calibration phase. Refsgaard and Knudsen defined the process of validation as demonstrating that a site specific model can make accurate predictions outside of the calibration period. The model is considered validated when it produces results within an acceptable range of the observed condition.
In the last phase of the modelling process, the validated model was then used to simulate the response of the hydrologic system to hypothetical future conditions. The objective of this stage was to determine catchment response to future rainfall and temperature changes. The Penman-Monteith equation was used to estimate evapotranspiration using temperature predictions from the PRECIS model. A time-series for both rainfall and evapotranspiration for the period 2030 to 2096 formed the meteorological data set used in this stage.

2.1 Data Collection

2.1.1 Catchment Characteristics

Information on the physical attributes of the NRS and its catchment was derived from digital shapefiles of land-use, soil types, contours and spot heights using a desktop GIS software. A watershed layer and a digital elevation model (DEM) were used to delineate the catchment boundary of the upper reservoir and other catchment properties such as area, slopes and length of flow paths. The NRU catchment was represented as a sub-basin element in the basin model and since its hydrologic response is an inflow to the NRU its properties were needed for the model setup.

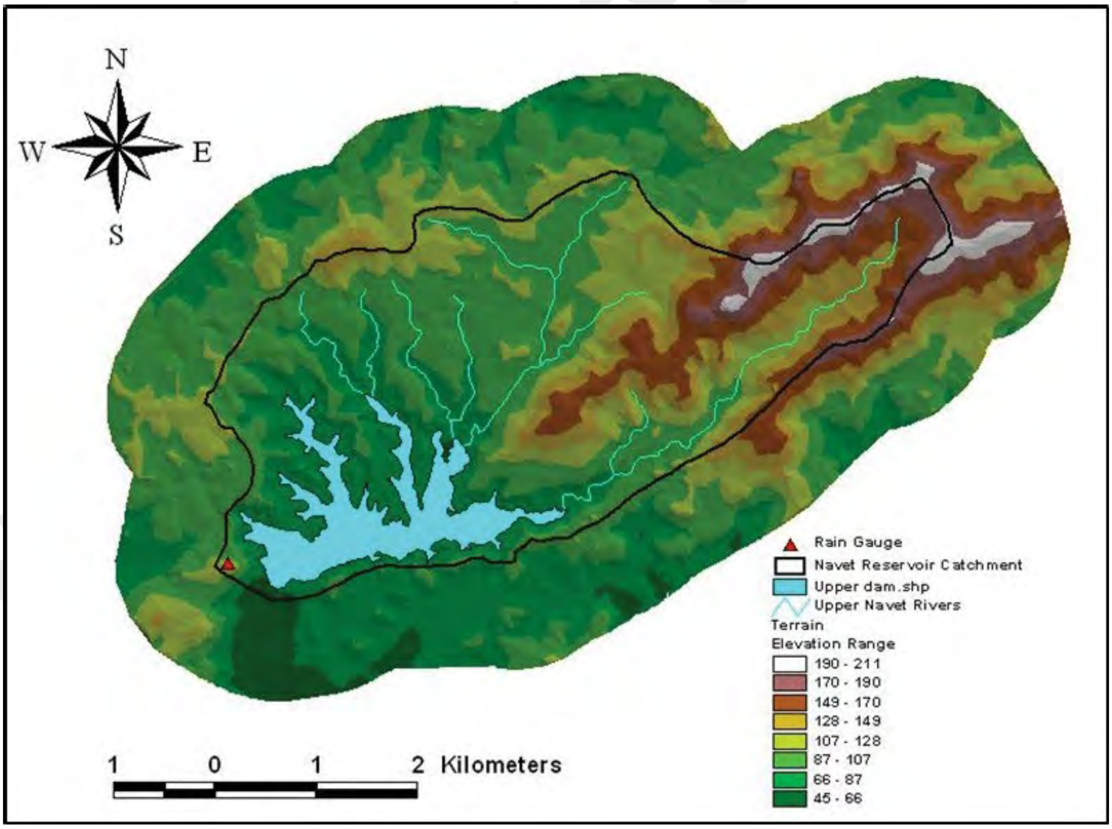


Figure 3 Digital elevation mode of the NRU and catchment

2.1.2 Meteorological Data

Meteorological data retrieved from the Water Resources Agency (WRA) consisted of daily rainfall and evaporation data recorded at the NWW. Daily production volumes from the water treatment plant and water levels in the upper reservoir were also provided. A seven-year period was identified from the year 2003 to 2009 as it was the longest period for which a complete record of daily rainfall was available. This rainfall was treated as being representative of the entire catchment because of a lack of data from neighbouring catchments for the same period.

Measured evaporation data was limited to 2003-2004 and a few days of 2005 Therefore an alternate source of evapotranspiration data was used. These values were reference crop evapotranspiration generated by the Crop Water Requirements (CROPWAT) software from data collected at the Navet dam site [7] and reported by Ekwue et al. in 2015. The values were generated as daily averages for the period 2001-2012 and were converted to monthly averages (Table 1) for use in this study.

Table 1 Monthly average evapotranspiration

Navet Reservoir	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov	Dec.
ET (mm/mth)	112	106	135	128	116	99	114	124	120	115	89	103

2.1.2 Meteorological Data for Climate Change

The data used to simulate the hydrologic system response to climate change consisted of daily rainfall totals and estimated daily evapotranspiration values generated by the Climate Studies Group Mona (CSGM) of the University of the West Indies, Mona, Jamaica. These projected values were the results produced by a Regional Climate Model PRECIS (Providing Regional Climates for Impact Studies) which downscaled the global model HadCM3 (Hadley Centre Coupled Model, version 3) to a regional level. The global model was set up for Special Report on Emissions Scenarios (SRES) A1B scenario. This scenario assumes a future world of very rapid economic growth, low population growth and rapid introduction of new and more efficient technology. Major underlying themes are economic and cultural convergence and capacity building, with a substantial reduction in regional differences in per capita income. In this world, people pursue personal wealth rather than environmental quality [8]. For the NRS, three model simulations were used:

- AENWH which represents the standard unperturbed model for the A1B SRES (with original parameter settings)
- AEXSA and AEXSK which are based on the AENWH model but with two variations in parameters.

The differences in the model outputs to these three scenarios are described by (Wilson and Cooper 2017). The authors compared IDF curves for all three models and noted that

- AENWH: shows decreases in intensity of events at most durations, with greater decreases shown for the most extreme events, ranging from decreases of ~30% (1-day, 100-year) to ~15% (10-day, 100-year).
- AEXSA: shows increases in intensity for all events, with the largest proportional increases in smaller more frequent events (e.g. the 2-year, 4-day event shows an increase of ~80%).
- AEXSK: shows increases in intensity for more frequent events (2, 5 and 10-year), but decreases in the less frequent, larger events (50 and 100-year) [9]

2.1.3 Reservoir Operations

Daily water production values, reservoir levels and Low Dam water extraction values were used to represent the reservoir operations. The water removed from the upper reservoir for treatment at the NWW is taken as the main output from the system. The same was done for the low dam production values which represented water transferred to the upper reservoir to maintain its stored capacity above the minimum pool level.

The spillway crest of the upper dam was modelled as a broad crested weir 10m wide [10] using a coefficient of discharge of 1.6 (typical value used for this type of weir) with a crest elevation of 94.7m This elevation represented the 100% storage capacity of the reservoir. An elevation-discharge curve was generated for the spillway using the general weir equation. The maximum elevation used on the spillway curve was 95.7m which is just above the maximum lake level observed for the study period. This equation allowed spillage volumes to be evaluated by the model and simulate the filling and emptying of the reservoir. Water levels above the maximum storage elevation was removed from the reservoir as spillage. This water empties into the Navet River and contributes to the inflow to the lower reservoir. This outflow was not connected to the lower reservoir in the model since limited information on the low dam only allowed it to be modelled as a water source and not a reservoir with its associate catchment inflow and outflow.

The daily storage capacity of the upper reservoir was calculated using the daily reservoir levels provided and its storage elevation curve. An elevation-surface curve was used to find the surface area of the reservoir to estimate daily evaporation losses.

2.2 Model Setup

2.2.1 Basin Model

The basin model used to describe the NRS consists of the upper reservoir which collects runoff from the Upper Navet catchment as well as a pumped supply from the Navet low reservoir. Water is extracted from the upper reservoir to supply the Navet waterworks which in turn outputs potable water to WASA's distribution network. The lower reservoir is located downstream along the Navet River which was dammed to form the upper reservoir. This reservoir collects runoff from a separate downstream catchment as well as excess whenever the upper reservoir spills.

2.2.1.1 Soil Moisture Accounting

The setup of the SMA within the HEC-HMS software requires the input of values for sixteen parameters. These parameter ranges were compiled by Wilson and Cooper (2017) by examination of those listed in literature. There were also recommended values based on knowledge of the physical properties of the Nariva catchment. These parameters allow the modelling of the hydrologic processes of interception, surface depression storage, infiltration, soil storage, percolation, and groundwater storage. The maximum depths of each storage zone, the percentage that each storage zone is filled at the beginning of a simulation, and the transfer rates, such as the maximum infiltration rate, are required to simulate the movement of water through the storage zones. Besides precipitation, the only other input to the SMA algorithm is a potential evapotranspiration rate [4]

2.2.2 Meteorological Models

A meteorologic model within the HEC-HMS tells the model how meteorological information is to be used. For continuous simulation of the hydrology of the catchment, evapotranspiration (ET) is a key component. This information is entered into the model as a time-series or alternatively an internal algorithm can model potential evapotranspiration using climatic data such as observed longwave and shortwave radiation.

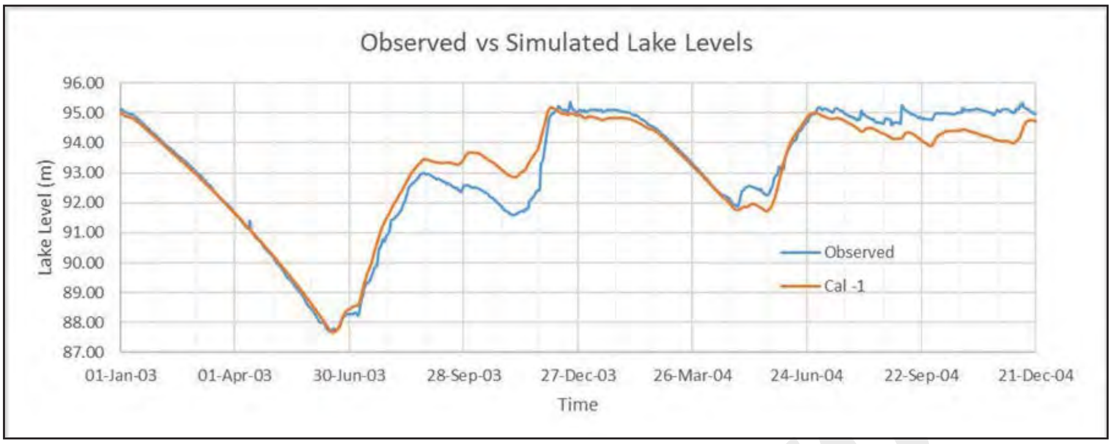
For the NRS during the period of study 2003 to 2009, there were two sources of evaporation data available for use in the meteorological model. These were:

- Measured daily pan evaporation at the Navet Dam site for the period 2003-2004 (2 years)
- Daily reference crop evapotranspiration generated by the Crop Water Requirements (CROPWAT) software from data collected at the Navet dam site [7] for the period 2001-2012

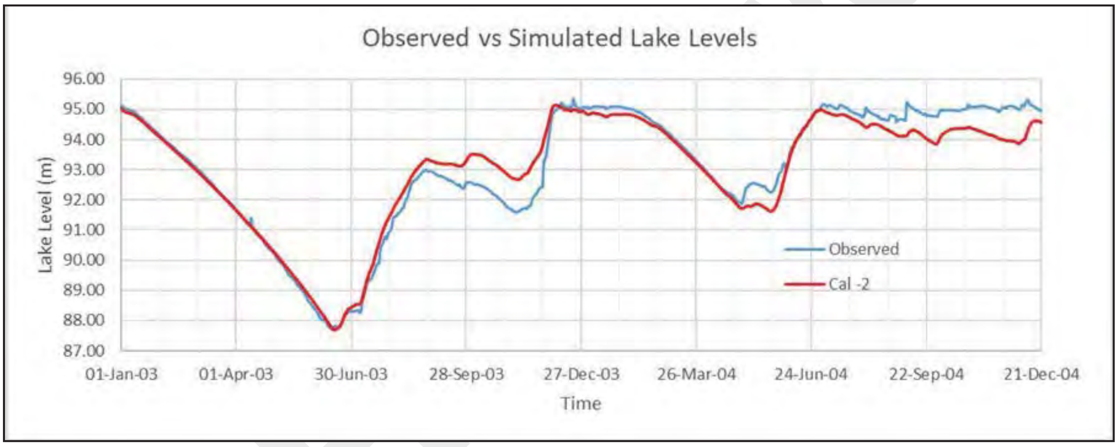
2.2.3 Model Calibration and Validation

Model calibration was performed for a twenty-four-month period from 01Jan2003 to 31Dec 2004. The plots in Figure 4 below show the comparison of observed water levels in the Navet Upper reservoir to those simulated by the model for the calibration. Inputs to the model for the calibration period were:

- Observed total daily rainfall in mm
- Total daily Evapotranspiration estimated from observed pan evaporation data in mm
- Water transfer from the lower reservoir into the upper in m^3/s and
- Raw water extraction from the upper reservoir in m^3/s.



(a)



(b)

Figure 4 - Observed Lake level vs Simulated levels for the Calibration Period using (a) measured pan evaporation data and (b) simulated ET from CROPWAT model

The lake level of the upper reservoir was used as the benchmark for evaluation of the model performance. This data was the only observable response of the catchment to rainfall input. The observed levels are represented by the blue lines on each of the plots. The calibration step used the observed evaporation data (2003, 2004) to create the meteorological model for the CAL-1 plot and the values generated by the CROPWAT model were used to create meteorological model by the Cal-2.

The model performance was evaluated for these alternative sources of evapotranspiration data to determine their suitability for use in the validation period. A correlation analysis performed on the model outputs using both sources gave identical correlation coefficients of 0.97 indicating high positive correlation between model outputs and observed values. Therefore, the CROPWAT data was suitable to supplement measured evaporation data for the validation period 2005-2009.

Model validation was performed for a second, 5-year period 2005-2009. For this period, the CROPWAT monthly values were used since there were no observed evaporation data. Figure 5 shows the observed levels represented by the blue line on the plot with Val-2 representing the model response during the validation period. This line has a 0.995 correlation coefficient. This high positive correlation for Val-2 indicates an almost perfect simulation of the validation period. This can be attributed to the inputs of air temperature, wind speed, humidity, radiation and rainfall used in CROPWAT were collected at the Navet Dam site for a period (2001- 2012) which encompassed the period for this study.

The model was again successful in simulating the response of the catchment for the validation period. The model accurately simulates both the pattern of the filling and emptying of the reservoir as well as the observed peaks and lows. The model was successfully validated.

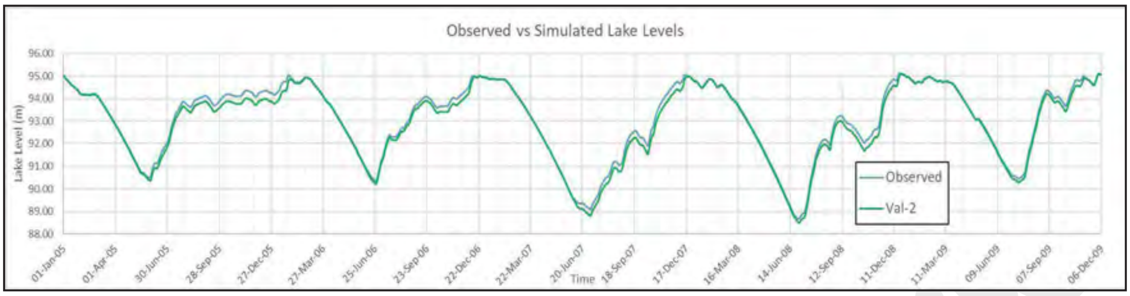


Figure 5 Observed Lake level vs Simulated levels for the Validation Period

3. RESULTS AND DISCUSSION

Having passed the calibration and validation split sample test, the model was used for predicting the catchment response to climate change. This modelling stage required the generation of hypothetical data sets. The inputs of rainfall and evapotranspiration are available from the PRECIS model for three scenarios, AENWH which favours a negative water balance while AEXSA and AEXSK may favour a positive water balance. Daily total rainfall and reference evapotranspiration values for all three scenarios are input directly into the model without changing to the model parameters used in calibration and validation.
The study period for the climate change predictions is 2030 – 2096 (01Jan2030 to 31Dec2096). The extraction of raw water from the upper reservoir fluctuates slightly during the year which is expected because of seasonal changes in demand. Initially, for the purpose of future operation it was taken to be 1.04 m^3/s which corresponds to the yearly average for the 2003-2009 period. Although an increase in demand can be determined from projected population growth for the future, these impacts were not factored in.
Similarly, a hypothetical time-series for extraction from the low dam was required. A correlation was found between monthly rainfall and monthly extractions from the low reservoir. A new time series was generated for the proposed low dam production for the period 2030 to 2096 by assuming that the monthly production totals repeats every year.

3.1 Future operations of the NRS.

The hydrologic model was run for each of the climate change scenarios for the period 2030 to 2096. For all three scenarios, the upper reservoir was completely depleted before the end of the simulation period which suggests that based on climate change predictions the NRS cannot supply water at its current rate of production.
To simulate a reduction in raw water extraction from the upper dam a number of raw water extraction time series were generated with each successive year being 5% lower than the previous. Following this methodology, the raw water extraction was reduced to 95% of present capacity, a rate of 0.99 m^3/s. The effect of this change was investigated for the AENWH climate scenario. At full production the reservoir was depleted before the end of the first year, 2030. At 95% production the reservoir emptied by the end of 2031. Production rates were successively reduced by 5% and the model run until a value was reached which allowed the model to run until the end of the simulation period.
The model ran to completion for a drastically reduced production rate of 0.73 m^3/s corresponding to a reduction in production by 30%. A stepped reduction in production rate was considered; however, the reservoir was still depleted over time because of the cumulative effect of a previous higher production rate. A more structured approach to reservoir management and operations may result in higher sustainable production rates, however, this was not investigated at this time.

A similar method was followed for both the AEXSA and AEXSK climate model scenarios. In both instances, the ultimate production rate was reduced to 0.63 m³/s which corresponds to production rate which is 60% of present production.

Reductions in the smaller, more frequent events for the AENWH scenario meant longer dry periods between the less frequent larger events. This can be seen in the model response in Figure 6 with a larger difference between reservoir low level and high level elevations indicating possible longer periods between rainfall events and high evaporation loss. The reservoir shows more frequent overflow events than the other two scenarios. These events also continue for longer periods when they do occur. This occurs almost yearly for the period 2059 – 2065 and can be seen as the horizontal sections of the lake level plot (blue line) in Figure 7. This volume of excess water will eventually be routed to the lower reservoir via the Navet river. The reservoir level also reached very close to it low level mark of 82 m in the years 2048, 2080, 2082 and 2096. It is also observed that these events occur during periods when there are no overflow events for a number of years prior. These observations are consistent with a reduction in the more frequent smaller rainfall events and an increase in the less frequent larger events. The AEXSA climate scenario showed the reservoir going through a very cyclic filling and emptying pattern. The range of level between the highs and lows are much smaller than those exhibited for the AENWH scenario. The reservoir under these conditions also exhibits a lower sustained supply.

This climate model showed an increase in both large and small events. The results of the simulation show that there are more frequent instances of reservoir overflow although very small in the first 40 years of the simulation however more frequent, larger overflow events after 2071. The more frequent, smaller rainfall events are sufficient to refill the reservoir at a reduced rate of production prior to 2071. However, post 2071 there are larger and more frequent overflow events indicating a higher water availability and a possible larger production rate may be supported.

The AEXSK climate scenario showed an increase in the more frequent smaller rainfall events as well as a decrease in the less frequent larger rainfall events. This scenario represents the most water scarce scenario of the three. The water level simulated by the model shows a very cyclic filling and emptying of the reservoir with a very shallow elevation range between. This together with almost non-existent overflow events indicates that the reservoir receives just enough water from small rainfall events to provide a reduced supply, 60% of present production.

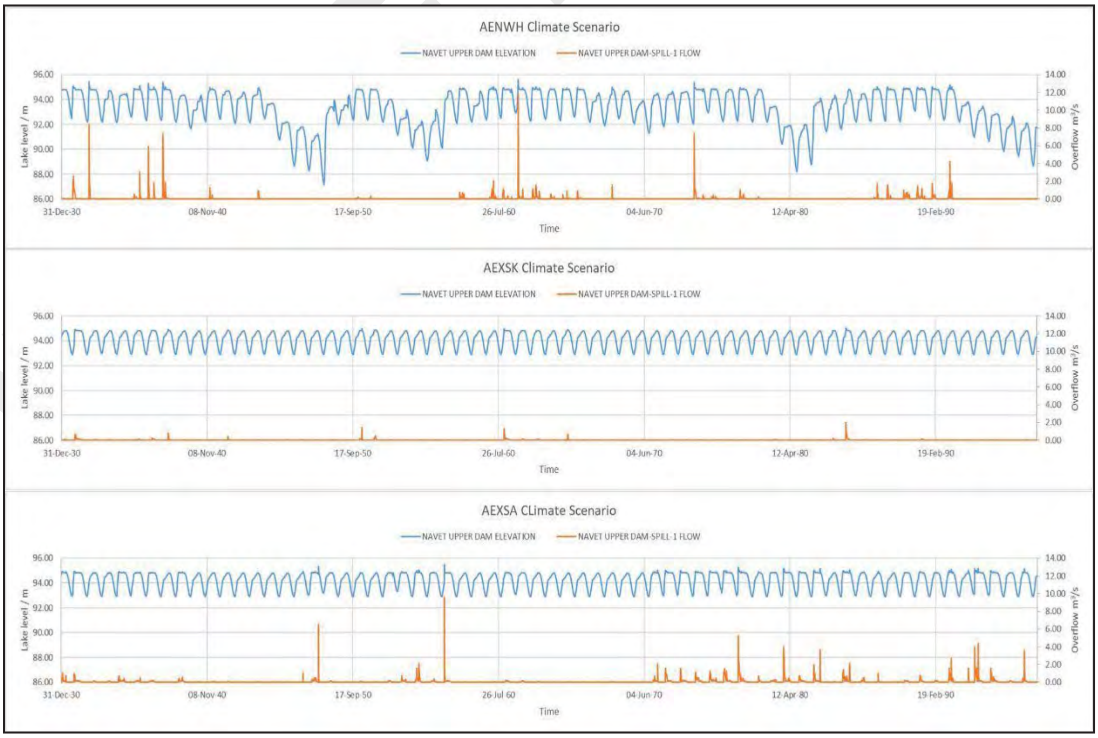


Figure 6 NRS responses to Climate Change Scenarios

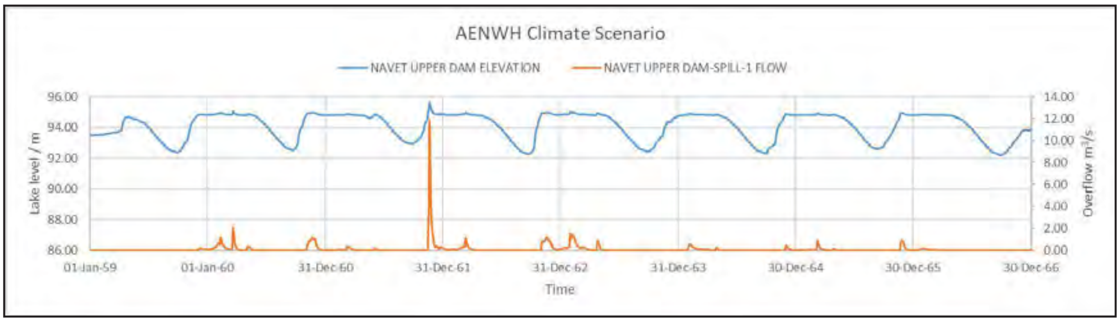


Figure 7 – AENWN overflow periods 2059-2065

4. CONCLUSION

The HEC-HMS Soil Moisture Accounting algorithm (SMA) has been shown to adequately represent the continuous movement of moisture in the Navet catchment and to allow long term simulation to be made. The simulated response of the catchment to predated rainfall and climatic conditions has shown that the model can be used to make decisions on operational changes at the dam. In the near future, based on predicted changes in climate the Navet reservoir may not be able to adequately supply the water demands of its dependent populations. This can mean additional sources of water may need to be brought online to satisfy increasing demands of a growing population. Additionally, it may signal the need for improved conservation practices, not the least of which is leak reduction. As this report has shown, the future availability of water is threatened, and present demands are unsustainable.
The capability of the model can be enhanced by the treatment of the lower reservoir and its catchment as a hydrologic system instead of a water source. This will give a complete view of how the reservoirs function to complement each other.

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Cumulative Fatigue Damage of Small-Bore Piping Subjected to Flow Induced Vibration

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ABSTRACT

The structural fatigue of a vertically oriented small-bore connection due to flow induced turbulence emanating from an upstream piping manifold is experimentally investigated. Dynamic strain measurements are taken at two perpendicular locations on a small-bore connection and the method of rain flow counting is used to determine the cumulative damage incurred. The influence of a number of factors on the cumulative damage are investigated and explained. These include the effects of (1) single phase and multiphase flow, (2) the upstream flow path through the manifold, and (3) steady-state and transient conditions. Specifically, key observations that may be useful to piping designers and engineers are observed and reported. For instance, for single phase water or air the largest bending stresses are due to the out-of-plane vibration of the small-bore piping, whereas the pulsating characteristics of the multiphase flow results in significantly larger in-plane bending stresses. It is also observed that under certain manifold outlet conditions, transient effects upon pump start-up can produce more than 300 times the cumulative fatigue damage compared to steady-steady operation.

Keywords: Cumulative damage, Fatigue, Flow induced vibration, Rainflow counting, Small bore connections.

1. INTRODUCTION

Piping networks are major assets for utility and industrial process plants in the Caribbean region and beyond. Depending on the application, piping systems can vary in geometric configuration, layout, material construction and operating conditions. Maintaining the integrity of piping systems is a key component in managing safety, business and environmental risks. Historically, it has been well documented that excessive vibration and fatigue in piping systems can lead to premature failure, often with disastrous consequences [1]. With respect to water supply and utility systems, piping fatigue can introduce cracks that result in leakage and service reduction [2]. Bradshaw et al., [3] identified 406 recorded incidents in a UK-based regional water supply system between 1997 to 2006. The second largest number of incidents were associated with material fatigue.

A major source of vibration in water and utility piping systems transporting high velocity flow streams, is flow induced turbulence. The excitation that arises from flow induced turbulence becomes significant in piping systems with geometric discontinuities such as mitre bends, 90º bends, and tee junctions [4-5]. Small bore connections (SBCs) are branched connections that protrude off the mainline piping. The nominal diameter of SBCs are typically 2 inches (50.8 mm) or less, and they are particularly prone to vibration induced fatigue failure [6].

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A common approach to designing SBCs is to use the screening procedure proposed by the Energy Institute's (EI) Guidelines for the Avoidance of Vibration Induced Fatigue Failure in Process Pipework [7]. The procedure determines a likelihood of failure, giving ranges of values for which SBC passes or fails the screening. If the initial design fails the screening process, the guidelines provide the analyst with two options; (1) redesign or resupport the SBC or (2) conduct detailed experimental or finite element based assessments. Although the EI guidelines [7] does not propose a finite element procedure, it does provide a vibration chart that allows analysts to use measured vibration levels to determine if the probability of failure is high.

Several other researchers have expanded on the EI Guidelines [7] and proposed both experimental and numerical methods to analyse the vibrations and fatigue of SBCs. Hamblin [8] proposed an empirical method to determine the dynamic bending stresses of cantilevered small bore connections. Application of the method was illustrated using laboratory and onsite field testing. Xue et al., [9] used a hybrid empirical and numerical approach to determine the cumulative damage of a simply supported small bore piping connection in a nuclear power plant. Moussou [10] proposed a root velocity experimental criterion to evaluate the risk of vibration-induced fatigue of small bore pipes.

In this paper we investigate the cumulative fatigue damage incurred by a small bore cantilevered connection located downstream a piping manifold. The problem is important since piping manifolds can generate large flow disturbances [11], which can result in downstream SBCs being susceptible to vibration induced fatigue. An experimental test rig that includes a manifold and a downstream SBC is used to perform the analysis. Dynamic strain measurements are taken for single phase air flow, single phase water flow and a multiphase mixture of air and water. A rainflow cycle counting algorithm is then used to determine the cumulative damage for a set period of time. The effect of transient (start-up) conditions compared to steady state operation is considered and discussed. The piping manifold used in this study, divides the inlet flow into three separate legs. By using ball valves to control the flow through the manifold, the influence of the flow path on the cumulative damage accumulated by the SBC is also investigated. Designers and engineers can use the methodology and results presented in this work to reduce the potential of failure of small bore connections due to flow induced fatigue.

2. DESCRIPTION OF THE PIPING TEST RIG

Consider the schematic of the test rig shown in Fig. 1. A 600-gallon polyethylene tank (A) situated on top of a metal frame (C) feeds the single impeller Aurora 1070 centrifugal pump (B). A GPI TM-300F turbine metre (D) measures the volumetric flowrate whilst a globe valve (G) is used to manually throttle the flow. Multiple piping supports (such as EF in Fig. 1) are placed at different points along the length of the piping system. A Kellog 452TV compressor located far upstream the mainline pipe, supplies pressurized air through a branched connection (H). The inlet air-pressure and flowrate are measured using a Wika pressure gauge and Hedland H771A-150 air flowmeter respectively.

Fig 1. Components of the experimental piping system

Downstream the air inlet (H), a piping manifold (I) is introduced into the system. The manifold consists of one inlet and three outlets as depicted in Fig. 2. The inlet of the manifold has an inner diameter and thickness of 77.93 mm and 5.49 mm respectively, which is similar to the mainline piping upstream of the manifold. The ends of the manifold are closed using two hemispherical caps, each having a nominal diameter of 76.2 mm. The caps are designed in accordance with ASME B16.9 [12]. The outlet of the manifold splits into three separate channels, each having an internal diameter of 52.5 mm and thickness 3.91 mm. Threaded ball valves (BV1-BV3 shown in Fig. 2) are installed to control the flow in each of the three legs of the manifold. Flow travelling through the legs of the manifold merges through a cross tee-fitting (O in Fig. 2) and subsequently returns to tank A via the piping span KL (Fig. 1). Additional details of the general features of the piping test rig may be found in Ref. [11].

Fig. 2. Manifold and ball valve configuration

The small bore connection (SBC) that will be investigated in this work is located directly downstream 'O' as shown in (Fig. 2 and Fig. 3). The SBC has a ball valve connected at its free end and is placed vertically upright. The outer diameter and thickness of the SBC are 33.40 mm and 1.65 mm respectively. A 3000-class threadolet having a nominal diameter of 25.4 mm is used to connect the SBC to the mainline piping. The mainline piping on which the SBC is connected has an inner diameter and thickness of 52.55 mm and 1.65 mm respectively. The lengthwise dimensions of the SBC inclusive of the threadolet and valve are also shown in Fig. 3. The pipe material for the SBC connection is cold drawn galvanized steel.

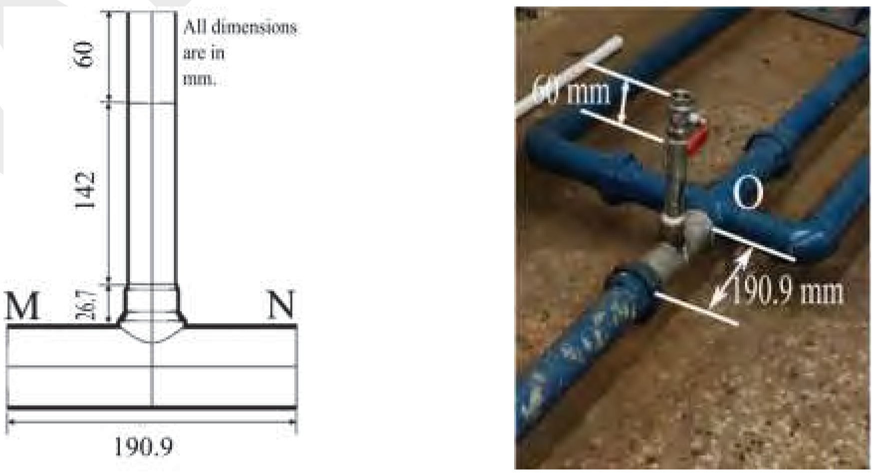

Fig. 3. Sketch and location of vertical small bore connection

2.1 Experimental methodology

Axial strain measurements were taken at the base of the SBC using a PCB RHM240403 dynamic strain gauge having a sensitivity of 10 mV/μE. Strain measurements were taken at two perpendicular positions (Location1 and Location 2) as shown in Fig. 4 for both single phase and multiphase fluid flow. Location 1 measures the out-of-plane strain whilst Location 2 measures the in-plane strain. The strain gauge was connected to a laboratory charge amplifier and data acquisition unit (Kistler Type 5165A4). A sample rate of 62500 Hz was selected and a low pass antialiasing filter with a cut-off frequency of 10 KHz was utilized. For each individual measurement taken; data was collected for a total of 60 seconds. It was observed that a sample time of 60 seconds consistently led to a well-defined asymptotic mean square strain value.

Fig. 4. Position of the strain gauge on the SBC; Location 1 (left) & Location 2 (right).

The first set of experimental measurements were conducted for single phase water (that is the air inlet was closed and the compressor taken offline). Both transient and steady state conditions are considered. In the transient case, the effects of pump start-up are included whilst in the steady state case, measurements were only taken after the transient effects became negligible. Consequently, steady state measurements were only taken after the pump was operational for at least five minutes after start-up. The strain produced by the fluctuating excitation forces were recorded for the following six ball valve combinations shown in Table 1.

Table 1: Ball valve configurations and flow path control

Ball valve configuration	Valve settings
1	BV1 opened, BV2 opened, BV3 opened
2	BV1 closed, BV2 opened, BV3 opened
3	BV1 closed, BV2 closed, BV3 opened
4	BV1 opened, BV2 closed, BV3 closed
5	BV1 closed, BV2 opened, BV3 closed
6	BV1 opened, BV2 closed, BV3 opened

The measured strain data was subsequently converted to dynamic stress and a rainflow cycle counting algorithm was used to determine the cumulative damage that occurred over the 60 second period. The rainflow counting algorithm, which adheres to ASTME 1049-85 [13], was executed in Matlab 2022a [14]. The second and third set of experiments were conducted by repeating the above procedure for single phase air and a multiphase combination of air and water.

3. RESULTS AND DISCUSSION

3.1 Single phase water and single-phase air

In this study, the manifold inlet velocity for single phase water is 1.94 m/s, which corresponds to a Reynolds number of 163000, thereby indicating turbulent flow. For Location 1 of the strain gauge, the average cumulative damage for all ball valve combinations at transient and steady state conditions are shown in Fig. 5. In Fig. 6, similar data is presented for Location 2.

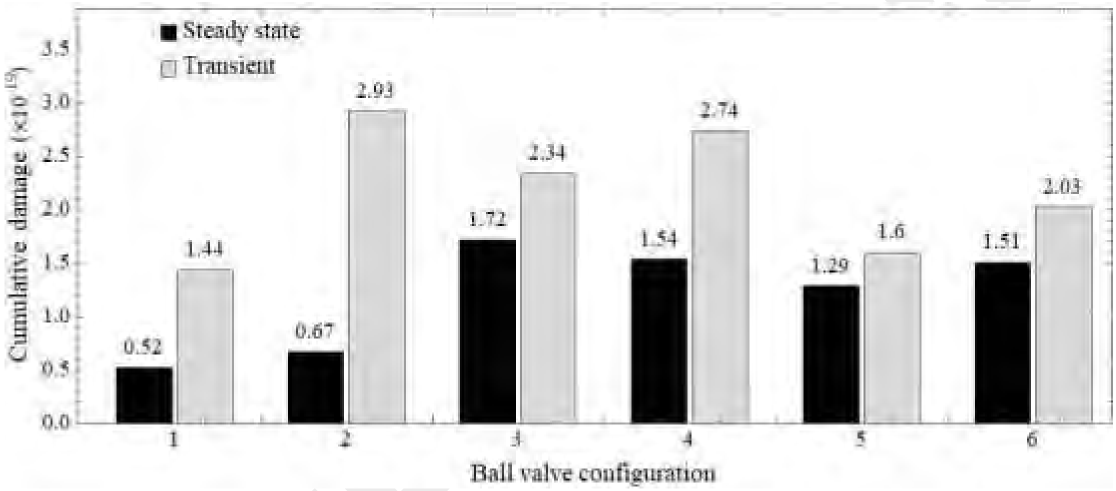

Fig. 5. Cumulative damage of the SBC at Location 1 for single phase water

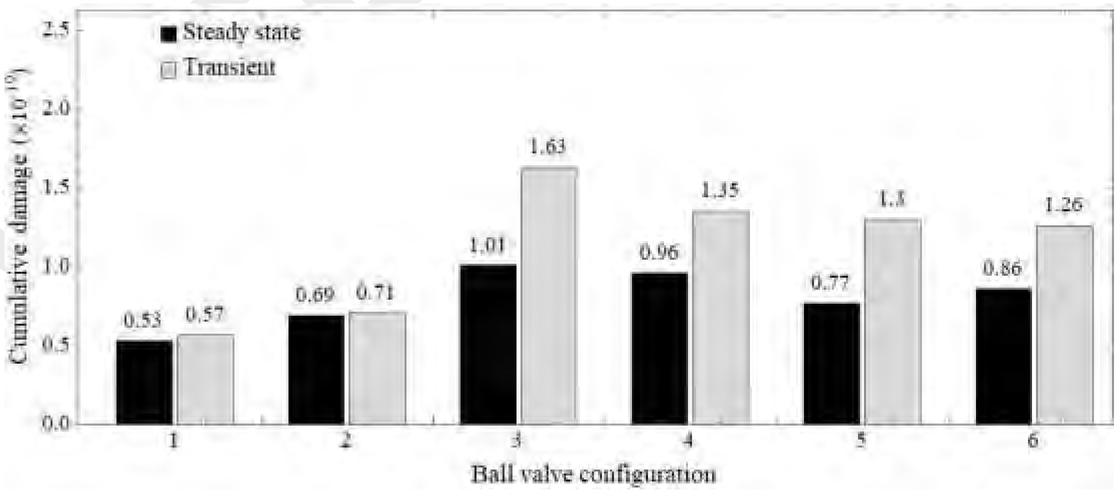

Fig. 6. Cumulative damage of the SBC at Location 2 for single phase water

It can be observed from Figs. 5 and 6 that Location 1 had a larger cumulative fatigue damage compared to Location 2. For steady state and transient conditions, it is shown that certain Ball-Valve-Configurations can lead to greater fatigue damage compared to others. It is also observed that the transient effects of pump start-up results in the cumulative damage being significantly larger than the steady state conditions. In Ball Valve Configurations 1 and 2, the transient cumulative damage is 177% and 337% greater compared to the steady state damage. In general, when all three ball valves are opened, the cumulative damage of the SBC is the smallest. Closing either one or two of the valves result in a marked increase in cumulative fatigue damage to the SBC. This can be explained by considering that there is an increase in the amplitude of the underlying excitation pressure as the already turbulent flow is restricted to pass through only certain valves at a higher speed. Interestingly, the combinations where BV2 is closed and flow passes through BV1 and/or BV3, a higher cumulative damage is generally observed compared to when BV2 is open. This can be explained due flow separation and turbulence associated with the 90º bends located upstream of the BV1 and BV3 valves. The mechanisms accompanying turbulent flow around a bend are shown in Fig. 7. The turbulent boundary layer between the fluid and the inner piping walls produces a strong pressure gradient. Due to the random nature of the flow, this encourages wall pressure fluctuations to develop producing high levels of localized kinetic energy which causes the piping to vibrate. Transverse plane waves propagating throughout the piping system can also be present, although the induced vibration is restricted to low frequencies [15].

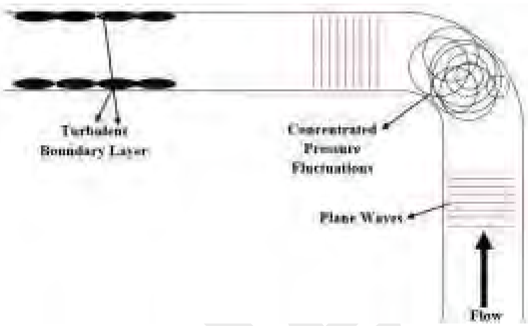

Fig. 7. Mechanisms producing wall pressure fluctuation from flow induced vibration

Single phase air was allowed to flow through the system with an inlet velocity of 3m/s, and a Reynold's number of 15844. The cumulative damage for all valve combinations at steady state and transient conditions are shown in Figs. 8 and 9 for Location 1 and Location 2 respectively. It can be observed that across both plots, the same trend present for single phase water is also seen for single phase air. The results emphasize the influence that both start-up conditions and flow path have on the cumulative damage induced by single phase flow.

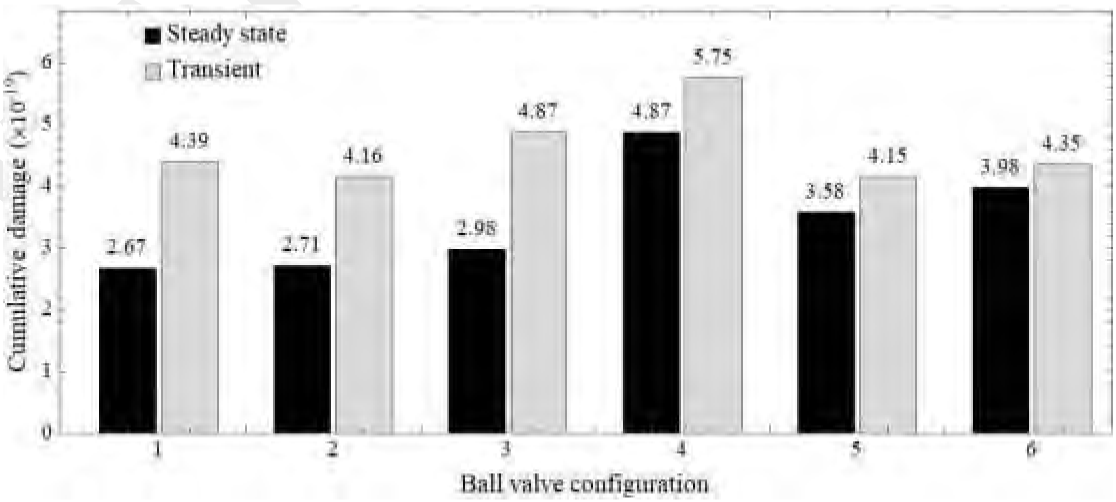

Fig. 8. Cumulative damage of the SBC at Location 1 for single phase air flow

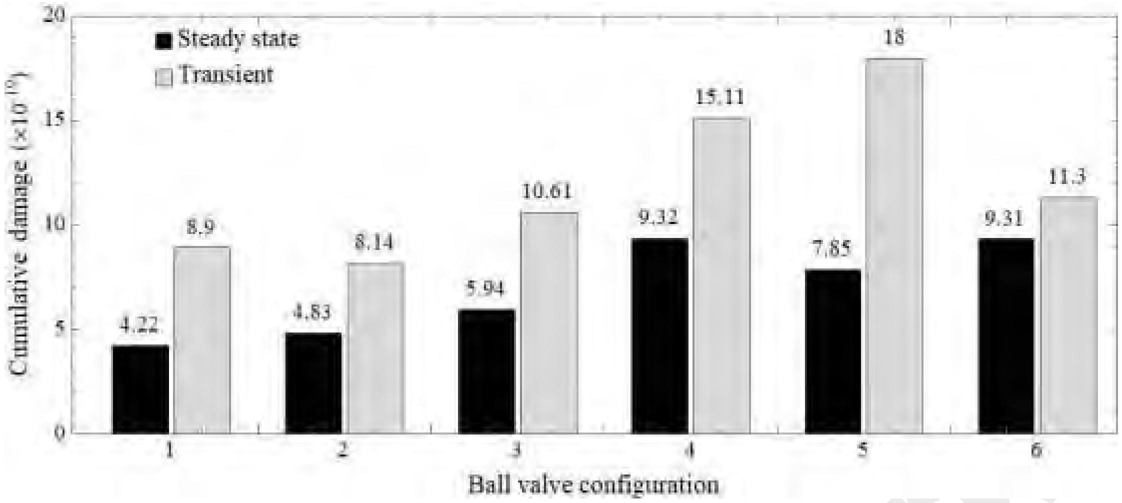

Fig. 9. Cumulative damage of the SBC at Location 2 for single phase air flow

3.2 Multiphase air-water flow

In the experimental study, the multi-phase air-water mixture had a manifold inlet velocity of 0.99 m/s for water and 3 m/s for air. The cumulative damage for all valve combinations at steady state and transient conditions are shown in Figs. 10 and 11 for Location 1 and Location 2 respectively. Two key differences are observed upon comparing the results with single phase flow. The first is that the amount of cumulative damage in the 60 second period is significantly greater across all ball valve configurations, despite the flow rate of the water being less compared to the single phase case. The second major difference is that the strain gauge at Location 2 records higher strain response levels compared to Location 1. These changes are due to the pulsations associated with the alternating periods of air followed by water. The pulsations cause the underlying pressure fluctuations at pipe walls to have larger magnitudes and therefore induces greater vibration and stress levels. Additionally, the pulsations occur along the longitudinal axis of the mainline piping. It follows this excitation induces larger in-plane deflections of the SBC (Location 2) compared to the out-of-plane motion (Location 1).

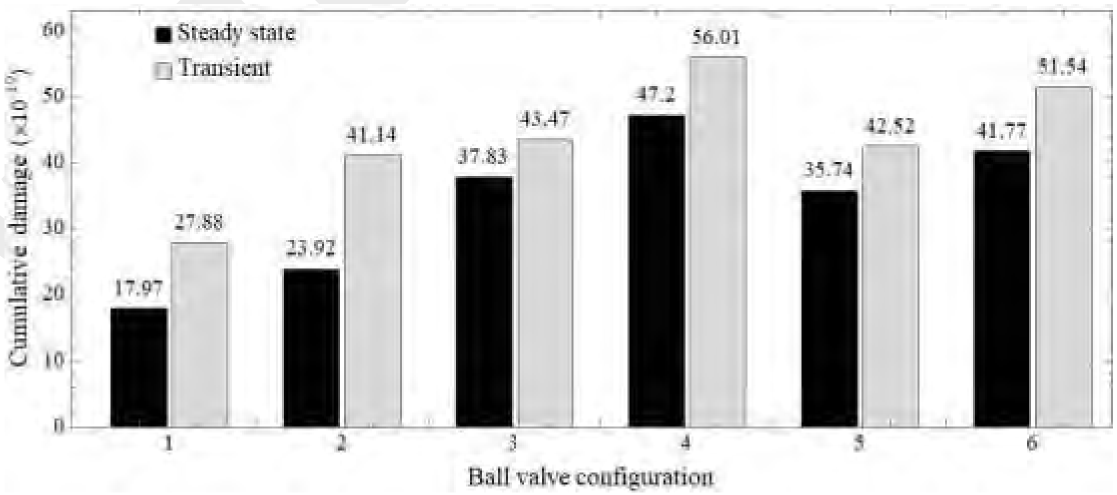

Fig. 10. Cumulative damage of the SBC at Location 1 for multiphase air/water flow

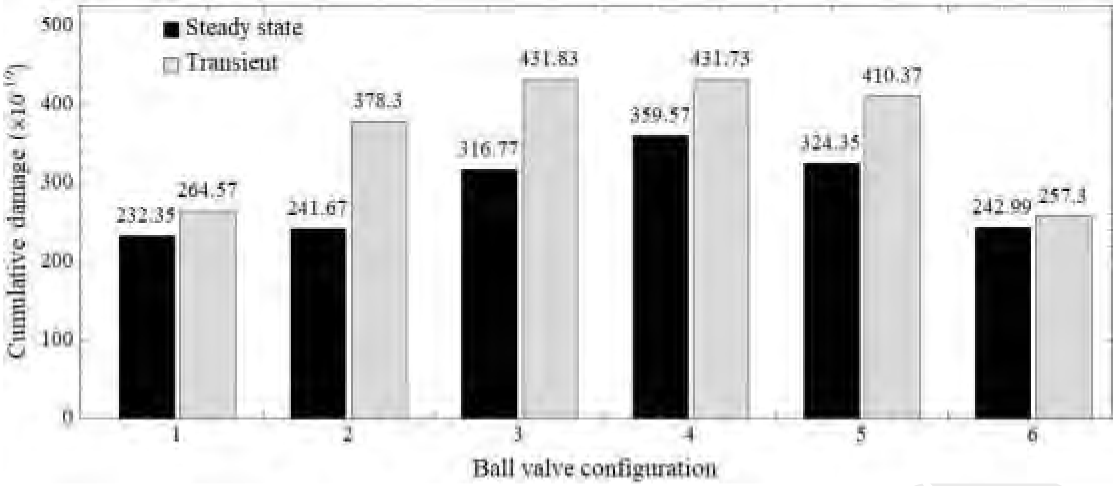

Fig. 11. Cumulative damage of the SBC at Location 2 for multiphase air/water flow

4. CONCLUSIONS

In-plane and out of plane dynamic strain measurements are taken on a small bore connection located downstream a complex piping manifold. The method of rainflow counting is applied to determine the cumulative damage for both single phase and multiphase flow. In the case of single-phase flow, it is observed that the in-plane bending stresses are largest whilst for multiphase flow the out-of-plane bending stresses dominate. The flow path through the manifold is controlled by manipulating the on and off position of ball-valves located at the outlet. It is shown that the flow path can significantly influence the magnitude of the cumulative damage. The effect of transient start-up conditions compared to steady state operating conditions are also investigated. It is observed that the latter can result in significantly larger amounts of cumulative damage.

COMPETING INTERESTS

The authors declare that they have no competing financial interests or relationships that could have appeared to influence the work reported in this paper.

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Improving Monetary Valuation Methods Used in Cost Benefit Analysis Of Water Infrastructure Projects

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ABSTRACT

An assessment of social utility and project benefits in the water sector should include financial, environmental and socioeconomic impacts that are comparable over the same unit in an analysis. Cost benefit analysis (CBA) is a decision support tool that can cover impacts identifiable within these three impact categories. However, most water sector CBA are only invested in forecasting future financial cost-benefits, while environmental and socioeconomic impacts are assessed by environmental impact assessments (EIA), life cycle assessments (LCA) and other qualitative assessments, or entirely ignored. Studies identify a need for more guidance on application of relevant monetary valuation methods to water infrastructure specific project impacts. This is feasible if a more approachable and implementable CBA framework tailored for the sector is made available. This paper suggests a universal set of umbrella categories for wastewater treatment plant (WWTP) impacts across the five project phases that comprise a project lifespan. The paper identifies literature on accessible and relevant monetary valuation methods for each impact category typical to a WWTP infrastructure project. The findings originate from literature on current practices in the water sector, academic innovations theoretically applicable to water sector projects, and methods borrowable from comparable sectors. These origins are the building blocks for consolidating knowledge of monetary valuation methods relevant to the water sector, which we tabulate in a reference matrix.

Keywords: CBA, WWTP, roadmap, monetary valuation methods

1. INTRODUCTION

Water sector infrastructure projects affect the livelihood of local, regional and global citizens and their surrounding living environments. In this context, there is a growing need to identify and explain such impacts to project stakeholders in a meaningful and comparable way [1,2]. These impacts should be presented to laymen and decision makers in a language most meaningful to them, and readable without expertise knowledge. Most stakeholders can personally relate to money and thus understand monetary valued impacts. In contrast, some stakeholders, especially the decision makers, may find qualitative and physical unit valuations of impacts more unfamiliar, abstract or of secondary importance to monetary cost-benefit discussions. Money is arguably a common language for all stakeholders, even though it can at times be a crude measure of project impacts. Indeed, monetary valuation of all impacts from the perspective of the developer, financier, end-user, the environment and other stakeholders has not always been possible.Despite the existence of many informative guidelines and requirements for comprehensive execution of cost benefit analysis (CBA), their practice is limited in reality, especially in monetary evaluation of socio-economic and environmental impacts (Ratnaweera et al 2020). Developments over decades have resulted in robust monetary valuation tools that are embraced to a varying degree by a range of public service sectors, funding bodies [3], and

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awareness campaigns [4]. Yet, the water sector is not at the forefront of using such tools, perhaps over financial barriers relating to sourcing data and executing impact assessments. One reason is that various project impacts are often expressed in scientific terms and languages that are either dissimilar and incomparable, or not expressed at all, when rigorous impact assessments are not performed as a result of limited resources [1]. The driving factors behind this reason seem to be that many water sector projects have limited resources; sourcing data and executing impact assessments can be financially costly and could require specialist competence and significant work-hours beyond the project's scope. Therefore, the water sector suffers from a gap between existing impact analysis frameworks and the incomplete ways these frameworks are implemented in practice.

Ratnaweera et al. (2020) outlines how this is not necessarily the case across public sectors and their infrastructures, and that this divergence in ability and methodology to present infrastructure benefits and social utility can create an unfair competition in the public funding arena. Ratnaweera highlights how valuation methods applied in other European or North American public sectors, such as roads and transport, express environmental and socioeconomic impacts in a unified, comparable language of monetary units, and contrasts with observed practices in the water sector. The need and potential for an impact assessment and monetary valuation roadmap for the water sector is evident; This paper presents a road map by i) evaluating the three most widely used assessment methods in the water sector and how they can interact, ii) valuation methods which largely exist in water academia, and iii) valuation methods borrowable from other sectors.

2. METHODOLOGY

Current practice of CBA in the water sector was reviewed by Ratnaweera (2020). This paper builds an extension of that review to include all dominant impact assessment tools in the water sector, namely CBA, life cycle analysis (LCA) and environmental impact analysis (EIA) to underpin a water-sector specific CBA roadmap. This paper focuses on valuation methods for wastewater treatment plant (WWTP) infrastructure projects as a case, but places emphasis on designing a roadmap applicable for all water infrastructure projects. The impact factors will be encompassing and have umbrella functions, under which sub-categories for impacts may be developed in the future. The roadmap will identify the range of impact assessments in the water sector, and their practical applicability to incorporate the theoretic needs for valuation methods in these widely used tools.

Current practices in water sector impact assessments and practically available valuation methods are evaluated in this paper. We explore the potential to what extent the CBA framework could be made more accessible by identifying widely established assessment and valuation practices in the wastewater sector, and conceptual proposals in the water sector in theoretical or pilot stages. When no valuation methods are found for impacts, a knowledge gap is identified and the paper highlights potential practices which are adaptable from other sectors. Building on these findings, a reference matrix and flowchart framework as a roadmap for a comprehensive CBA applicable to the water sector is developed. Figure 1 visualises this process and outlines the structure of this study.

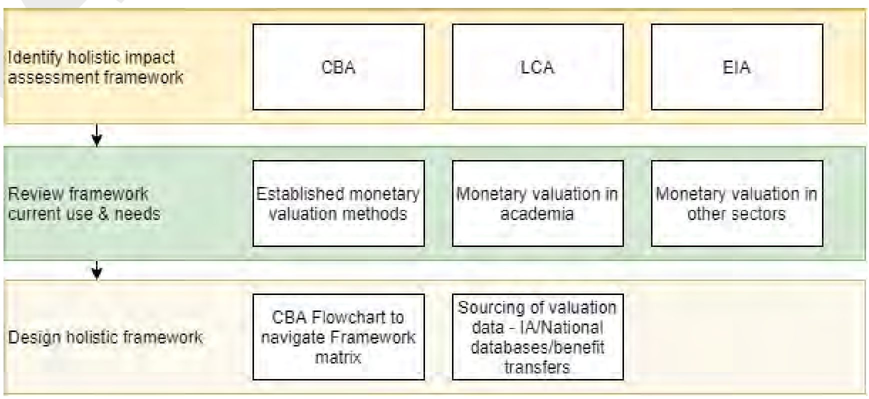

Figure 1: Analytical framework to identify a more practically applicable and comprehensive CBA decision support tool for use in the wastewater sector.

Any impact from water projects is the result of a physical infrastructure development, and typical structures include pipelines from water sources, drinking water treatment plants (DWTP),

storage tanks, wastewater treatment plants (WWTP), distribution and collection pipeline network to and from domestic and industrial users, pumping stations associated with the pipelines, sludge management and waste disposal, nutrient and energy recovery processes, and the network to return treated water to a water body[5]. The context of building these infrastructures vary across geography and water sources, with variations in population densities, surrounding land use elevations, climate, discharge water body type etc. Strategic planning, political will, financing capabilities and technological innovations also drive the variety of infrastructures that can be built. In this study we focus on exploring cost-benefits of impacts associated with wastewater infrastructures – e.g. WWTP in centralized to semi- and fully decentralized design; in open, tunnelled, secluded or submerges landscapes; with mechanical, chemical and/or biological treatment stages; using conventional, infiltration or nature-based solutions; with polishing and recovery processes embedded etc.
All wastewater infrastructures carry a social utility, i.e. the benefit to society as a whole, for those who consume the water and wastewater services, use the source and discharge water bodies or live along the path of the various water infrastructures [6]. Therefore, it is critical to account for the total impacts of these infrastructures over time at a relevant scope, to help identify projects and solutions that best serve society.

CBA, LCA and EIA are decision-support tools (Skovgaard et al., 2007, Ratnaweera et al., 2020) designed to aid decision makers assess impacts resulting from e.g. WWTP infrastructure projects to their local and global community. These are the stand-out assessment tools in a field of qualitative and quantitative impact assessment tools in the water sector [8], because of their wide use and quantitative nature. Other less commonly used tools such as multi-criteria decision analysis (MCDA), cost-effective analysis (CEA), strengths-weaknesses-opportunities-threats analysis (SWOT) and many other impact assessments have been omitted to fully explore the building of an accessible impact assessment framework based on the most widely used analyses in the water sector.
The three tools can be defined and distinguished as follows, when viewed in the context of water infrastructure projects or investments:
CBA is a decision-support tool that helps identify the best investment decision which maximizes utility to society at a national spatial boundary [9]. This tool compares and discounts impact cost-benefits into the future or a project's lifetime. CBA is a tool that exclusively expresses all values in money, where the aim is to identify the project with highest net present value – an indicator the scenario with highest social utility [10]. In theory, CBA cover financial, environmental and socioeconomic impacts of a project. In practice in the water sector, many socioeconomic and environmental impacts are not included, due to lack of knowledge on how to apply monetary valuations to such impacts, or the lack of time to collect data and perform these valuations (Ratnaweera 2020). Relevant monetary valuation methods can be found by approaching project impacts through the lens of the total economic value framework [11], such as revealed preference (hedonic pricing, travel cost method etc.) and stated preference (contingent valuation, choice experiment etc.) [10]. Although sensitivity analyses of net present values in a CBA are recommended, they are often de-prioritised in already sub-standard CBA reports, as the sensitivity analysis is not a required step [12].
EIA is a decision support tool that assesses the environmental and socioeconomic impacts of a project, at a local spatial boundary [13]. An EIA qualitatively and quantitatively document environmental impacts, from the perspective of environmental utility [14]. The quantitative impacts are expressed in physical units, and the impacts are typically screened for a project's lifespan. EIA normally do not address financial aspects of project impacts [13].
LCA is a decision support tool that strictly assesses environmental impacts of processes or products of a project over its entire lifetime, from cradle to grave [15]. The LCA operates with planetary spatial boundaries, requires a sensitivity analysis and expresses process impacts in physical units such as mass, volume, energy etc.; from raw materials extraction, production and transport and operation, to recycling or decommissioning [16]. In the water sector, the aim of the LCA is to quantitatively document and improve the environmental utility of a given water infrastructure, with physical units instead of monetary valuation.

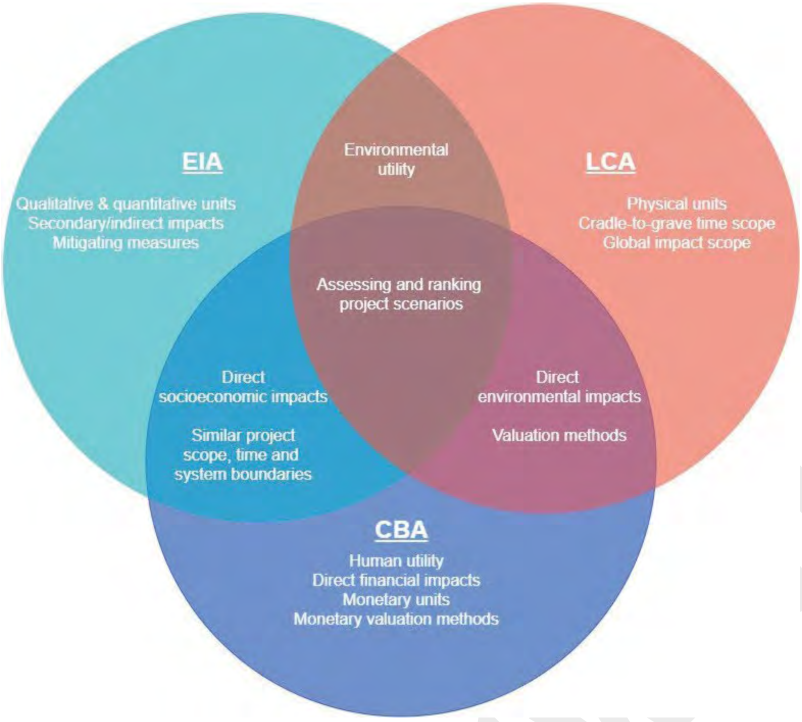

Figure 2: Venn diagram of overlapping and distinct features of impact assessment tools EIA, LCA and CBA

Figure 2 highlights the similarities (features listed in overlapping sections) and differences (features listed in separate sectors) between the three tools. .There are several variations to each of these methods that are extensively described in academia, particularly for LCA. Hoogmartens et al. (2014) detail the many different established and novel sub-categories of LCA (e.g. life cycle cost (LCC), e/s-LCA of LCA) that can bridge social and financial impacts into a global, cradle-to-grave LCA perspective. However, while several steps are made to make LCA a comprehensive evaluation tool, including the development of the new ISO standards, the whole-life-cost (WLC) framework outlined in ISO 15686-1 (ISO 2004) illustrates both the traditional restrictions of an LCA and its LCC cousin. The comparison of LCC in the WLC framework helps identify the theoretical advantages a CBA has over this otherwise widely implemented assessment tool, since a correctly practiced CBA covers the WLC framework.

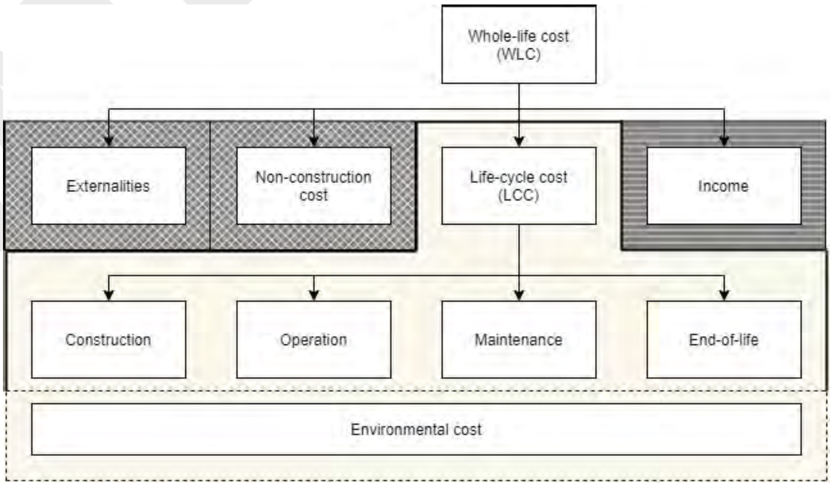

Figure 3: WLC framework combined with grey sections to indicate Hoogmartens' LCA sub-categories.

The grey sections refer to areas outside the traditional LCA and highlight the problem of dependency on sub-categories outside the original LCA's scope; criss-cross areas link WLC branches to s-LCC, and horizontal areas link WLC branches to f-LCC.

However, the LCA approach is both global in scope and cost-only focused. Figure 3 illustrates the WLC framework and the limited coverage of that framework by life cycle costing (LCC), the monetary valuation version of an LCA. While a comprehensive CBA covers the gamut of financial, environmental and socioeconomic impacts at a local project scope level, figure 3 underlines how the LCA concept ignores externalities such as socioeconomic impacts and impact benefits in a project. Hoogmartens' LCA categories are embedded into the WLC framework with textured grey boxes to illustrate how financial-LCC (f-LCC, horizontally lined box) and social-LCC (s-LCC, criss-cross boxes) are additional decision support tools to help monetarily value an LCA according to the WLC framework. Hoogmartens highlights the challenges in aligning these analyses, the differences in global and local scope being a main challenge [17]. In comparison, the CBA is both benefit-cost oriented and focused on valuing all impacts in a uniform monetary unit and scope, unlike the variety of physical units found in traditional LCA or the sub-categories of LCC with differing underpinning scope .

While CBA, LCA and EIA provide insights across financial, socioeconomic and/or environmental impacts, their system boundaries and impact valuation units can differ. Among the three tools, CBA is the only one designed to collect and compare project impacts across financial, socioeconomic and environmental factors. However, WWTP CBA often only include impacts already expressed in monetary units, such as financial impacts (Gillot et al., 1999). In contrast, most typical water sector LCA and EIA describe socioeconomic and environmental impacts in physical units or qualitative evaluations. Furthermore, most financing institutes (Ward et al., 2019), and even the U.S. government (National Archives Executive Order, 1981), have incorporated CBA as a required component in significant disbursement activities, where the EU Cohesion Policy is an example with its guidelines for executing mandatory CBA for any projects seeking funding that cost over €30 million (Sartori et al., 2014). The water sector has an established practice of requiring CBA, but reports indicate the execution is poor in terms of socioeconomic and environmental impact valuation. Therefore, we focus on developing more a tailored, accessible roadmap for a comprehensive CBA for the water sector, where EIA and LCA data are integrated as data drivers.
Research highlights how the water sector underutilises monetary valuation methods designed to convert socioeconomic and environmental impacts to monetary units (María Molinos-Senante et al., 2010). Table 1 and Corominas et al. (2013) underline how EIA and LCA are more widely used assessment methods for socioeconomic and environmental impacts of water projects where impacts are expressed in physical units. Therefore, we focus on developing more a tailored, accessible and comprehensive CBA for the water sector, where the more widely practiced assessment formats EIA and LCA are integrated as data drivers for a more comprehensive CBA. We present i) widely established practices for CBA ii) conceptual proposals which are still theoretical or at pilot stage and iii) knowledge gaps, where other public sectors have come further in applying valuation methods to socioeconomic and environmental impacts. The above analysis is contextualised with WWTP impacts in mind. The latter two domains are areas where more widely collected EIA and LCA data could provide additional data for monetary valuation and use in a comprehensive CBA.

Table 1: Current use of monetary valuations among water sector CBA, and potential contributions from other decision-support tools and valuation concepts to a comprehensive CBA methodology.

Monetary valuation of project impacts	Financial	Environmental	Socioeconomic
CBA	High	Low use, high potential	Low use, high potential
LCA		High data availability	
EIA		Medium data availability	Medium data availability

Secondary data sources	Low data availability	Low data availability	High data availability

The table interprets impact assessment practitioners' experiences with, and observation of, the practical use of CBA, LCA and EIA as decision support tools over decades, alongside the use of secondary data sources in those tools, as reported to various government and financing institutes such as the World Bank, Asian Development Bank and European Commission [5,13,18], These review reports discuss the status-quo use rate of the three decision support tools in a wide body of public sector projects funded by various financing institutes. Of projects relevant to the water sector, financial impacts have the highest rating and rank as the most widely and rigorously assessed aspects of projects that submitted CBA. Meanwhile, both environmental and socioeconomic aspects of CBA straggle behind, reiterating the earlier discussion on low use of monetary valuation for such impacts. The data availability rate from the other decision support tools and secondary data use indicate from what areas CBA should retrieve driving data to improve the ease with which projects assess environmental and socioeconomic cost-benefits in water projects. Secondary data is used for impacts where monetary valuation methods extrapolate and estimate site impact cost-benefits by adjust source data to site data. The frequency of this methodology is low in water sector CBA, where the use of CBA to its fullest extent is low already. The trade-offs of using secondary data are many, with coarser and more aggregate impact analysis that depend on assumptions and methodological trust from the reader also leading to data collection cost and time saved. The use of secondary data from e.g. willingness to pay repositories, in combination with site location coefficients, leads to these savings in water sector projects. The table illustrates the availability of water specific survey data for environmental (low availability) and socioeconomic (high availability) in repositories such as Environmental Valuation Reference Inventory (EVRI), and the low availability of financial data in form of reference values in academic papers. EIA provide qualitative and physical quantifications of project impacts at similar scopes to CBA, and on aspects relating to social and environmental utility. IN contrast, water sector LCA provide physical quantifications of project impacts at global scopes, on aspects only relation to environmental utility. The LCA is relatively highly available since the tool is widely practiced with physical units that can be transferred to monetary units with valuation methods, while the practice of quantified EIA is relatively at a medium level at the same study site as where the CBA is being practiced. The key to using data from these two tools is to select data that is within the same scope as the CBA, and avoids forms of double counting.

3. RESULTS

Above we identity the need to increase the rate of use of a more comprehensive and applicable CBA in water sector projects, particularly the need to evaluate socioeconomic and environmental impacts. This need is further exasperated by CBA requirements from funding bodies, and reports of high rates of incorrect CBA use in infrastructure projects. Ultimately, a lack of practical applicability of impact assessments such as CBA result carry significant consequences; non-financial project cost-benefits that affect stakeholders of projects go unassessed or monetarily unaddressed; and decision makers are left without socioeconomic and environmental cost-benefit insight they should be afforded. As pointed out earlier, this lack of practicing comprehensive CBA in various water projects places the entire water sector at a disadvantage when competing for limited public funding vis-à-vis other sectors that include socioeconomic and environmental costs more effectively and progressively in their CBA. As outlined in figure 1, the following section outlines the current use and knowledge gaps of CBA in the water sector as applicable to impacts typically associated with WWTP infrastructure projects. The results present findings from study of CBA performed in European and North American government and/or financing institute funded projects for WWTP infrastructure of small to large scale, conventional to nature-based and decentralised options. The findings are discussed below.

3.1 Established and potential practices:

The typically financially focused CBA applies discount rates and accounts for net present values on investment costs, capital expenses (CAPEX) and operational expenses (OPEX) of projects across five common stages: pre-construction, construction, operation, maintenance and decommissioning [19]. To illustrate this in a water sector context, we use the example of an infrastructure project establishing a WWTP. Here, construction, relocation, expansion, rehabilitation or improvement may be commissioned due to a variety of socioeconomic and environmental reasons such as legislative requirements [20], demographic or industrial drivers [21] and technological advancements [22–24]. Gillot et al. (1999) highlight the typical impacts of physical infrastructures that address the above drivers in a WWTP cost-optimizing study; financial impacts due to costs for machine and process equipment, buildings, explosions, drilling, ditches and tunnelling, fuel and energy, electrical installations, plumbing, automation, design and execution, mass storage and handling, polymer use, waste disposal, land acquisition, exploratory studies, legislative and insurance costs, alongside income from user tariffs and subsidies. Financial costs are typically included in WWTP CBA and include tax, as the inherent impacts from return of taxes to social utility may be beyond the scope of the project's CBA. However, e.g. in EU Commission CBA certain taxes are omitted to avoid complexities linked to variation in tax law across members states [9].

Other widely covered costs include legislative penalties on governing bodies of infrastructures if particulate matter pollution emission limits are surpassed. These penalties may not reflect the total cost of these emissions to the environmental, human health, cultural and recreational quality and access, e.g. EU member states "…shall determine the penalties applicable to breaches of the national provisions adopted pursuant to this Directive. The penalties shall be effective, proportionate and dissuasive."[26]. While the pollutant list is not exhaustive, these listed pollutants are widely given monetary valuations in CBAs which other pollutants can follow. However, as the legislations primarily aim to deter harm to human health by financial penalties, the monetary valuations of said penalties may not reflect the actual avoided harm to human health and biodiversity.

Some OPEX costs specifically occur in construction and maintenance phases, such as costs pertaining to fuel and emission from transport of equipment and masses, re-directing of traffic and creation of alternate transport routes [27]. In the decommissioning phase, a rest value of the physical infrastructures is also discounted for in a typical water CBA [9].

Chemicals and energy use are two other factors often considered in OPEX. Chemicals are used in water supply, wastewater treatment and sludge management for coagulation, pH adjustment, micro-nutrients, carbon-sources and other process enhancements. The consumption rates and purchasing costs data for both chemicals and energy use of machines and processing units are available and vary with water volumes, specific influent parameters, or an overarching treatment method [28].

Water sector consumes about 4% of the world energy output, while country averages vary between 3% to 9% (IEA, 2016, Ak et al.v 2017). Energy is one of the larger financial components in water sector projects. CBA for energy costs are widely practiced in the water sector, where energy units used by machinery, processes, pumps and buildings are accounted for and multiplied with energy prices adjusted for influxes into the future with discount factors. Since avoided costs are considered benefits in the CBA perspective [29], energy reducing measures are typically highlighted in the water sector; when traditional CBA system boundaries limits the analysis to in-house energy consumption, a typical comparison is in-house use of energy for relatively energy efficient chemical treatment processes or energy demanding biological treatment processes.

The profit potential in harnessing biogas from WWTP to save external energy purchases and resale to the national grid from WWTP (BioCycle, 2012) are also typically highlighted. Energy recovering processes are also typically accounted for, e.g. heat recovery from tanks and pipelines transporting relatively high temperature wastewater in cold environments, for in-house reuse or external distribution [30]. Studies show that WWTP's effluent discharge can be utilised to generate hydropower (NYSREDA, 2011). All these innovations should be quantified in water sector CBA as investment costs, and net present values highlight the benefit, i.e. avoided cost and income generated, of e.g. harnessing the energy for in-house use or external distribution vs. paying for that energy from a supplier.

Resource recovery is another widely established valuation perspective, where water, energy, nutrients and metals are key components. While treatment of wastewater is a recovery process and the primary purpose of all WWTP, some regions of the world value recycling of treated water with significant positive net present value compared to extracting otherwise scarce water resources [31]. These WWTP return treated water for use in agricultural instead of discharging to a water body. The avoided cost of using treated potable water in agriculture is a widely practiced CBA. Furthermore, the extraction of phosphates and nitrates from sludge in WWTP can also

be returned for agricultural purposes and stockpiled for future sales in phosphate and nitrate non-renewable markets that are significantly depleting on a global scale [32].
Water sector's contribution to carbon emissions is another factor which as a rule neglected in CBA in practice. A study reports that 1% of UK's carbon emissions are related to the water sector (CIWEN, 2013). The net carbon emissions in a specific project could widely vary depending on the project type and alternatives scenarios. While LCAs generally address this component, monetary costs are often neglected despite the widely available financial data. There is a need to limit the scope at which a water sector project is accounting for its carbon footprint to the project-oriented scope of the CBA.

3.2 Conceptual proposals in theoretical and pilot stages

This section explores the theoretical and pilot scale valuations of impacts found in research and innovative industrial examples in the water sector. In the operational phase of a project, surveys on groups of individuals' willingness to pay (WTP) help reveal e.g. the value of buffer zone land surrounding WWTP in terms of odour control, agricultural and recreational potential [33]. Here, the choice experiment monetary valuation method underpins the survey. Using the survey data expressed in dollars, the avoided cost of having to build odour reducing infrastructure can be compared against the investment needs and social benefits end-users anticipate from having recreational or agricultural access to land used as e.g. odour buffer zones around the WWTP. Surrounding natural scenery, recreational value (bathing water quality of recipients) and visual aesthetics (obstructive infrastructure) can be permanently affected by a WWTP. The most frequent complaints relate to odour, which is measured in point source concentric emission circles in unit odour [22]. Conversely, projects could also result in positive impacts by re-establishing historical water quality and use. In the operational stage, the physical presence of a water treatment plant above surface, underground, embedded in rock formations or (semi-)submerged solutions can have aesthetical impacts reflected in the housing market. A study on WWTP and sewer construction's impact on water bodies neighbouring potential tourism and hotel destinations [34] uses revealed preference techniques such as hedonic pricing and travel cost method to place value on the cost-benefit of this water project's impact on improved recreational areas, odour and aesthetics. The hedonic pricing method looks at housing market prices before and after the project construction to derive how much the water project impacts cost or benefitted the surrounding property owners. The travel cost method calculates the fuel, toll, entry, food, lodging etc. costs a customer is willing to spend to visit an e.g. tourist destination. The study [34] transfers travel cost values from a similar site to indicate the potential profitability of introducing sewers and converting waste-filled lands into desirable tourist destinations.
The socioeconomic costs-benefits to human health must be accounted for if any project phase carries risk for improvement or injury to human health and mortality. United Nation's Sustainable Development Goal 6 call for access to safe and affordable water, sanitation and hygiene, and ending open defecation [3]. Access to adequate treatment of wastewater is valued by avoided negative human health impacts such as medical and sick day costs. Some water sector studies explore implementing the value of a statistical life (VoSL) [35], which is the WTP to reduce the risk of a health impact as a result of the water project. In CBA, the dose response method use these VoSL value and multiply with previous or estimate amounts of deaths to assert the human health impact cost of this project [10]. These valuations account for the need to invest in safety measures during project stages against the cost avoided to compensate for workplace injuries and death, should incidents occur [36].
Transport cost-benefits in any stage of a project include investigating (non-)renewable fuel consumption and pollution, road dust and wear, noise and odour emission and all cost local populations in various environmental and socioeconomic ways [5]. The conversion of these physical values to CBA monetary values are outlined for a number of LCA models [37].
The WWTP building footprint and its temporarily expanded construction worksite footprint can impact access and conditions of culturally and economically significant objects and areas. The impact on the nation's culture can be measured by travel cost and WTP surveys [37].
Project impacts on safety for nearby residents is measured in terms of access and traffic safety to nearby recreational sites or travel pathways alongside the new WWTP, typically with VoSL methodologies and lost work time [38].

3.3 Potential processes adaptable from other sectors

The use of monetary valuations for CBA and other analysis tools are practiced in many industries, where certain sectors have a stronger track record than the water sector in terms of valuating environmental and socioeconomic impacts. Existing gaps in practical and academic application of monetary valuations in the water sector may be filled by borrowing or adapting methods more widely applied in these sectors. The following section aims to fill these gaps by encouraging the development or adaptation of valuation techniques from other comparable and transferrable sectors.

Some sectors stand out in terms of strong historic and current track records of comprehensive use of CBA, meaning the inclusion of socioeconomic and environmental impacts alongside financial impacts of projects, Roads and Transport, Public Health and the Energy and Agricultural sector. In many countries, these areas are public sectors which outbid the water sector from the same limited government funding pot. This may be due to these sectors being more proficient at expressing their projects' comprehensive impacts and net benefits to society in comparison to the water sector. At first glance, these sectors may also have more immediate impacts on non-market costs, hence the development of human life valuations for road safety, carbon emissions from non-renewable fuel consumption on roads and the life-saving value of hospitals. The cost-benefits of universal access to clean water and sanitation may impact over half the deaths in this world, but the link between water infrastructure and benefits to human health may not be as immediate as the other sectors. The water sector is guilty of not reporting these impacts themselves when these valuation tools are available.

The Roads and Transport sector are at the forefront of placing monetary value on direct impacts of infrastructure projects on local populations. In Norway, EIA valuation manuals that include elements of monetary, non-monetary and qualitative assessments of public sector projects inherit their sector manuals from the Norwegian Public Roads Administration's master guide [39]³⁹(Norwegian Public Roads Administration 2018)³⁹(Norwegian Public Roads Administration, 2018)(Norwegian Public Roads Administration, 2018) . Many advances were made in the past 20 years in noise, odour, congestion and air pollution (Navrud et al., 2006). Noise pollution valuations are established, where dB emissions are tiered into radii of near, medium and long distance from the noise point source at the project site (Navrud 2002). These data are multiplied by length of exposure to show decision makers the avoided cost of subjecting the project site's neighbours to noise. The roads sector also have robust models estimating the cost of traffic delay, in terms of lost work hours or additional fuel costs, due to temporary construction work surrounding project sites [42].

The Public Health sector have been closely tied into the development of VoSL and quality of life indicators on human health such as DALY (Disability adjusted life year), VOLY (value of a life year lost) and QALY (quality adjusted life year), alongside health indicators such as avoidable deaths, premature deaths, reduced excess deaths. Papers from this sector assess impacts of avoided cost of injury and death in a region by air pollution improvements with VOLY [43], justifying treatment to intensive care patients with QALY [44], and the economic value of an African hospital ward with disability life years averted by its existence in the community using DALY [45].

The energy and agricultural sectors are also of publicly funded infrastructures which have come further in estimating bequest and existence values, impacts that belong on the non-use side of the total economic valuation tree. Existence and bequest values are non-use impacts which should enter into CBA frameworks for the water sector. The bequest or preservation of fish stock and the existence of fish stock in a water body is a value individual are willing to pay for, which is surveyed and usable as a valuation as a cost should a project disturb those values (Navrud, 2008). These non-use valuations people not living near the project site may still have direct impact on project cost benefits, such as national value placed on investing in sustainable climate-saving infrastructures [47] or preserving a region's cultural and heritage as monetary valuated through the travel cost method [48].

Table 2 lists which project impacts are typical to wastewater infrastructure. The references in the table provide a list of resources for how water sector decision makers may begin to place monetary value on socioeconomic and environmental impacts of their project. Each reference contains equations, algorithms or process descriptions on which baseline data are needed to arrive at a net present value for projects involving these impacts.

Direct WWTP impacts	Project phases I	C	O	M	D	References
Financial impacts						
Deed/land acquisition	■					(Tsagarakis et al., 2003)
Permits, lab analysis	■	■	■	■		(Yue et al., 2020)
Labour	■	■	■	■	■	(Gillot et al., (1999); Zessner et al., (2010); Molinos-Senante et al., (2010)
Exploratory reports	■				■	(Tsagarakis et al., 2003)
Materials/chemicals		■	■	■	■	(Venkatesh & Brattebø, 2011)(Vanrolleghem et al., 1996)
Equipment		■	■	■	■	(Gillot et al., 1999), (Tsagarakis et al., 2003)
Fuel/Energy		■	■	■	■	(Sonesson, 1996), (Longo et al., 2016)
Water tariff			■			(Begum et al., 2006)
Rest value					■	(Lue-Hing et al., 1996), (Ozdemir & Yenigun, 2013), (Zessner et al., 2010)
Waste storage		*	*	*	*	(Djukic et al., 2016)
Waste disposal			▒	▒	▒	(Norwegian Public Roads Administration, 2018)
Societal tax financing	*	■	*	*	*	(Bos et al., 2018)
Environmental impacts						
Avoided accidents		*	▒	*	*	(Ikpe et al., 2008), (Haslam et al., 2005), (Tang et al., 2004)
Particle matter pollution		■	■	■	*	(Norwegian Public Roads Administration, 2018),
Climate change impacts		*	▒	*	*	(Markandya & Chiabai, 2009)
Smell pollution			▒	▒		(Gössling et al., 2019), (Batalhone & Nogueira, 2002)
Human health end users		*	*	*	*	(Elixhauser et al., n.d.)
Noise pollution		*	▒	*	*	(Ståle Navrud, 2002)
Resource reuse			▒			(Lema & Suarez, 2017), (Iacovidou et al., 2017), (Garcia & Pargament, 2015)
Eutrophication			▒			(Bateman et al., 2006)
Micropollutants and PPE		▒	▒	▒		(Logar et al., 2014)
Socioeconomic impacts						
Recreational value		▒	▒	▒		(Segui et al., 2009), (Verlicchi et al., 2018)
Traffic/commute delay		*		*		(Rister & Graves, 2002; Sala & Student, Eu-Asia, 2013; Thoft-Christensen, 2009)
Terrestrial ecology		▒	▒	▒	▒	(Ko et al., 2004)
Heritage & Culture		*	*	*		(McLoughlin et al., 2006), (Mizokami et al., 2003)
Safety of onsite workers		*	*	*	*	(Ikpe et al., 2008), (Charles et al., 2008)
Agric. & natural resources			▒	▒		(Haruvy, 1997), (Arborea et al., 2017)(Libralato et al., 2012)
Tourism/tertiary industry		▒	▒	▒	▒	(SACEP & UNEP, 2000)
Existence & bequest value			▒	*		(Loomis et al., 2000)(Becker et al., 2007)

Legend: ■ Established practice ▒ Academic/pilot studies * Unpractised/borrowable practice
I = investment, C = Construction, O = Operation, M = Maintenance, D = Decommission

In table 2, most common impacts (rows) related to the five project phases (columns) that are categorised in to three status tabs (1) regularly practiced in the water sector in some form of impact assessment (dark grey, established practice); impacts that are academically explored in theory or pilot scales in the water sector (light grey, academic/pilot study); and impacts that should look to borrow methodology from other sectors where the valuation is more widely practiced (starred, unpractised/borrowable practice). The tabs are distributed across the five main phases of any WWTP infrastructure project (I = Investments, C = Construction, O = Operation, M = Maintenance, D = Decommission), and the impacts are screened within traditional CBA boundaries; direct impacts within the project's economic lifetime at a local to national scale. Instructional resource(s) are referenced for each impact, providing a platform for the most essential methods that could underpin a more accessible CBA framework. Multiple references have been provided in table rows where an impact may be widely practiced for one project stage and less in others (different legends in same row), or multiple relevant sub-category examples are found (multiple references in a mono-coloured row). The table is a matrix of executable monetary valuation methods and is a useful accompany to any CBA flowchart, for which we present a suggestion specific to practically applicable CBA for the water sector, see figure 4.

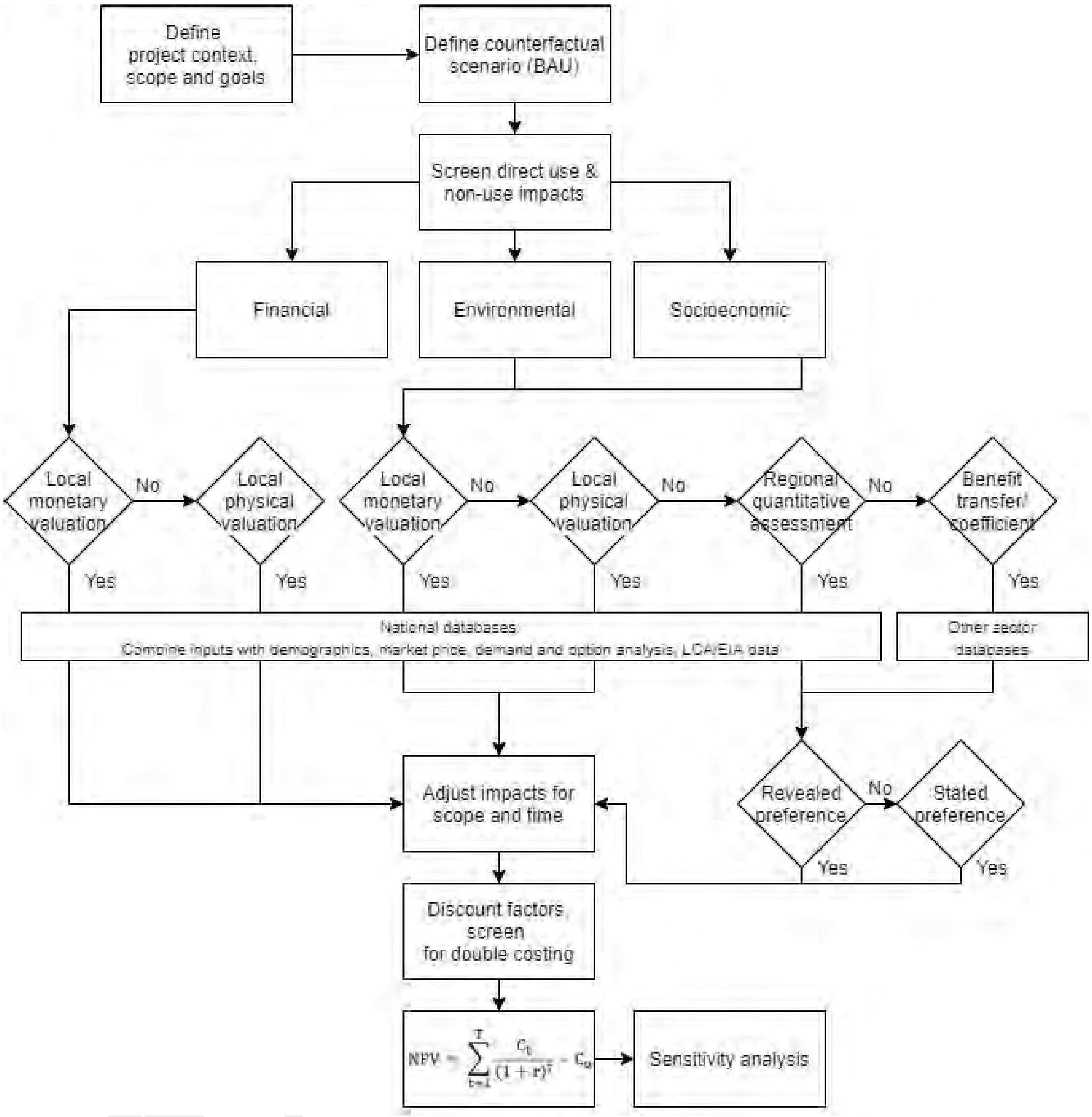

Figure 4: The roadmap flow chart of executing CBA for each project impact, combining LCA and EIA data as drivers, expanding on reference matrix in table 2 and the comprehensive CBA framework by Ratnaweera et al. (2020).

Each impact is not equally impactful, nor is the list exhaustive. The listed impacts are intended as umbrellas for a range of sub-category impacts developable into an ever-growing database. The impacts relate to WWTP and water sector impacts, and expand on the impact factors identified by most CBA guidelines as outlined by Ratnaweera (2020). Listed impacts are not equally impactful within each project, nor does each impact category (financial, environmental, socioeconomic) have the same amount of umbrella impacts listed. However, the tabular distribution in table 2 is notable in that most widely practiced valuation methods (dark grey squares) are found among the financial impacts. Environmental impacts are not as often equated to monetary values in the water sector as they are in other sectors, and stand to benefit from borrowing methods. The reason may be the that the water sector is currently very fluent in reading environmental impacts in physical units as produced by LCA's and EIA's. The clustering of tabs in table 2 indicate that many of these light grey tabs are methodologies rooted in economic principles that are applied for the first time in a case study or pilot WWTP plant.

These findings align with the institutional reports tabulated in table 1, insofar that CBA's stand to benefit from other assessment tools and non-water sector practices.
The impact valuation matrix (table 2) needs a road map for combining baseline data, the table's listed impact valuation methods and CBA calculations over a project lifetime, for each impact in each project phase. The flowchart in figure 4 helps WWTP projects guide this process, to arrive at net present value (NPV) data that help decision makers identify the comparatively net beneficial project scenario alternative.
The flow should guide water sector decision makers in utilising references from table 2 in a systematic way with statistics, market data, LCA and EIA data available to them. The purpose of the flow chart is to guide decision makers into both remind and only select impacts relevant to the scope of their project's CBA. The references may contain algorithms on how to monetary valuate impacts, but this flow chart outlines the hierarchy of preferred valuation methods, and which considerations must be made towards arriving at a comparable net present value between project scenarios-

4. CONCLUSIONS

CBA, LCA and EIA are the most commonly used impact assessment tools in water sector projects. Of those, CBA is the only decision support tool with an existing framework that promotes impact comparison in a unified unit language, namely monetary values. The water sector is encouraged and often required to perform CBA when justifying the use of governmental or institutional funding for public goods such as WWTP. However, in practice, most reports indicate only the financial CBA is widely executed in the water sector. This is supported by findings in this paper. Reports and publications on typical financial, socioeconomic and environmental impacts relating to WWTP infrastructure reveal that environmental factors are largely mapped by qualitative data or LCA and EIA physical unit data. Meanwhile, socioeconomic impacts are rarely valued at all. Both of these latter impact categories are more widely assessed in other public sectors such as roads, public health, energy and agricultural sectors, in comparison to the water sector.
This paper's findings are a compendium of monetary valuation methods that are relevant to an umbrella of impacts that occur over the lifespan of a WWTP infrastructure project. The findings are categorised into building blocks that shall provide the foundation for a roadmap to apply CBA in a comprehensive and approachable way for the water sector. The building blocks are divided into monetary valuation methods currently practiced in water sector, as evidenced by reports and guidelines; currently developed in academic theory and/or tested in pilot scale, as evidenced in academic publications; currently practiced in comparative sectors that compete with water sector for limited public funds, as evidenced by reports and publications. The findings are tabulated in a reference matrix, where the references guide readers to equations, algorithms and data sourcing methods relevant for water sector projects needing monetary valuation of their project impacts. A flow chart accompanies the reference matrix to guide decision makers in the hierarchy of which types of monetary valuations are preferred, based on robustness and commonness.
The focus of these efforts is to make the comprehensive CBA more accessible to water sector project managers and decision maker. The reference matrix is underpinned by the paper's building block results, and the proposed CBA flowchart is tailored for WWTP and water sector projects and provide a roadmap for project managers.
That roadmap outlines how to make aggregate, rough estimates of which financial, socioeconomic and environmental impacts should be looked at closer, and which impacts perhaps demand sourcing of more primary data, before going ahead with an intended project. The purpose of the water sector specific CBA framework, comprised of the reference matrix and flow chart, is to help decision makers read the social utility of their projects in a uniform unit language, i.e. monetary currency. The future need for the completion of this more approachable, applicable and comprehensive CBA roadmap is a compendium of techniques involved in each valuation method, alongside a suggested ranking or precedence of monetary valuation methods per water sector umbrella impact category, a proposed catalogue of relevant primary and secondary data sources, alongside conversion methods of secondary data from source to site composition.

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Rainwater Harvesting Design Approaches

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ABSTRACT

Different approaches have been developed to design rainwater harvesting systems. However, these have not been widely applied in schools and are not well understood. This paper presents findings from a study in which different rainwater water harvesting design approaches are selected and applied to a case study school. The approaches are introduced and the results of their application to the case study school are critically evaluated. The study finds that the methodologies have different strengths and weaknesses. A critical analysis of the results indicates that one approach may be too simplistic and provide misleading results. Another approach does not provide adequate outputs to fully design rainwater harvesting systems. A third methodology is complex to apply and requires data that is difficult to obtain. Based on the study, recommendations are made on how the selected approaches can be improved to enable these to be used more easily to design school rainwater harvesting systems.

Keywords: Schools, rainwater harvesting, rainwater harvesting modelling and calculators

1. INTRODUCTION

Climate change is resulting in higher average temperatures, an increase in the number of very hot days, changes in rainfall and more frequent droughts (IPCC, 2022). This is resulting in water scarcity in many areas of the world, and this is particularly severe in countries with hot dry climates. UNESCO (2019) projects that two-thirds of the world's population will experience water scarcity for at least one month of the year and there will be a 40 % gap between water demand and supply by 2030 (UNESCO, 2019).

Water scarcity can have significant negative impacts on schools. Students and teachers need water for drinking, cleaning, washing and flushing toilets. When there are water outages, tankers need to be arranged to bring water to schools. This arrangement can be very expensive and only a limited supply of water can be delivered. Where water cannot be brought in, the school has to close. This disrupts teaching and learning, results in lower educational achievement and if prolonged, can limit the ability of students to access employment opportunities.

As water supplies become increasingly unreliable it makes sense for a school to understand their patterns of water use, so that they can use water more efficiently. It is also important that they explore alternative sources of water that can be used to augment or replace current unreliable supplies.

One of the simplest alternative water supplies available to schools is rainwater (Thuy et al., 2019). Schools have roof areas that can be used as collection surfaces and usually have the space for rainwater tanks. It is, therefore, surprising that more schools do not adopt rainwater harvesting. Several reasons have been put forward for the low adoption rates of rainwater harvesting systems including a lack of information and accessible methods for developing designs.

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This gap is addressed in this paper through a review of the uptake of rainwater harvesting systems and an analysis of three rainwater harvesting system modellers and calculators. Modellers and calculators are applied to a case study school and the results are critically analysed. The review and analysis are developed into recommendations that can be considered to support rainwater harvesting in schools. The study aims to address the following research questions.

- What are rainwater harvesting systems and why are they not adopted more widely?
- How do the rainwater harvesting calculators and modellers work and how can they be used to understand more about the design of systems at schools?
- What can be learnt about the calculators and tools analysed? Can these be improved?

2. LITERATURE REVIEW

Ghaffarian Hoseini et al. (2016) suggest that rainwater harvesting could meet 80–90% of overall household water consumption globally. Steffen et al., 2012 also point out that the widespread installation of rainwater harvesting systems in cities avoids the need to construct new centralised water supply systems. However, despite clear benefits, there has not been widespread adoption of rainwater harvesting systems.

Several factors are attributed to the slow adoption of rainwater harvesting systems. One factor has been the actual and perceived cost of systems (Rahman et al, 2014; Campisano et al., 2017; Akuffobea-Essilfie et al., 2020). Another concern has been about water quality (Rahman et al, 2014; Fewtrell and Kay, 2007). A lack of information on rainwater harvesting systems may also have contributed to their slow adoption (Akuffobea-Essilfie et al., 2020; Sheikh, 2020). In some areas concerns about regulations and maintenance have also been an obstacle to adoption (Campisano et al., 2017).

However, it appears that once rainwater harvesting systems have been installed, users appear strongly supportive of this technology (Gould,1997; Fuentes-Galván et al., 2018; Sunkemo and Essa, 2022). In developed 'water-rich' countries rainwater harvesting systems are usually considered backup systems whereas in dry developing countries they may be the primary supply (Cook, et al., 2013). Supportive local regulations and incentives are effective in supporting the increased adoption of rainwater harvesting systems (Domenech and Saurí, 2011). Kerlin et al. (2015) indicate that rainwater harvesting systems have a range of additional benefits and in schools can provide a useful teaching and learning resource.

Different systems have been developed to model and calculate the potential of rainwater harvesting systems. These calculate or simulate rainfall capture and water demand over time and, by matching these, calculate the extent to which rainwater harvesting systems can meet organizational water requirements (Campisano et al., 2017).

These models are based on the following approach (Campisano et al., 2017). Firstly, a behavioural model is developed based on water demand. This model may draw on literature reviews and benchmarks or metered data and shows flows of water out of the system as it is used over time.

Secondly, a rainwater model is developed to present rainwater captured. Flows are based on climate data or local rainfall records and volumes may be presented over time intervals of a minute, hour, day or month. This takes into account losses due to runoff, filtration and overflows. This represents the flow of rainwater in the system, as it is captured.

Third, a calculation module simulates volumes of water in the rainwater tank. This calculates the balance between the water used and rainwater captured. This represents the volume of water in the tank over time.

Fourth, data from the three components are presented so that water inflow and outflow patterns can be discerned and understood. Data may be presented in table or graphic form to show flows and volumes over a year. This data is designed to support decision-making on the feasibility and design of rainwater harvesting systems. It can also be used to understand the impact of interventions such as increasing catchment areas or reducing water consumption can make (Campisano et al., 2017).

3. METHODS

The methodology for the study can be described in four steps. First, an integrative literature review is undertaken to identify the key issues that may be of concern to schools interested in school rainwater harvesting systems. The literature review analyzes empirical findings from previous research to understand more about the field and to develop the research approach and methodology for the study (Tranfield, Denyer, & Smart, 2003; Snyder, 2019).

Second, three calculators and modellers are selected. These are selected as they represent different approaches to rainwater harvesting. Each of these calculators and modellers is introduced and explained.

Third, the selected calculators and modellers are applied to a case study school and the results are captured. Data on the school, including learner and staff populations, are obtained from the school. School plans and Google maps were used to acquire school and site areas and other data on school infrastructure. A walkthrough of school facilities was used to obtain data on equipment and fittings that used water. The results are shown in the form of reports from the calculators and modellers and are critically analysed.

Fourth, results from the application of the calculators and modellers are discussed in terms of the literature review and findings developed. These assess the value of the three approaches and identify how they could be improved to support improved uptake of rainwater harvesting in schools.

4. RAINWATER HARVESTING CALCULATORS AND MODELLERS

Three different calculators and modellers are selected for the study. These are the Centre for Affordable Water and Sanitation Technology (CAWST) calculators, the School Rainwater Use Model (SWARUM) and Rainwater Use Model (RUM). These present three different approaches to understanding and developing rainwater harvesting systems and are presented below.

4.1 Centre for Affordable Water and Sanitation Technology (CAWST)

The Centre for Affordable Water and Sanitation Technology (CAWST) developed a manual on rainwater harvesting which provides calculations and practical guidance on how to design rainwater harvesting systems (CAWST, 2011). This methodology is selected for this study as it is accessible and simple to use as it consists of easy-to-understand equations and data. It is also likely to be representative of the most commonly used method of designing rainwater harvesting systems.

4.2 School Rainwater Use Model (SWARUM)

The School Rainwater Use Model was developed by the author as a tool that schools could use to understand rainwater and water use in their buildings better. It aims to enable schools to be able to quickly see the potential of rainwater harvesting systems and understand the implications of interventions such as increasing rainwater collection surfaces or reducing water consumption. The tool was selected as it aims to support knowledge and understanding of rainwater harvesting systems by non-technical users.

4.3 Rainwater Use Model (RUM)

The Rainwater Use Model has been developed by the author to model rainwater capture, water use and tank levels over a year on a daily scale. The methodology requires daily rainfall figures and daily water consumption volumes. This tool was chosen as it is representative of the rainwater harvesting simulation tools that aim to reflect the detailed performance of water and rainwater systems.

5. CASE STUDY SCHOOL

The case study school is in Pretoria, South Africa (25° S, 28° E). Annual rainfall figures for the site are about 700mm (Worldclimate, 2022). The school is set on grounds of 3.33ha and has a roof area of 5,970m^2 that can be used for rainwater harvesting. The school is occupied 206 days a year and has 1,024 occupants (including staff). Equal numbers of female and male occupants are assumed.

Water fittings at the school have the following performance: toilet flush rates are 9 litres/flush, urinal flush rates are 2 litres/flush and wash hand basin flow rates are 10litre/minute. The school also has a swimming pool and irrigated grounds. Calculations indicate that annual daily water consumption per occupant is about 30litres. This figure is used for consumption in the calculations and models.

6. RESULTS

Data from the school and climate is used in three different calculators and modellers to generate results. These are presented below.

6.1 Centre for Affordable Water and Sanitation Technology (CAWST)

Applying the Centre for Affordable Water and Sanitation Technology (CAWST) calculations to the case study is outlined below. To calculate rainwater supply the following calculation is used.

- Amount of rainfall: 700mm
- Catchment area: 5,907m^2
- Runoff coefficient: 0.9 (for a corrugated iron roof)

Rainwater supply (m^3/year) = Rainfall (m/year) x Catchment Area (m^2) x Runoff Coefficient

Rainwater supply (m^3/year) = 0.7 x 5907 x 0.9

Rainwater supply = 3721 m3 or 3,721,410 litres

This overall supply can be compared to water requirements by calculating the water requirements for the school as follows.

- Number of occupants: 1024
- Consumption per occupant: 30litres
- Number of occupied days: 206

Annual water requirements (l) = number of occupants x consumption per occupant x number of days

Annual water requirements (l) = 1024 x 30 x 206

Annual water requirements (l) = 6,328,320litres (30,720litres per occupied day)

In this example, as the annual requirements (6,328,320litres) are higher than the rainwater harvested supply (3,721,410litres) the CAWST calculations indicate that systems would only be able to meet about 50% of the water requirements of the school.

The CAWST calculations for the size of the rainwater tanks are outlined below.

Size of rainwater tank required (l) = Longest dry period (days) x daily water consumption (l)

A review of monthly rainfall figures indicates that May, June, July and August have very low rainfall and therefore the dry period is approximately 4 months or 120 days. Over this period the school is occupied for 73 days.

Size of rainwater tank required (l) = 73 x 30,720

Size of rainwater tank required (l) = 2,242,560 litres.

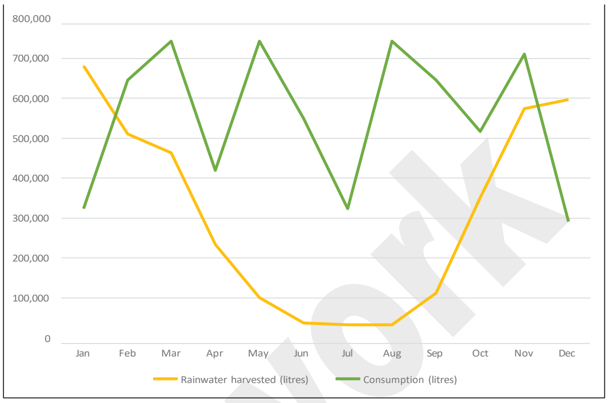
6.2 School Water and Rainwater Use Model

Entering the case study data into the School Water and Rainwater Use Model generates the report shown in Figure 1.

School Water and Rainwater Use Modeller

School name	Case Study School
Address	Pretoria, South Africa

Female occupants	512	number
Male occupants	512	number
School year	206	days
Drinking water per occupant		litres
Washing water per occupant		litres
Water consumption per WC flush		litres
Water consumption per urinal flush		litres
Collection surface area for rainwater harvesting	5907	m2
Collection surface runoff factor	0.9	
Water consumption per occupant per year	6,489	litres/yr
Water consumption per occupant per day	32	litres/day/person
Rainwater harvested per collection surface m2	630	litres/year
Volume of water deficit (figures in orange)	-3,586,374	litres/year
Percentage of water needs met from rainwater	65	%



Month	Rainwater harvesting				Balance	School water consumption							
Month	Rainfall (mm/month)	Collection surface (m2)	Runoff factor	Rainwater harvested (litres)	Surplus/deficit (litres)	Consumption (litres)	Days (premises occupied)	Female (staff and students)	Male (staff and students)	Drinking (litres per person)	Washing (litres per person)	WC (litres per flush)	Urinal (litres per flush)
Jan	128	5907	0.9	**680,486**	357,926	**322,560**	10	512	512	3	6	9	2
Feb	96	5907	0.9	**510,365**	-134,755	**645,120**	20	512	512	3	6	9	2
Mar	87	5907	0.9	**462,518**	-279,370	**741,888**	23	512	512	3	6	9	2
Apr	44	5907	0.9	**233,917**	-185,411	**419,328**	13	512	512	3	6	9	2
May	19	5907	0.9	**101,010**	-640,878	**741,888**	23	512	512	3	6	9	2
Jun	7	5907	0.9	**37,214**	-511,138	**548,352**	17	512	512	3	6	9	2
Jul	6	5907	0.9	**31,898**	-290,662	**322,560**	10	512	512	3	6	9	2
Aug	6	5907	0.9	**31,898**	-709,990	**741,888**	23	512	512	3	6	9	2
Sep	21	5907	0.9	**111,642**	-533,478	**645,120**	20	512	512	3	6	9	2
Oct	66	5907	0.9	**350,876**	-165,220	**516,096**	16	512	512	3	6	9	2
Nov	108	5907	0.9	**574,160**	-135,472	**709,632**	22	512	512	3	6	9	2
Dec	112	5907	0.9	**595,426**	305,122	**290,304**	9	512	512	3	6	9	2
	700			**3,721,410**	**-2,923,326**	**6,644,736**	**206**						

Fig. 1. The School Water and Rainwater Use Modeller (Author)

The top left-hand corner of Fig 1 shows data entered into the SWARUM. The table at the bottom indicates monthly rainfall and days occupied (in blue columns). The balance column in this table substracts water consumption from rainwater harvesting to indicate a surplus or deficit over the month. The graph in the right-hand corner indicates water consumption (in green) and rainwater harvesting (in yellow) over 12 months.

6.3 Rainwater Use Model (RUM)

The data from the case study is entered into the Rainwater Use Model to produce the report shown in Figure 4. The table at the top has information about the site. Underneath this is a graph that shows daily water consumption patterns over a year. This is followed by a table that provides an analysis of water use and a table that enables data on rainwater harvesting collection surfaces to be captured. The graph beneath this shows daily rainfall over a year and is followed by a table that provides an analysis of this. A graph of rainwater capture and use is then provided, which shows levels of water in rainwater tanks as well as rainwater capture and water use over a year. At the bottom of the report is an analysis of the performance of the system as a whole.

RUM 2020

Rainwater Use Model version 2020

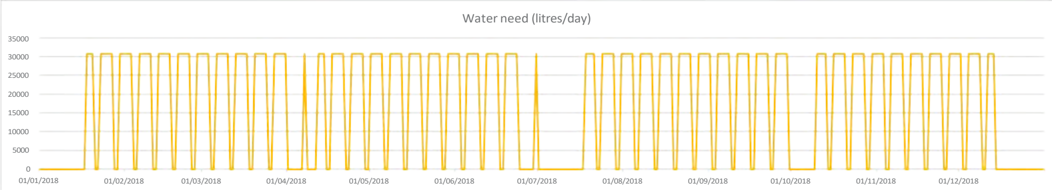
A tool for understanding water needs, local rainfall patterns and modelling rainwater harvesting systems.

The Rainwater Use Model uses **Water needs (B)**, **Collection surface (D)**, **Precipitation (E)** and **Rainwater tank size (H)** data to model rainwater capture, storage and use by a rain water harvesting system (E). The light blue areas can be populated with data and changed, other cells are generated by the model and are fixed. Training on the tool is available.

A. Project details

1	Name of project	RUM test one
2	Address of project	Pretoria
3	Function of site/building	School
4	Responsible person and contact details	
5	Purpose of rainwater harvesting	School use

B. Water needs



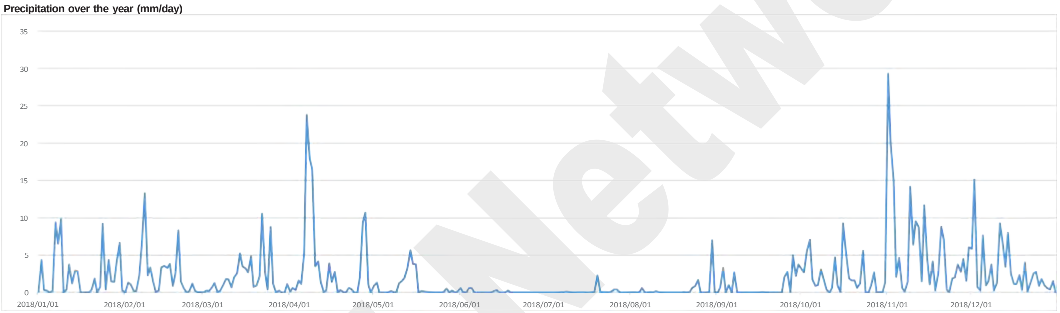
C. Water needs analysis

1	**Month**	**Daily water requirements (l)**	**Notes**
2	Average daily requirement	30,720	litres/day
3	Average monthly requirement	535,040	litres/month
4	Yearly water requirements	6,420,480	litres per year

D. Collection surfaces

1	**Location/name of collection surface**	**Area (m2)**	**Runoff coefficient**
2	*Area 1*	5,907.00	0.9
3	*Area 2*	0.00	0.9
4	*Area 3*	0.00	0.9
5	*Area 4*	0.00	0.9
6	*Area 5*	0.00	0.9
7	*Area 6*	0.00	0.9
8	*Total area of collection surface*	5907	

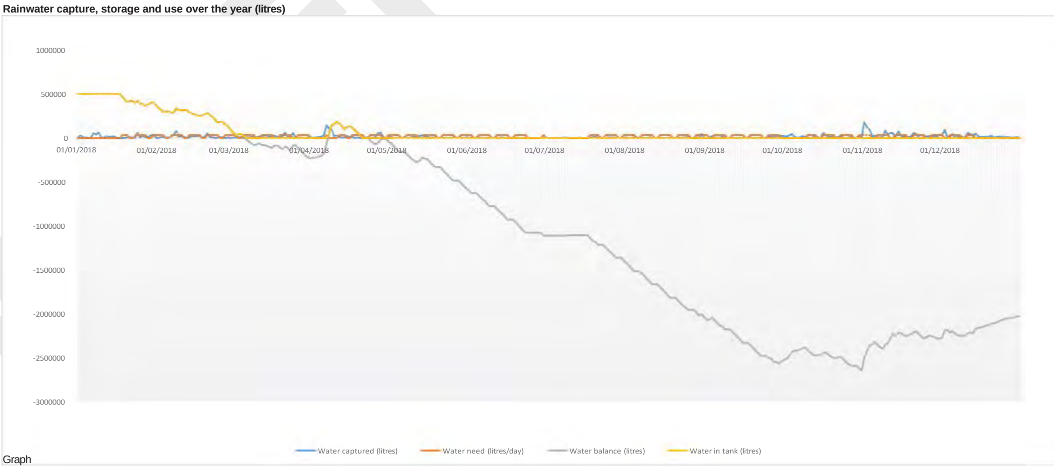
E. Precipitation over the year (mm/day)



F. Precipitation analysis

1	Annual rainfall	700	mm/year
2	Number of days with no rain	82	days
3	Number of days with rain	283	days
4	Longest period with no rain	64	days
5	Longest period of consequtive rain days	10	days
6	Maximum rainfall over 24 hours	29	mm

G. Rainwater capture, storage and use over the year (litres)



Graph

H. Matching rainwater harvesting and storage to water needs

1	Starting water level in rainwater tank	500,000	litres (this must be lower or equal to the water level of the last day)
2	Size of rainwater tank	500,000	litres
3	Contingency (days supply at lowest level)	0	days
4	Maximum volume of water in tanks	500,528	litres
5	Minimum volume of water in tanks	0	litres
6	Amount of water needed over the year	6,420,480	litres
7	Amount of rainwater captured off surfaces	4,134,807	litres
8	Rainwater harvested, stored and used	1,351,680	
9	Over(+) / under(-) capture of water	2,783,127	litres
10	Rainwater capture:harvest ratio	306	%
11	Percentage of need met from system	21	%

Fig. 2. Rainwater Use Model (Author)

7. ANALYSIS AND DISCUSSION OF RESULTS

The CAWST model calculates that a rainwater harvesting system can meet about half the school's water needs. It also indicates that a rainwater tank of 2,242,560 litres is required. This is calculated by multiplying the longest dry spell by the water consumption of the days that the building is occupied.

A review of the SWARUM model indicates that there are only 2 months (December and January) when rainwater harvesting exceeds consumption. In all other months, water consumption varies with the number of days there are occupants at the school but is consistently well above the rainwater harvesting. In Fig 1 consumption is shown as the yellow line, which oscillates depending on the number of days the school is occupied. Rainwater harvesting is shown as the green line which is only above the yellow line (consumption) for December and January and then drops rapidly in the dry season when very little water is harvested.

The WUM indicates that available water captured during the wetter months of January and December is rapidly used, emptying the rainwater harvesting tanks. The rate at which water is used prevents tanks from filling as any water captured is used immediately. This is reflected in the WUM graph in Fig 2. This shows the rainwater tanks have some water early in the year, but soon go into deficit and never recover. The WUM enables different rainwater tank sizes to be tried and 200, 300, 400, 500, 600, and 700 thousand litre tanks are tried. The WUM indicates that tank sizes above 500,000 litres are not worth installing as the additional storage volumes are never used as water is used before this can happen.

These are interesting results as they indicate the importance of understanding water use and rainwater harvesting systems as dynamic entities which need to be tracked and understood over a year. They also indicate the importance of understanding the capacity limitations of the different components of the rainwater harvesting system.

A review of the findings from SWARUM and RUM indicates that water consumption is so high and regular that any rainwater harvested is used rapidly, as water consumption substantially exceeds the amount of harvested rainwater for 10 months of the year.

It also shows that the size of the collection surface is a key limiting factor and that even in months with high rainfall such as December and January, rainwater collected is only slightly above the water consumption.

The results suggest that the tank size recommended in the CAWST calculation is too large and only 20-30% of this capacity would be used. The results indicate that a tank of about 500,000 litres would more appropriate.

The results suggest that if the school wanted to become self-reliant for water, they would have to do two things. Firstly, they would have to substantially increase the area from which rainwater was collected, as this is currently a key limiting factor. Secondly, they would have to reduce the amount of water consumed.

This is demonstrated in Fig 3. which shows how the school can be effectively off-grid and that water levels in the rainwater harvesting tanks are maintained above zero (are never empty). This is achieved by doubling the volume of rainwater harvesting tanks (100,000litres storage), doubling the area of the collection surfaces (11,000m^2) and halving water consumption rates (15litres/person/day).

The study indicates that the CAWST methodology can be undertaken rapidly and easily. However, it suggests that the CAWST approach results in rainwater tanks that are larger than necessary as it does not enable a picture of water use and rainwater capture over a year.

The SWARUM presents patterns of water consumption and rainwater harvesting capture so that these can be easily understood over a year. A weakness of this approach, however, is that it does not enable the size of rainwater harvesting tanks to be calculated. In addition, this methodology does not provide for 'transfers' of rainwater between months, when this is not used. This reduces its accuracy and results in it underestimating the impact of a rainwater harvesting system.

The study indicates the RUM is useful as it enables patterns of water consumption, rainwater harvesting and levels in rainwater tanks to be simulated. This provides for different scenarios to be tested. The inclusion of a rainwater harvesting tank water tank level indicator is also a valuable indicator of the resilience and sufficiency of the system. A drawback of this tool, however, is that it is based on daily rainfall and water use data, which may be difficult to obtain.

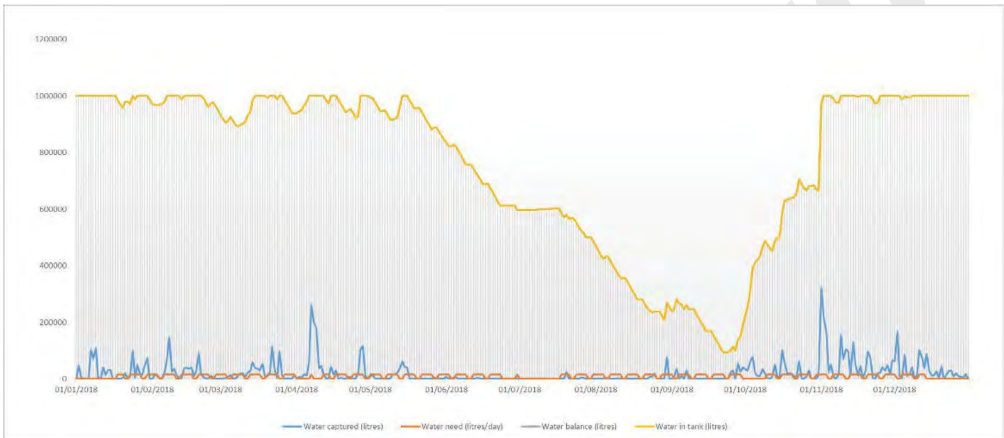


Fig. 3. A report from the Rainwater Use Model indicating water flow patterns over a year for increased collection area and reduced water consumption at the case study school.

8. CONCLUSIONS AND RECOMMENDATIONS

The study reviews three rainwater harvesting calculations and modelling approaches by applying these to a case study school. The results indicate that the calculations and modellers have different strengths and weaknesses. It indicates that while the CAWST approach may enable quick results, care should be taken in using it to design rainwater harvesting systems and tanks, as tank sizes may be oversized. The SWARUM's strength is its ability to produce yearly graphs of water consumption and rainwater harvesting quickly. The RUM tool appeared to be the most accurate tool and most suitable for designing rainwater harvesting systems. The detailed data required for its use, however, made it less suitable for general use.

The study recommends that care should be taken in using static calculation methods to design rainwater harvesting tanks and instead suggests that more dynamic tools that model the interrelationship between the capture of rainwater and the consumption of water be used. Of the approaches reviewed the study indicates that the SWARUM is most able to provide ready feedback to schools on the potential impact of installing a rainwater harvesting system. It is recommended that this system is developed to enable it to size rainwater harvesting tanks and use water levels in this as part of the model. The RUM is found to be most suitable for designing rainwater harvesting systems. However, its general use is restricted by its requirement for detailed daily rainfall data. It is therefore recommended that it be developed to use more readily available data, such as monthly rainfall data.

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School Water and Rainwater Use Modeller

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ABSTRACT

With climate change, schools in hot and dry areas are increasingly experiencing water shortages. This can affect the health of students and teachers, disrupt education and in the worst case, lead to school closures. Rainwater harvesting can help address water shortages by providing a safe alternative source of water. However, there is limited research and guidance on how rainwater harvesting systems can be applied to schools. A lack of guidance and knowledge has meant that schools are not aware of the potential of rainwater harvesting systems and do not adopt these systems. There is a need, therefore, for a simple tool that can be used by schools to understand the potential of rainwater harvesting systems at schools. This study aims to address this gap by developing the School Water and Rainwater Use Modeller (SWARUM). The modeller is presented and applied to a case study school in a drought-stricken area of Southern Africa. The findings of the application and the modeller are critically evaluated. The study finds that the modeller can be used to show the potential of a rainwater harvesting system at schools and enables different scenarios to be modelled and understood. The study makes recommendations for the improvement of the modeller and its application.

Keywords: Schools, rainwater harvesting, School Water and Rainwater Use Modeller

1. INTRODUCTION

Climate change is resulting in increased temperatures, long dry spells and the occurrence of droughts and water scarcity in many areas (Diedhiou et al., 2018; Makki, 2015; IPCC, 2022). Rapid urbanization exacerbates this problem by placing additional demands on existing water supplies that are already struggling to meet demands (UN-Habitat and IHS-Erasmus University Rotterdam, 2018). Limited capacity and resources in water utilities and municipalities mean that water infrastructure may not be maintained resulting in increased leakage and unreliable supplies (Wensley and Mackintosh, 2015). This combination is leading to increasingly unreliable water supplies and shortages in many areas.

A lack of water at schools has severe consequences and can lead to closure as they cannot function without drinking water, water for cleaning and water for flushing toilets (Jasper and Bartram, 2012). The closure of schools, even for a short time, has negative impacts on teaching and learning and the ability to achieve required education outcomes. If closures are prolonged, this in turn can negatively affect students' access to employment opportunities.

Large-scale interventions can be carried out to improve the resilience of municipal water supplies, such as increasing locally stored water (Gibberd, 2017). However, these interventions require significant resources and capacity and may take a considerable time to implement. It is therefore important that schools investigate what they can do themselves to improve the reliability and resilience of their water supplies. One of the most effective ways to do this is to have onsite water storage and to use rainwater harvesting. This reduces the reliance of the

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school on external water supplies and enables the schools to run for a part, or all, of the school year, on their supply.

However, as there is limited guidance on rainwater harvesting at schools, it is difficult for schools to ascertain whether this solution would work in their circumstances. This study presents a simple modeller that has been designed to enable a school to model key characteristics of water consumption and rainwater harvesting at a school. The modeller presents water use and rainwater harvesting capture patterns and allows the implications of different interventions to be determined. The modeller is applied to a case study school to show how it can be used to inform decision-making. The results of this application are critically reviewed to evaluate the value of the modeller and to make recommendations for its improvement. The study aims to address the following key questions:

- How can a school rainwater harvesting modeller be developed?
- What can a school rainwater harvesting modeller be used for?
- Is a school rainwater harvesting modeller useful in supporting decision-making in schools?

2. METHODOLOGY

To develop the school rainwater harvesting modeller an integrative literature review is undertaken. Literature reviews are used to analyse empirical findings from previous research to develop new models (Tranfield, Denyer, & Smart, 2003). Reviews can be systemic and based on highly defined rules for the selection of literature or, they can be integrative, where literature and data are sought from a range of sources to develop new models and knowledge (Snyder, 2019). An integrative approach is used to develop a specification for a school rainwater harvesting modeller and to develop a tool that responds to this. The modeller is developed in Excel as this software is readily available to schools and supports rapid development and testing.

To test the modeller and ascertain how it can be used to support decision-making, it is applied to a case study school. The case study school is selected because it is typical of many schools in drought-stricken areas. The modeller is used to investigate the impacts of installing a rainwater harvesting system at the school and to explore the implications of making changes to the water and rainwater harvesting systems. The case school is in the Eastern Cape of South Africa and data on the school and local climate are sourced from site visits, school databases and online resources such as Google Maps

Finally, results are critically reviewed and discussed to ascertain the value of the modeller as a tool to support decision-making about rainwater harvesting systems at schools. Conclusions are drawn and recommendations made for further research and development of the modeller.

3. RAINWATER HARVESTING IN SCHOOLS

To develop the school water rainwater harvesting modeller it is important to understand the different components and characteristics of a rainwater harvesting system. A typical rainwater harvesting system is shown in Fig 1 and consists of the following elements. A. This is the catchment surface, where rain is collected. As this surface can be dusty or have other debris it is usual to filter runoff, this is shown as B. Clean rainwater is then directed to rainwater harvesting tanks, shown as C. From the rainwater tanks, a distribution system then takes water to where it will be used. If the water will be used for drinking there may be further filtering, shown as D. Rainwater uses in and around buildings include irrigation, cleaning, and flushing toilets, shown as E.

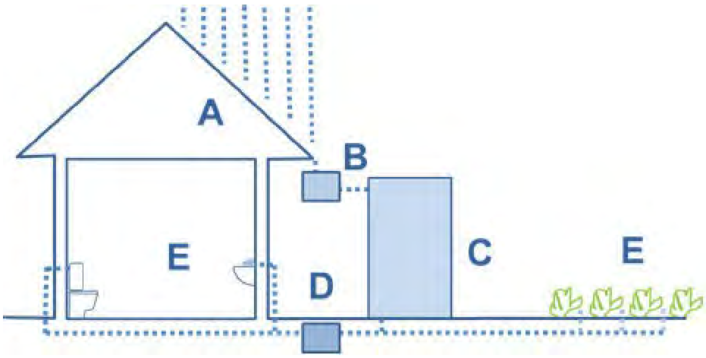


Fig. 1. A rainwater harvesting system

Calculating the amount of rainwater that can be captured off a hard surface requires data on rainfall patterns and an understanding of the physical characteristics of the hard surface. Simple rainwater harvesting calculators use readily available annual or monthly rainfall figures measured in mm/year or mm/month (Centre for Affordable Water and Sanitation Technology, 2011). More complex tools use daily rainfall throughout the year (Gibberd, 2020). However, daily rainfall data is more difficult to access.

The key attribute of collection surfaces that must be ascertained for a rainwater harvesting system is the runoff coefficient of the surfaces. The runoff coefficient is the percentage of precipitation that appears as runoff and is a result of the physiographic characteristics of the drainage area and is expressed as a constant between zero and one. Runoff coefficients for different roof types and surfaces are shown in Table 1.

Table 1. Runoff coefficients (Farreny *et al.*, 2020; Goel, 2011).

Roof type/surface	Runoff Coefficient
Sloping corrugated metal roof sheeting and tiled roofing	0.9
Flat concrete roofing with gravel topping	0.8
Level cement surfaces, such as driveways and tennis courts	0.8
Pavements and roads	0.70–0.95
Parks and pastures	0.05–0.30

Rainwater harvesting can be calculated for the collection surface using a simple calculation that multiplies the area of the collection surface, the volume of rain that falls on the surface and the runoff coefficient. This is shown in the example below for a 200m^2 corrugated iron roof in an area with an annual rainfall of 500mm.

Catchment area: 200m^2
Amount of rainfall: 500mm
Runoff coefficient: 0.9 (for a corrugated iron roof)

Rainwater supply (m3/year) = Rainfall (m/year) x Catchment Area (m2) x Runoff Coefficient
Rainwater supply (m3/year) = 0.5 x 200 x 0.9
Rainwater supply = 90.00m3 or 90,000 liters

3.1 Water consumption in schools

Water consumption in schools can be calculated by metering the volume of water supplied by water systems, for instance, a municipal water supply to a school. Water consumption can also be modelled by identifying all possible water uses in a school and then calculating the projected water usage.

There are many different water uses in schools including drinking, cleaning facilities, flushing toilets and irrigation. To model these, it is important to understand the equipment used to deliver water, how this equipment is operated and how often and how much water is used each time the equipment is used. This can be illustrated through a simple example.

Research indicates that female students use the toilet 4 times a day. If the water used by the toilet after each use is 8 litres, water consumption by toilets for 1 female student would be 4*8 or 32 litres a day.

3.2 Water balance

Rainwater harvesting systems can be designed to supplement an existing water supply and therefore reduce the pressure on this supply. Alternatively, it may be designed to provide for all the water needs of a school. This means that the rainwater harvesting system would enable the school to be ‘’off-grid’’ for water. One of the most important aspects of designing a rainwater harvesting system is understanding which of these need to be achieved and then designing the system for this. The balance between the ‘’production’’ of water from rainwater harvesting and the ‘’consumption’ of water within the school over a school year is an important aspect of this.

3.3 Specifications for a School Water and Rainwater Use Modeler

Based on an understanding of rainwater harvesting systems, a specification for a School Water and Rainwater Use Modeler (SWARUM) was developed. This is outlined below.

The modeller must achieve the following objectives:

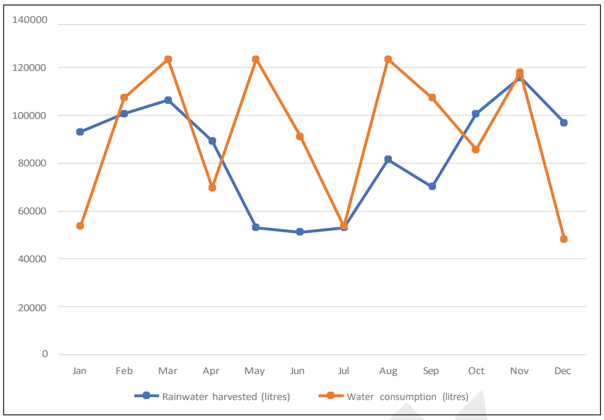
1. It must be very simple to use and be readily understandable by teachers, students and school governing bodies
2. Data required in the modeller must be easily accessible and easy to input.
3. The modeller should enable users to model:
    a) A simple rainwater harvesting system at a school and understanding the patterns of rainwater production over a year.
    b) How different factors within the water and rainwater harvesting system can be manipulated to improve performance.

Based on this specification, a School Water Use Rainwater Modeler (SWARUM) was developed and is presented next.

4. SCHOOL WATER AND RAINWATER USE MODELER

The School Water Use Rainwater Modeler (SWARUM) has three parts: the input section, the table report and the graphic report, and is shown in Fig 2. To use the modeller, inputs are required in the light blue areas, as explained below.

School Water and Rainwater Use Modeller		
School name		
Address		
Female occupants	105	number
Male occupants	105	number
School year	206	days
Drinking water per occupant		litres
Washing water per occupant		litres
Water consumption per WC flush		litres
Water consumption per urinal flush		litres
Collection surface area for rainwater harvesting	2106	m2
Collection surface runoff factor	0.9	
Water consumption per occupant per year	5,253	litres/yr
Water consumption per occupant per day	26	litres/day/person
Water consumption per occupant per day		
Rainwater harvested per collection surface m2	480	litres/year
Volume of water deficit (figures in orange)	-214,922	litres/year
Percentage of water needs met from rainwater	84	%



Month	Rainwater harvesting				Balance	School water consumption							
Month	Rainfall (mm/month)	Collection surface (m2)	Runoff factor	Rainwater harvested (litres)	Surplus/deficit (litres)	Consumption (litres)	Days (premises occupied)	Female (staff and students)	Male (staff and students)	Drinking (litres per person)	Washing (litres per person)	WC (litres per flush)	Urinal (litres per flush)
Jan	49	2106	0.9	**92,875**	39,325	**53,550**	10	105	105	3	6	6	2
Feb	53	2106	0.9	**100,456**	-6,644	**107,100**	20	105	105	3	6	6	2
Mar	56	2106	0.9	**106,142**	-17,023	**123,165**	23	105	105	3	6	6	2
Apr	47	2106	0.9	**89,084**	19,469	**69,615**	13	105	105	3	6	6	2
May	28	2106	0.9	**53,071**	-70,094	**123,165**	23	105	105	3	6	6	2
Jun	27	2106	0.9	**51,176**	-39,859	**91,035**	17	105	105	3	6	6	2
Jul	28	2106	0.9	**53,071**	-479	**53,550**	10	105	105	3	6	6	2
Aug	43	2106	0.9	**81,502**	-41,663	**123,165**	23	105	105	3	6	6	2
Sep	37	2106	0.9	**70,130**	-36,970	**107,100**	20	105	105	3	6	6	2
Oct	53	2106	0.9	**100,456**	14,776	**85,680**	16	105	105	3	6	6	2
Nov	61	2106	0.9	**115,619**	-2,191	**117,810**	22	105	105	3	6	6	2
Dec	51	2106	0.9	**96,665**	48,470	**48,195**	9	105	105	3	6	6	2
	533			**1,010,248**	**-92,882**	**1,103,130**	**206**						

Fig. 1. School rainwater harvesting modeller

In the input section of the modeller, in the top left-hand corner, data on the school is entered. Outputs from the modellers, such as the percentage of water needs met by rainwater harvesting, are also provided, as indicated below.

- **School name:** The name of the school is entered here.
- **School address:** The address of the school is entered here.
- **The number of female and male occupants:** The number of female and male occupants at the school is entered. This includes students, teachers and school staff, such as administrators.
- **School year:** This is the number of days the school is occupied by students and staff over a year. This is the sum of the days occupied each month which is shown in the table section (see below)**.**
- **Drinking water per occupant:** The amount of drinking water used per occupant is indicated here. This can be based on studies at the school or a norm, such as 2 litres per day per person (Meinders and Meinders, 2010).
- **Cleaning and washing water per occupant:** The amount of cleaning and washing water used per occupant is indicated here. This can be based on studies at the school and would include washing hands after using the toilet and meals, cleaning cooking and eating utensils, as well as building cleaning such as mopping floors. A guideline of 2-5 litres in water-restricted areas can be used.
- **Water consumption per WC flush: T**he amount of water each time the toilet at the school is flushed is indicated here. Flush rates can be obtained from the manufacturer or physically measured by measuring the amount of water required to fill a toilet cistern after a flush. Flush rates vary between 9 and 3 litres per flush, with most modern toilets having a flush rate of 6 litres per flush (Aurelien et al., 2013)
- **Water consumption per urinal flush:** This can be obtained from the manufacturer or measured by placing a container under the flushing mechanism and measuring the amount of water per flush. Urinal flushes vary between 1 and 2 litres per flush (Aurelien et al., 2013)

- **Rainwater collection surface**: The rainwater collection surface is the area used to collect rainwater. This may include roof surfaces and hard surfaces such as tennis and netball courts.
- **Runoff factor of rainwater collection surface:** The runoff factor relates to the type of material of the collection surface and can be read in Table 1.
- **Water consumption per occupant per year:** This provides the amount of water consumed at the school by each occupant over a year. It is calculated by dividing the total amount of water consumed by the school divided by the number of occupants.
- **Water consumption per occupant per day:** This provides the average amount of water consumed at the school by each occupant per school day. It is obtained by dividing water consumption at the school by the number of occupants and the number of school days.
- **Rainwater harvested per collection surface m^2 :** This provides the amount of rainwater captured by each square metre of the collection surface.
- **The volume of water deficit (figures in orange):** This provides water volume of the water deficit which is the volume of water consumed minus the volume of water captured and indicates the amount of water that will be needed to operate the school over the amount harvested from rain.
- **Percentage of water needs met from rainwater:** This indicates the percentage of water consumed at the school that has been harvested from rain.

The table at the bottom of the modeller presents input and output data. Areas in light blue require data to be entered, while other data is generated by the modeller. Monthly rainfall for the site should be entered in the light blue column on the left. This data is readily available for most sites (World Climate, 2022). The light blue column on the right requires the days the school is occupied by month to be entered.

Data in the columns on the left are used in the modeller to calculate the rainwater harvested per month using the following equation:

Rainfall*area of collection surface* runoff factor

Data in the columns to the right are used in the modeller to calculate water consumption at the school using the following equation:

Female occupants*toilet use per day*WC*flush rates + Male occupants* toilet use per day*WC flush rates + Male occupants* urinal use per day* urinal flush rate + Occupants*drinking water per occupant+ Occupants* Cleaning water per occupant.

The modeller is based on the assumption that users will use the toilets at school 4 times a day. Female users will use the toilet 4 times, while Male users will use the toilet once and the urinal 3 times.

In the coloured ‘Balance’ column in the middle, water consumption at the school is subtracted from the rainwater harvested to provide a negative or positive balance. This balance indicates whether the consumption of water is above or below the volume of water harvested that month. Positive balances are indicated in green and negative balances are indicated in orange.

The graph on the top right corner of the modeller indicates volumes of water harvested per month (in blue) and volumes of water consumed per month (in orange). This indicates whether water consumption is above or below the volumes of rainwater harvested each month.

5. CASE STUDY

The case study school is near Loerie in the Eastern Cape, in South Africa. The area has experienced severe droughts and water rationing over the last 5 years. Figure 2 shows photographs of the school indicating the large roof and hard surface areas available as collection surfaces.

Fig. 2. Photographs of the case study school.

Figure 3 shows a plan of the school with the main collection surfaces shown in dark grey (roofs). The roofs of the building are corrugated iron and can be used as collection surfaces. From Table 1 the collection surfaces have a runoff coefficient of 0.9. The collection surface area available for rainwater harvesting is 2,160 m^2 on a school site of 67,500m^2.

Fig. 3. A plan of the school indicating the site and the roof collection surface.

Fig. 4 shows rainfall patterns for the case study site over a year. This indicates that most rain falls in the summer months (November – March) when there is between 70 and 80mm per month. This decreases between April and October when rainfall is between 60 and 70mm. This indicates that rainfall is fairly regular throughout the year and there are no long dry periods.

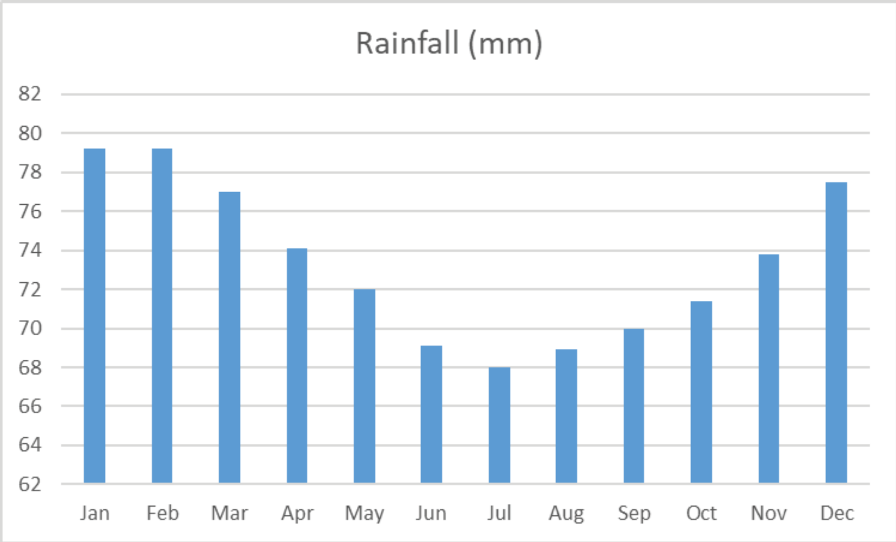


Fig. 4. Monthly rainfall in Loerie, Eastern Cape.

The school has 202 learners and 8 full-time staff equivalents and therefore has 210 occupants on site. As gender ratios vary over time, for this exercise the number of male and female occupants were made equal at 105 female and 105 male. The number of days the school is occupied is entered per month and is determined by the school calendar which remains broadly the same from one year to the next.

5.1 Application of the School Water Use and Rainwater Model to the Case Study

To test the modeller, it is used to model the following interventions at the school:

- Intervention 1: Use the school roofs for rainwater harvesting.
- Intervention 2: Use school roofs for rainwater harvesting, and reduce water consumption through increased efficiency including reducing washing water per occupant from 6 to 4 litres and reducing WC flush rates from 6 to 4 litres and urinal flush rates from 2 to 1 litre.
- Intervention 3: Use school roofs for rainwater harvesting, reduce water consumption through more efficient fittings, and increase collection area from 2,106 to 3,106m^2 by including yard hard surfaces.
- Intervention 4: Use school roofs and hard surfaces for rainwater harvesting, reduce water consumption through more efficient fittings, and install waterless sanitation. This means that all water associated with water-borne sanitation is reduced to zero.

The results of this application are shown in Fig 5. Results in Intervention 1 show that water consumption at the school generally exceeded the amount of rainwater harvested. This indicates that the rainwater harvesting system could meet about 84% of the school's water requirements.

Results for Intervention 2, indicate that water consumption has dropped at the school resulting in more months where rainwater harvested water exceeds water consumed. This results in 95% of the school's water needs being met from rainwater harvesting.

Results for Intervention 3, indicate that rainwater harvesting exceeds the amount required at the school and that 100% of the school's needs are met. An exception is in May when consumption of water is equal to the amount of rainwater harvested.

Results for Intervention 4 indicate that rainwater harvesting far exceeds the amount of water required at the school and that the school could operate on 100% rainwater harvesting

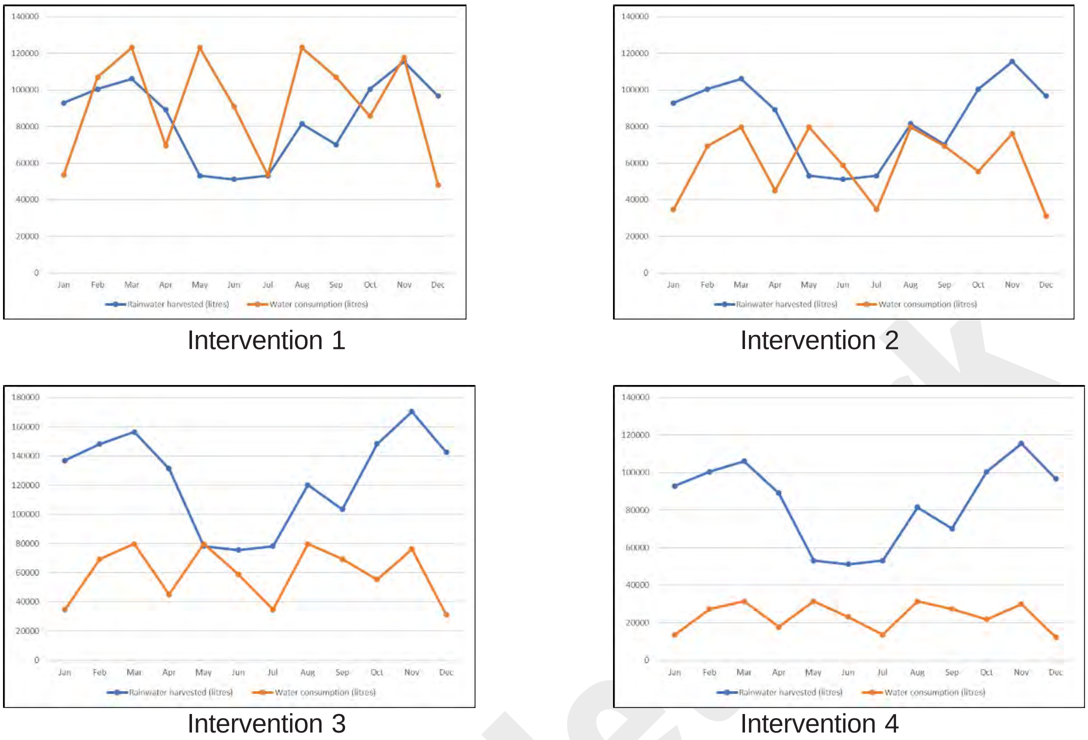


Fig 5. Rainfall harvested, water used at the school under conventional conditions and the difference (author).

6. DISCUSSION

A review of the results in Fig. 5 indicates that the modeller provides useful insight into water use at the school. Patterns of water use per month relate directly to the number of days the school premises were occupied. The school premises are occupied for 23 days in May and August and as a result, water consumption is highest for these months and the rainwater harvesting system is not able to match consumption in Intervention 1. However, during months when the occupation is lower, such as April (13 days) and July (10 days), rainwater harvesting exceeds water consumption (see Intervention 1, Fig. 5).

This finding would be interesting for a school because it highlights the possibility of matching occupancy with rainwater harvesting patterns. Thus, there could be more days of school during months of high rainwater harvesting volume and reduced occupancy during drier months. This alignment of water consumption to rainwater harvesting production would reduce reliance on external water supplies and improve the school's water resilience.

The results also show that a simple rainwater harvesting system based only on school building roofs (Intervention 1) and improved efficiency (Intervention 2) may not enable the school to be off-grid and wholly rely on rainwater harvesting for their needs. The results however do show that increasing the collection surface of the rainwater harvesting systems (Intervention 3) is likely to enable the system to meet all the school's needs. The results also show that using a dry sanitation system makes a very significant reduction in water consumption at the school and that combined with a simple roof-based rainwater system there is sufficient water to meet the school's needs throughout the year (Intervention 4).

These types of results enable schools to understand their water use and the potential of a rainwater harvesting system. The modeller demonstrates that using a simple rainwater system and improving efficiency could reduce their reliance on external water sources, such as municipal water, by up to 90%. It also indicates that should the school wish to be off-grid for water, more radical changes such as the use of external hard surfaces for rainwater collection and installing dry sanitation will be required.

The findings confirm that the modeller is easy to use and enables the implications of different interventions to be readily ascertained. However, it also shows that the modeller may not provide an accurate basis for the detailed design of a rainwater harvesting system. This is due to the following. Firstly, only one flush rate for toilets and urinals is provided. Schools with many toilets may have toilets and urinals with different flush rates and this diversity is not captured. Secondly, the modeller only allows for one collection surface runoff factor to be entered. Schools may have different collection surfaces with different runoff factors. Thirdly, schools may have other water uses, such as irrigation, that currently are not included in the modeller. Fourthly, the ‘lag’ effect of large rainwater tanks is not considered. Thus, water harvested, but not consumed entirely in a month, such as during January, is not reflected as a ‘’credit’’ to the following month, as would happen in an actual rainwater harvesting system. Fifthly, the modeller assumes regular rainwater patterns with limited variation between years. Data from climate change projections indicate that rainwater patterns are likely to change and therefore this should be considered (Maúre et al., 2018).

A review of the original specification of the modeller indicates that the SWARUM appears to meet most of the defined requirements. The tool is easy to use and understand, the data used by the modeller is easy to access, and the modeller provides an indication of water use and rainwater harvesting over a year and enables the implications of interventions to the water and rainwater harvesting system to be understood.

The modeller can therefore be said to support increased knowledge and understanding and contribute to “responsible decision making” by the school governing body and staff (Williamson, 2010). Achieving a high degree of accuracy was not part of the specification and the complexity associated with achieving this is likely to have made the tool highly complex and difficult to use (Borgstein, et al., 2016). However, Williamson (2010), notes that while achieving a high degree of accuracy in modelling tools may not be necessary, tools should provide an indication of the levels of accuracy achieved, so this is understood. This recommendation should be incorporated into the SWARUM.

7. CONCLUSIONS AND RECOMMENDATIONS

The study develops a school water use and rainwater harvesting modeller and applies this to a case study school in a drought-stricken area of South Africa. The specification for the modeller indicates that school staff must be able to use this to understand the potential of rainwater harvesting and enables interventions to improve water resilience to be modelled. Applying the modeller to the case study school indicates that it can generate graphs of water consumption and rainwater harvesting over a year. It also shows that the modeller enables the implications of interventions such as more efficient use of water, increasing rainwater collection surfaces and the use of dry sanitation to be ascertained.

The modeller, therefore, shows some potential to support increased awareness and enable decision-making about water and rainwater harvesting systems by schools. This could make a valuable contribution to improving water resilience at schools. A recommendation, therefore, is to develop the modeller and test it with school decision makers, such as school management teams and governing bodies. The simple manual and notes on the modeller should indicate its role as means of supporting decision-making rather than as a design tool. These notes should also indicate the accuracy of the tool.

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An elementary review of wave energy potential at Mauritius Island

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ABSTRACT
During the last decade renewable energy sector has attracted a significant interest from a range of stakeholders. Ocean Renewables in specific are an attractive solution for covering countries' high energy demand with low or none environmental impacts. Within this context wave energy power generation potential around small islands has extendedly been investigated as well. Following that a review has been carried out in here, to compile existing research findings on wave energy potential, in specific around Mauritius Island. The island is geographically favored for ocean energy extraction in the context of energy extraction from offshore winds, waves and currents, and ocean thermal and saline energy. Mauritius relies on fossil fuels to cover its energy needs. But the island has set a target to be able to cover its electricity needs by utilizing 35% from renewable energy sources by 2025. From the reviewed literature and a range of secondary calculations two sites come through as highly favorable locations for WECs (Wave Energy Converters) installation. Findings highlight that the wave source itself is abundant stressing out the need for further research to understand when and if WECs at the Mauritian sites could be fully commercialized in the future. For these sites and from the point of better understanding the spatial distribution of wave energy resource, high resolution wave transformation and hydro-morphodynamic numerical modelling is further suggested. Numerical modelling can support exercises to identify precise locations for WECs deployment along the coastline, prior to reaching any commercialization.

Keywords: Mauritius Island, Wave Energy Converter (WEC), Ocean Wave Energy, Renewable Energy, Small Island

1. INTRODUCTION

Mauritius is an island country located near the Inter Tropical Convergence Zone (ITCZ), in the south-western Indian Ocean, off the eastern coast of Africa (Figure 1). It is a Small Island Developing State (SIDS) that includes an Exclusive Economic Zone (EEZ) of about 2.3 million km^2, extending over the islands of Rodrigues, Agalega, Cargos Carajos Shoals, Chagos Archipelago and Tromelin. Currently, the island heavily relies on fossil fuels to cover its energy needs. However, with fossil fuels prices fluctuating in the international market, the island becomes vulnerable with regards to covering its electricity needs. Following that Mauritius has set a target to be able to cover its electricity needs by utilizing 35% from renewable energy sources by 2025 (Mawooa et al., 2018).

According to the Mauritius government, the island is geographically favoured for ocean energy extraction in the context of energy extraction from offshore winds, waves and currents, and ocean thermal and saline energy. Research has taken place during the last few years to explore in specific the high potential of harnessing ocean wave energy around Mauritius Island. Following that this paper has reviewed and presents in here the relevant research outputs on Mauritian wave energy potential, focusing specifically at six offshore sites (Figure 1). The six sites were initially deemed highly favourable for harnessing ocean wave energy. This study further utilised oceanographic secondary data at the six sites to determine the potential annual wave power output of deploying Generic Wave Energy Converter (WEC)

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device at each site, at varying device efficiency levels. For each site, the percentage that the single WEC power output could represent in the annual Mauritian energy mix was further determined at varying device efficiency levels.

Fig. 1. Location of Mauritius Island (Upper Figure). Test sites reviewed as potentially favourable for ocean wave power exploitation around Mauritius Island (Lower Figure). (Source: "Mauritius Island". 51°53'01.7"N and 1°26'19.0"E. Google Earth. October 4, 2013. July 4, 2022).

2. METHODOLOGY

Mauritius is exposed all year around to waves generated by westerly and south-easterly winds. The eastern part of the island is exposed to most energetic wind and wave patterns. The western part at the lee side of the island and in specific the south western part is more protected from wave action. In open sea, wind generated wave periods range between 3-11 secs with wave heights that do not exceed 0.5 m during summer time and 3 m during winter time. In the south, south-west and south-east parts of the island wave heights vary between 1.5-2.5 m in summer and 2.5-3.5 m in winter. In the southernmost areas swell induced significant wave heights may reach up to 5 m with wave periods of up to 20 secs. Strong offshore ocean currents are generated during winter around Mauritius. The island is exposed to the South Equatorial Current (SEC) flowing towards west-east throughout the year. Ocean currents are further influenced by strong wind patterns that are induced by anticyclones in the South Indian Ocean parts, travelling from east to west. Ocean current speeds increase in magnitude through the channels found within the Mascarene Plateau, to form strong gyres in the leeside of the island. Once currents reach Madagascar, they split to travel northward and mix with the Agulhas current of the Mozambique Channel and the East Africa current and southward along the Madagascan coast, forming anticyclone gyre southwards (ASCLE, 2012; Doorga et al., 2018; JICA, 2015).

Two main studies have researched so far the wave energy potential around Mauritius at six different sites (Figure 1). The first study was published in 2017 and forms a collaboration between the Mauritius Oceanographic Institute (MOI), the University of Western Australia (UWA) and the Carnegie Clean Energy (CCE), that conducted a ~10-year wave climate and resource assessment for the sites of North East Bay, Blue Bay and Souillac around Mauritius (Figure 1) (UWA, 2017). In-situ buoy measurements and SWAN energy transformation models were used to assess the wave climate and simulate wave energy transformation and distribution patterns from the offshore towards the nearshore at the three sites. The second study was published in 2018 and investigated the wave energy potential at Tamarin, Flic-en-Flac and Roches Noires sites around Mauritius (Figure 1), by use of in-situ buoy measurements and subsequent analyses. From both studies the characteristics of the buoys network deployment (time period of deployment, buoys distance from shore, and buoys water depths) have been compiled and are presented in Table 1 below.

Table 1. Data buoys network at *Tamarin, Flic-en-Flac, Roches, North East, Blue Bay, Souillac Sites* at Mauritius Island. Information extracted from (UWA, 2017) and (Doorga et al., 2018)

Site	Time Period (years)	Buoy Distance from Shore (m)	Depth (m)
Tamarin	28/03/14-22/04/14	935	10
Flic-en-Flac	02/04/07-20/04/07	490	12
Roches Noires	09/02/12-10/11/14	1228	46
North East	01/01/12-31/12/14	1000	50
Blue Bay	01/07/09-31/12/15	1000	17
Souillac	01/07/16-01/03/17	1500-2000	40-50

The wave power output (KW/m) in deep waters can be calculated as follows (Doorga et al., 2018):

$$P_w = \frac{\rho g^2}{64\pi} H_{m0}^2 T_e$$

$$= 0.491\, H_{m0}^2 T_e \qquad \text{(Eq.1)}$$

Where H_{m0} is the significant wave height and T_e is the energy period. Equation (1) is valid for use at offshore points in water depths exceeding 10 m and distances from the shore exceeding 200 m. One approximation approach to calculate T_e is given by:

$$T_e = aT_p \qquad \text{(Eq.2)}$$

Where a is a coefficient that is determined from the shape of the wave spectrum and T_p represents the peak wave period. For elementary calculations carried out in here coefficient a was approximately taken equal to 0.86 for a Pierson–Moskowitz spectrum. However, it is important to note that the coefficient a may vary geographically and has been proved to range between 0.84-0.90, 0.92-0.98 and 0.79-0.85 for wind-sea, swell and total sea respectively (Seongho, 2021).

A reasonably accurate (±50 %), estimation of the annual energy production (AEP) from generic WEC can be further calculated by the following equation (Pecher and Kofoed, 2017):

$$AEP = Pw * width(absorber) * n_{w2w} * availability * hours\,(annual) \qquad \text{(Eq.3)}$$

Where Pw is the mean wave power (kW/m), $width(absorber)$ equals to the width (m) of the absorber, n_{w2w} is the overall wave-to-wire efficiency and $hours\ (annual)$ equal to the yearly production hours.

3. RESULTS AND DISCUSSION

The power output Pw (kW/m) was first calculated at each site by using the Hm0 significant wave height and Tp peak wave period taken from buoy measurements. Each pair of Hm0, Tp represents the highest joint probability occurrence between significant wave height (Hm0) and peak wave period (Tp) for the time period within which buoy measurements were taken per site. The full range of joint probability occurrences between Hm0 and Tp can be found in Mauritius Wave Energy Resource Assessment (Report by UWA, 2017) and Doorga et al. (2018). Results are presented at Table 2 below.

Table 2. Data buoys network measurements (wave characteristics) and Pw (Eq.1) calculations at *Tamarin, Flic-en-Flac, Roches, North East, Blue Bay, Souillac Sites* at Mauritius Island. Each pair of Hm0, Tp represents the highest joint probability occurrence between Hm0 and Tp for the time period buoy measurements (see Table 1) were taken per site. Information extracted from (UWA, 2017) and (Doorga et al., 2018)

Source: Mauritius Wave Energy Resource Assessment (Report by UWA, 2017)	**Site**	H_{m0} (m)	T_p (sec)	P_W (kW/m)
Cool season	*North East*	2.50	9.00	23.75
	Blue Bay	1.50	9.00	8.55
	Souillac	2.00	10.00	16.89
Warm season	*North East*	1.00	9.00	3.80
	Blue Bay	1.50	8.00	7.60
	Souillac	1.00	8.00	3.38

Source: Doorga et al. (2018)	**Site**	H_{m0} (m)	T_p (sec)	P_W (kW/m)
Warm season	*Tamarin*	0.34	5.81	0.28
	Flic-en-Flac	0.4	13.95	0.94
3-year representation	*Roches Noires*	5,5	18.60	237.58

As can be observed from Pw calculations of pairs Hm0, Tp that are found at the highest joint probability occurrence in Table 2, North East presents the highest wave energy potential of 23.75 kW/m during the cool season and Blue Bay presents the highest wave energy potential of 7.60 kW/m during the warm season. Roches Noires presents a significant wave energy potential of 237.58 kW/m during a 3-year representation from buoy data. Doorga et al. (2018) have further stressed that Roches Noires can be a favourable location for WECs installation with annual mean Pw equal to 29.7 kW/m, exceedingly significantly the value of 15kW/m that is regarded as a threshold for any site, in order to be regarded as a potential site for wave energy extraction.

Following that, from data, the mean wave power $Pw_{(mean)}$ was also determined. By further assuming the width of a generic absorber (WEC) equal to 15 m and for 95% availability, AEP has been calculated from Eq.3 at six different buoy sites around Mauritius (Figure 1), for values of n_{w2w} 20% and 50% respectively. The percentage that AEP at each site could represent in the Mauritian energy mix is finally indicated by dividing the AEP with the total electric energy consumption (TC) (2,800 GWh/year according to 2018 EIA estimation). Results are presented at Table 3 below.

Table 3. Mean Pw (kW/m), AEP (GWh/year), and ratio of AEP to total electric consumption (TC) calculations *(2,800 GWh/year according to 2018 EIA estimation)***at** ***Tamarin, Flic-en-Flac, Roches, North East, Blue Bay, Souillac Sites*** **at Mauritius Island, by assuming the width of a generic absorber (WEC) equal to 15 m, for 95% availability and n_{w2w} 20% and 50% respectively.**

Site	*Mean Pw (kW/m)*	*AEP (GWh/year)* n_{w2w}= 20 %	*AEP (GWh/year)* n_{w2w}= 50%	AEP/TC n_{w2w}= 20 %	AEP/TC n_{w2w}= 20 %
North East	17.00	424.71	1061.78	0.15	0.38
Blue Bay	17.00	424.71	1061.78	0.15	0.38
Souillac	20.00	499.66	1249.15	0.18	0.45
Tamarin	0.10	2.498	6.24	0.00089	0.0022
Flic-en-Flac	0.96	23.98	59.95	0.008566	0.021
Roches Noires	29.7	742.00	1854.99	0.27	0.66

Table 3 results provide a useful insight on the potential that could the different sites bring in supplying wave energy resource, to cover significant percentage of the Mauritius electricity needs, when compared against conventional sources. For example Roches Noires presents the highest AEP/TC ratios of 0.27% and 0.66% for n_{w2w}= 20% and n_{w2w}= 50% accordingly. Of course it is recognized that there are yet limitations that do not allow WECs to reach full commercialization (Doorga et al., 2018). But the wave source itself is abundant highlighting the need for further research to understand when and if WECs at these sites could be fully commercialized in the near future, combined with successfully implemented governmental measures (Cramer et al., n.d.). More information on successfully implemented governmental measures case studies regarding ocean renewables can be found in (Cramer et al., n.d.). Suddhoo (2012) has also indicated as suitable offshore wave farm site the Southern Coast of Mauritius consisting of 20 km of shoreline between Souillac and Blue Bay, with average power output of 41.5 kW/m. This site has been deemed as most suitable as well for WECs deployment in UWA (2017), due to incident waves undergoing limited diffraction and refraction until they reach the location of deployment.

4. CONCLUSION

Various studies have recognized the huge potential that exists around Mauritius Island of producing clean energy from ocean waves. This paper provided a review of the research outputs so far on Mauritian wave energy potential, focusing specifically at six offshore sites (Tamarin, Flic-en-Flac, Roches, North East, Blue Bay, and Souillac). From reviewed literature, and by use of oceanographic secondary data, the potential annual wave power output of deploying Generic Wave Energy Converter (WEC) device was calculated at each site, at varying device efficiency levels. For each site, the percentage that WEC power output could represent in the annual Mauritian energy mix was further determined at the varying device efficiency levels. Results highlight that the wave source itself is abundant stressing out the need for further research to understand when and if WECs at these sites could be fully commercialized in the near future. From reviewed literature and secondary calculations Roches Noires and the Southern Coast of Mauritius come through as favorable locations for WECs installation. For these sites and from the point of better understanding the spatial distribution of wave energy resource, high resolution wave transformation and hydro-morphodynamic numerical modelling is further proposed. Numerical modelling will help further determine precise locations for deployment along the sections of coastline, prior to WEC deployment trials.

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#3. Sustainable transitions

Digitalisation in the water sector: Opportunities and challenges in the next decades

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ABSTRACT:

The water sector is undergoing rapid digitalisation providing new opportunities for improved, more efficient and economical services. Digitalisation may efficiently enable water utilities to overcome numerous challenges in the next decade by implementing, for example, real-time detection of water quality, process optimisation, efficient management of transport systems' rehabilitation, and reduction of utilities' physical and energy footprints. Many countries consider water supply and wastewater management critical services, and small islands are not an exception. Disruption of these services can lead to devastating consequences for the functioning of a society. In addition to natural catastrophes, the increasing number of man-made disruptions requires the earliest possible detection. The rapid digitalisation of the water sector has increased its vulnerability to cyberattacks, making it the third most attacked sector based on probability. Preparedness for immediate control of the attacks and the recovery actions after the incident is vital and should be planned. When a utility is under attack, identification is also critical, as man-made attacks can last several hours or days before being identified. This paper presents the emerging opportunities of digitalisation and the challenges associated with cyberattacks. Examples requiring increased preparedness, efficient detection, and the use of digital tools to minimise the impacts are outlined.

Keywords: water utilities, digitalization, remote control, cyber risks

1. INTRODUCTION: CHALLENGES FOR THE NEXT DECADES

The water sector is facing several challenges, independently of the size or wealth of a nation. Several reviews and think-tanks agree on the main challenges to be faced in the next decades (EurEau, 2020; ICMA, 2018; IWA, 2020; World Bank Blogs, 2020): a growing population requiring increased capacities of water supply and wastewater management systems, the lack of water and land for the expansion of facilities in an urban landscape, improved legal framework requiring improved removal of conventional pollutants and emerging pollutants, and the need for optimal, economical and efficient operation and treatment. The digitalisation of the water sector may provide efficient and economical solutions for some of these challenges addressed in this paper.

2. METHODOLOGY

This mini-review is based on a literature search in various scientific databases, own research and publications and communications with utility owners.

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3. DISCUSSION ON STATUS, PROSPECTS AND CHALLENGES

3.1 Status and prospects of digitalisation in the water sector

Many industrial leaders and economists agree that the digitalisation of the world is no longer an option, but a necessity to respond to the emerging needs, expectations and maximising benefits (Effective Managers, 2020). Products and services related to our daily life are rapidly embracing the digitalisation trends, leading to their common access that a decade ago was only a dream. The world's leading economies and service providers have identified digital transformation as a strategic business opportunity emerging with innovations (Boikova et al., 2021; Lee & Falahat, 2019).

In this vein, the water sector is not an exception. As emphasised in an interview with IWA, embracing digital tools and concepts in the water sector is crucial (Jimenez, 2018). Nevertheless, it still lags behind other sectors (Cabrera, 2020).

Creative innovators produce time and resource-saving concepts and products that would improve the services in the water sector. The IWA's white paper on digitalisation (IWA and Xylem, 2019) presents a comprehensive overview of its potential benefits for the water industry. However, the water sector has a slow uptake of innovations (Dyson, 2019a; Kiparsky et al., 2013; Speight, 2015). Conservative attitudes among the decision makers of the utilities, the slow adoption of technological innovations from other industries and lengthy project timelines make the water industry lack in the implementation of innovative technologies. Products of other industries become obsolete before the water industry even completes a regular project timeline (Dyson, 2019b).

The role of digitalisation has become more evident in the post-pandemic world (The Economic Times, 2022). COVID-19 has forced many utilities to involuntarily engage in digitalisation. In the increasing absence of operation staff due to COVID-19, utilities had no option but to rapidly initiate remote surveillance and control (Water & Wastes Digest, 2021). In many cases, this was an unplanned transformation without much training. On the other hand, this urgency has forced many utilities to activate remote process control without considering all the risks and consequences (Water & Wastes Digest, 2021).

The International Finance Corporation has noted that, especially after COVID-19, experiences of crisis preparedness and the resiliency of staff, systems and equipment could lead to increased investments in digital solutions (IFC-World Bank, 2020).

While digitalisation provides a huge potential for improving the water industry services, it also increases their vulnerability to disastrous conditions, adding up to the increasing occurrence of natural catastrophes. Increasing awareness of both opportunities and challenges is, thus, essential to ensure safe and uninterrupted water services for the population, being also the objective of the UN's Sustainable Development Goal 6 (UNEP, 2020).

3.2 Digitalisation-originating opportunities in the water sector

Several think tanks have developed material focusing on the advantages and opportunities of digitalisation in the water sector. IWA's handbook on digitalisation (IWA and Xylem, 2019) provides a good overview of the potential opportunities.

Digitalisation creates opportunities provided to water sources and the environment, for instance, surveillance and predictions through supervisory control and data acquisition (SCADA) and telemetry, hydraulic models, models for the management of water resources and decision-making tools (IWA and Xylem, 2019). During collection, distribution, and transport of water and wastewater, it can provide comprehensive surveillance, process control, operation and maintenance optimisation, investment (rehabilitation and upgrading) planning, etc. The new generation of end customers is involved and expects much more than earlier generations; this can be illustrated, for instance, with the customer engagement and information systems, metring and billing.

Digital workforces and access to specialised resources in cost-efficient ways are other benefits. In a scenario where the utilities must improve their operational economies considering the increasing salary costs, digitalisation provides unique possibilities. For example, in Norway, it is common to have no technical personnel at water utilities from 3:00 p.m. on Fridays until 07:00 a.m. on Mondays. The responsible personnel has remote access to the SCADA systems and maintains surveillance and control from home, if and when needed (Simenson & Søraker, 2021). Many small and medium-sized utilities often cannot afford in-house specialists and are dependent on external consultancy and service organisations to cover their needs. Digital tools make these activities much faster, accessible and cost-efficient for the service providers and the utilities. These are also especially useful for utilities on small islands.

3.3 Surveillance of water quality and operational security

Compared with conventional off-line monitoring, real-time monitoring provides an array of advantages. The online systems are often cheaper due to the reduced manpower required for their operation, more accurate due to fewer human errors, and provide information used to identify operational failures and immediate operational improvements induced by errors or malfunctioning (Paepae et al., 2021). The conventional laboratory analysis is often time-consuming and cannot be used in the immediate management of process failures.

Gustaf Olsson, almost five decades ago (Olsson, 1974) elaborated on smart process control in wastewater treatment plants, discussing the role of process control and sensors. Twenty-five years ago, predictions on how the treatment plants of the year 2025 would be controlled and automated were presented (Olsson & Newell, 1999) painting an accurate picture of the current situation. Nowadays, processes in the water sector are using instrumentation, control and automation tools to become smarter and more efficient. Relevant conferences organised biennially by IWA bring specialists, innovators, practitioners and end users together (IWA, 2022).

Although water quality sensors were already available several decades ago, they were not widely used due to the exorbitant costs of acquisition and the technical challenges of installation, maintenance and data transfers (Paepae et al., 2021). With the rapid digitalisation of this sector, the sensor, surveillance and control systems are becoming more and more affordable, resulting in their wide use (Dey et al., 2018; Marques & Pitarma, 2020; Pasika & Gandla, 2020). Digital tools are also becoming more common in the water industry, for example, in process surveillance and control of the coagulation process (Ratnaweera & Fettig, 2015). The drastic reduction in the cost of online sensors and communication tools makes them more relevant for small utilities and utilities on small islands (Mamun et al., 2019).

However, not all critical water quality parameters, such as phosphates, can be measured using physical sensors due to technological unavailability (Ratnaweera & Fettig, 2015). In such cases, automated analysers, often based on flow injection analyses, are used. Such analysers are expensive both to acquire and operate. A new range of sensors based on advanced algorithms has provided a solution, where tens of parameters can be measured using a single probe ((GO Systemelektronik, 2020; S::CAN, 2020). They are based on UV-visible spectral scanning and are used by water authorities, although they can be expensive for many utilities.

A new generation of sensors based on hybrid concepts using cheap physical sensors and historical data has been developed, with successful application in wastewater surveillance and process control (Nair, Hykkerud, et al., 2022), as well as automated faulty detection, (Nair, Weitzel, et al., 2022). The hybrid sensors are cost-efficient, which can be useful for small-island and small utilities.

3.4 Potential risks of digitalization

With the increase in digitalisation of the water sector, more and more utilities are turning towards automation of process surveillance and control. As noted earlier, during the COVID-19 period, this process was implemented in most utilities (probably too soon) without adequate tools and trained staff.

Most online sensors require periodic cleaning and calibration, and they might not work on a daily basis, at all times. Thus, it is necessary to establish good routines of maintenance and systems that detect measurement failures (Nair, Weitzel, et al., 2022). Inadequate control of online sensors may give false security to the operators and disrupt not only the downstream processes but also the final water quality.

However, the biggest threat in digitalisation is the increased vulnerability of operations to cyber threats (AWWA, 2019). Cyber threats have a history of more than two decades, while their frequency and subsequent damage have increased over the years. One of the first reported cyberattacks was against a wastewater treatment plant in the Shire of Maroochy (Australia) in 2000. Two incidents in San Francisco and Florida and one in Norway are the latest reported attacks in 2021 (Boubaker, 2021), keeping in mind that there could be other unreported incidents. Many water utilities are not only unprepared for cyberattacks but also unaware of the potential threats. A cybersecurity expert has warned that over 50,000 water supply utilities in the USA may be under cyber threats (Collier, 2021).

Cyber risks include accidentally or intentionally mishandling confidential data, disturbing the operation of servers and network infrastructure, connecting infected media, accessing malicious email or websites, exploiting vulnerabilities in devices and networks, using social engineering techniques to gain access to internal resources and valuable sensitive information, among others. The consequences could be critical to human health, the environment and society.

Increasing the awareness of utility owners concerning cyberattacks' potential is important. Utilities should have systems that prevent and timely identify attacks, as well as be prepared for immediate actions to minimise the potential damage. Various authorities have published guidelines, which are valuable in preparing utilities for this threat (Ampcus Cyber, 2022; EU Network and Information Security Directive, 2016; Govt of UK, 2017).

While digitalisation tools and processes create many opportunities for smaller utilities and water utilities in small islands, it is important to invest in creating awareness, establishing prevention and detection systems, and improving preparedness protocols.

4. CONCLUSIONS

Water utilities will face many challenges in the next decades, such as the increasing population and urbanisation, water and land scarcity, and the requirement for better treatment efficiency and a more economical operation. Digitalisation in the water sector and utilities may provide efficient solutions to these challenges, especially in the case of small utilities and utilities on small islands, accounting for their limited resources.

However, digitalisation is also increasing the vulnerability of the water sector due to the unawareness of and unpreparedness for cyber security threats. This has become obvious after the unplanned initiation of remote process surveillance and control in various utilities during the

COVID-19 pandemic. It is vital to focus on and strengthen these aspects to avoid or reduce these incidents and their impacts.

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COMPETING INTERESTS

The author declares that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Potential implications for deployment of low carbon construction materials in the water industry

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Abstract

The introduction of sustainable and low-carbon construction materials into the built environment is one approach to reduce emissions of greenhouse gases to the atmosphere. By increasing the use of renewable energy and waste, new materials and processes can be incorporated into the materials supply chain. The present work relates to the mineralisation of anthropogenic C02 gas in construction materials, and their potential for use in water treatment, particularly in small island nations. We provide an overview of developments in this area of interest, with specific reference to the capture and use of point-source emissions, carbonate-able cementitious binders and waste in the manufacture of construction aggregates/monolithic products, including concrete and potential filter media. As the amount of construction materials used in new and existing water-related infrastructure is significant, there is potential for meaningful long-term carbon sequestration. In respect of this, we discuss the potential sustainability gains from replacing/reducing carbon intensive materials, quarried and crushed stone with low-carbon substitutes with particular reference to typical water supply and wastewater treatment facilities in the SE of England. Further, we estimate this potential more widely in order to gain a 'global' for carbon storage potential figure within fresh and waste water infrastructure.

Development of Sustainable Building Design in Hong Kong: Exploring Lean Capabilities

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ABSTRACT

When a building design fails to meet the end-users' requirements after construction, it is regarded as a faulty design. Faulty designs often lead to renovation, demolition, and material waste. The need to implement innovative tools and systems that continuously provide designers with the end-users' design requirements and feedback in the built environment cannot be ignored. This study explores the potentiality of implementing a Lean Premise Design (LPD) scheme in Hong Kong to facilitate sustainability practices, ensure energy conservation, promote innovative green technologies and water efficiency, and reduce abortive works in high-rise residential (HRR) buildings. A comprehensive review of literature on concepts similar to the LPD scheme and sustainability practices in the design and development of high-rise buildings was undertaken. In addition, interviews were adopted to validate the identified barriers and drivers to the LPD scheme. These facilitated the identification of perceived barriers to the LPD scheme adoption in the local context. Furthermore, the relevant drivers that can promote its implementation were examined. The study focused on sustainable building design relating to users' behaviour patterns and expectations, social needs, green maintenance technologies, and government initiatives. About 77% of the experts affirmed the availability of comprehensive building codes and guidelines. Nevertheless, 62% of the experts confirmed the insufficiency of the current regulations to promote sustainable building design. Similarly, the literature review revealed that while there are many sustainable concepts in the development of high-rise buildings, little or none of these concepts focused on LPD.

Keywords: High-rise buildings; Lean Premise Design; Residential buildings; Sustainability, Waste; Hong Kong.

1. INTRODUCTION

1.1 Background

The concept of sustainability has been gaining momentum for more than four decades [1]. The UN Conference on Human Environment in 1972 was the first major international discussion on sustainability at the global level [2]. Brundtland Report in 1989 included the integration of economic, social and environmental developments in sustainable development [3]. The World Summit on Sustainable Development in 2002 saw a major shift from the environment toward human and economic development. After the financial crisis in 2008, there was a shift in the paradigm to a "Green Economy", which incorporates renewable energy, green buildings, water and waste management [4]. Countries having succeeded in implementing green stimulus programs which are backed up with strong policy incentives and commercial frameworks to

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catalyze future green investments are benefiting both financial growth and environmental goals simultaneously [2,5].

The Hong Kong landscape is mostly built-up of high-rise buildings, including skyscrapers, which are in large-scale developments [6]. In 2019, the residential buildings in Hong Kong consumed 61,026 TJ of energy, with 42,937 TJ being electricity [7], while electricity generation accounted for about 66% of total carbon emissions [8]. In the same vein, 31.34 million tonnes of carbon dioxide were generated locally in 2020, contributing to about 0.09% of the world's greenhouse gases (GHG) [9]. As a result, it is important to select low-carbon materials and green construction technologies; however, only a small amount of renewable energy is produced within Hong Kong, which is mainly solar energy, wind energy, biogas, and biodiesel [7]. Therefore, it is necessary to promote renewable energy development and innovative green technologies.

1.2 Research aim and value

According to the recent policy addresses, the HKSAR Government is determined to accelerate the transformation of Hong Kong into a low-carbon city with a target of reducing carbon intensity by 70% of the 2005 level (at most by 2030) and intends to draw up a long-term decarbonization strategy in 2050. The HKSAR Government is also determined to apply commercially available green building technologies in public projects in the immediate years ahead. The Water Services Department is also promoting water conservation at home. The low-carbon transformation will take a multi-pronged approach like reducing waste, enhancing energy efficiency, calling for energy and natural resources conservation, and green building establishments. Public support to use less carbon-based materials or consider sustainable alternatives are necessary for reducing waste [10]. Benefits of climate mitigation in buildings include efficient homes and less wastage, while renovating buildings according to natural retrofit lifecycles is another avenue for energy-saving and low-carbon improvement [11].

The current paper explores the potential in the introduction of the concept of the Lean Premise Design (LPD) scheme to reduce abortive work in residential buildings. LPD means only basic furnishing, such as water and gas supply, waste discharge, sanitary fittings, windows, and the like, will be provided in new residential buildings. Internal finishing, built-in fixtures like kitchen and bathroom cabinets, cooking oven, and air-conditioners will, however, not be installed. The purchaser (potential end-users) can choose to buy a "lean premise" at a lower price or a fully furnished premise at a regular price. When fully developed, the LPD scheme is expected to impact construction cost positively and time as such works (interior finishing work, installation of built-in fixtures, and the like) can be subcontracted at different stages of construction. The study's deliverables will also contribute to conserving energy and natural resources, reducing construction and demolition (C&D) waste, and facilitating the development of a green economy.

2. LITERATURE REVIEW

2.1 Sustainability in Hong Kong: An overview

Hong Kong is a city well known for its high density [12]. Commercial and residential buildings are developed into high-rise establishments due to high land costs [6]. The construction of city buildings and infrastructure has resulted in large-scale consumption of materials and resources, leading to several degrees of environmental pollution during the construction and operational phases [13]. The exploding population and growing economies in major cities of the world have led to increasing urbanization globally as well as a continuous rise in population density in urban areas [14]. High-rise buildings were developed to cope with the dense population in metropolitan cities. Although construction projects can meet social needs and boost the Gross Domestic Product, the subsequent adverse effects on the environment bring much attention.

The rapidly growing world energy use has already raised concerns over supply difficulties, exhaustion of energy resources and negative environmental impacts [15]. Meanwhile, the advent of concepts such as sustainable development and green building has inspired the construction industry and stakeholders to contribute positively and proactively toward environmental protection [16,17].

Hong Kong is a metropolitan city characterized by high-rise developments [6]. Tall buildings are energy guzzlers consuming a large amount of energy in their construction and operation processes with resultant high carbon footprint and GHG emissions [18]; as well as contributing to global warming, energy resource depletion, and local and regional pollution in addition to adverse impacts on natural habitats [19]. There is a need for more application and implementation of sustainable practices as early as the building design stage to incorporate the needs and perceptions of the potential end-users.

2.2 Building Maintenance

Undeniably, maintenance and improvement work of residential buildings consumes natural resources and non-renewable energy while producing a large amount of demolition waste and carbon dioxide to the environment. More than 50% of the latent defects are caused by faulty design [20]. Currently, there is no (effective) feedback system that allows building owners and management staff to reflect end-users' requirements and maintenance problems to building designers (architects and engineers), leading to a reiteration of faulty design. Research efforts were made to bridge this gap by developing relevant End-users-Oriented Design (EOD) principles for sustainable high-rise residential development in the form of a cloud-based EOD platform which will facilitate proper appraisal of project designs, engaging stakeholders as early as the planning and design stage, gathering of post-occupancy feedback, and incorporating necessary feedbacks into future design. Moreover, guidelines of EOD for residential buildings will be formulated to assist building designers to reduce faulty design, thus facilitating sustainable building development.

2.3 Impacts of Faulty Design

Maintenance carries the function of repairing as well as restoring a building towards ensuring that the building element or amenity can perform to the prescribed standard [21]. Errors and defects occurred during the design stage, resulting in high maintenance costs for labour, materials, and energy resources [22,23]. Dismantling the internal finishing and fittings will release the embodied carbon content and overload landfills [24]. Inadequate information, unawareness, wrong assumption and the lack of knowledge contribute to latent defects at the design stage [25]. Fifty-eight per cent of the defects originated from faulty design, of which misjudgment of users' intended use is an important factor [20,22]. Defects due to faulty design will appear during the occupancy stage. The faulty design generally causes more latent defects than workmanship generating many maintenance problems [26]. Currently, there is little or no communication between end-users and building designers globally, especially in Hong Kong, where the preference of end-users for high-rise residential buildings is not considered in the choice of finishing and built-in fittings for their new homes, resulting in frequent abortive work, which ends up with a large amount of C&D waste in most cases.

2.4 Scope and Justification

Several studies in recent literature have investigated the issues regarding sustainability applications in buildings – such as Ref. [27], which examines the economic benefits of green buildings, and Ref. [28], which highlights how sustainability affects building buildability. Other studies have developed sustainability assessment systems for new and existing buildings [29,30]. However, little research has addressed the environmental problems generated from abortive work due to the mismatch between building design and end-users' requirements. Since social issues are important components of sustainable development [31], the relationship between advances in sustainable technologies and the impacts on behaviours of end-users should be studied [32]. Literature on how green maintenance can contribute to economic growth is also limited. Hence, the proposed research will focus on sustainable building design relating to users' behaviour patterns and expectations, social needs, green maintenance technologies, and government initiatives. This paper reports the preliminary findings of the potentials, barriers and drivers of the LPD scheme in the development of sustainable high-rise buildings, and its capabilities to reduce waste and conserve water usage in households.

3. METHODOLOGY

A qualitative research method was adopted to achieve the study aim, which commenced with a comprehensive review of the relevant literature via systematic review and content analysis of the literature and practices to identify concepts similar to the LPD scheme being developed for Hong Kong in this project [33]. In-depth reviews were also conducted in areas such as flexible building design, end-user design preferences, building maintenance and operation, and how Lean principles can enhance the implementation of sustainable practices in HRR buildings and reduce abortive work. Meanwhile, interviews with project stakeholders of HRR buildings were conducted to collect their interpretations of standard building provisions, expectations on ease of maintenance and facilitating measures to promote LPD, and their acceptance of energy/natural resources saving design/appliances.

The relevant findings from the desktop literature review and preliminary interviews were consolidated for further validation via expert interviews to assess how stakeholders in Hong Kong's built environment perceive the LPD scheme to enhance sustainable development in the region. The perceived barriers to the LPD scheme adoption in the local context were also examined as well as the relevant drivers that can promote its implementation. Purposive and snowball sampling techniques were adopted to target the questionnaire survey and interview respondents. Targeted respondents included building designers (architects & engineers), developers, building owners, building managers, maintenance staff, local authorities and end-users. Meanwhile, content analysis was used to synthesize the results from the expert interviews.

4. RESULTS AND DISCUSSION

The semi-structured interviews were conducted online via Microsoft Teams, Zoom App. and Google forms. The latter enabled interviewees to respond to the interview questions at their convenience. In addition, online interviews for qualitative research enable accessibility to participants [34] despite the social distancing restriction during the Covid-19 pandemic. The interviewees were selected based on their willingness to participate in the research interview, professional experience, type of profession, and years of experience in residential building design and/or construction, as outlined in Table 1. The interviewees were selected either as individuals or as representatives of their team, organization, or industry [35]. Questions were asked on the potentiality of the LPD scheme implementation in Hong Kong. The questions included sections on the barriers, drivers and facilitating measures of the LPD scheme. All the interviews were recorded, transcribed and analyzed. While the average duration for the interviews conducted via Video Conferencing Applications (Microsoft Teams and Zoom) was about 60 minutes, the average time for the interviewees who responded to the interview questions at their convenience was not determined.

Table 1. Building stakeholders' information

Code	Sector	Profession	Company Size	Experience (Years)
R1	International	Structural Engineer	>1000	5 - 10

Code	Sector	Profession	Company Size	Experience (Years)
R2	International	Structural Engineer	>1000	5 - 10
R3	Local	Structural Engineer	>1000	> 30
R4	Local	Building Services Engineers	<1000	> 30
R5	Local	Architect	<1000	> 30
R6	International	Building Developer	>1000	> 30
R7	Local	Project Coordinator	<1000	10 - 20
R8	Local	BIM Consultant/Digital Solutions Provider	<1000	20 - 30
R9	International	Building Surveyor	>1000	10 - 20
R10	International	Architect	<1000	10 - 20
R11	International	Digital Manager	> 1000	10 - 20
R12	Local	Facility Manager	<1000	> 30
R13	Local	Building Developer	> 1000	> 30
	Status	**Apartment Floor Level**	**Mismatch between Requirements and Design**	
R14	Homeowner	36/F	Yes	
R15	Tenant	12/F	No	
R16	Tenant	GF	No	

As depicted in Figure 1, the building professionals were asked to give their opinions on the current code of practice, building code and design manuals/guidelines. While about 77% of the interviewees (experts) affirmed the availability of comprehensive building regulations in terms of codes of practice and design manuals/guidelines in Hong Kong, about 62% of the experts confirmed the deficiency of the current building regulations to promote sustainable building design in Hong Kong. For instance, R2 asserted that "codes of practice should allow for simple retrofit and upgrade of the roofing structure to allow for Solar panels and green roofs to flourish if the building owner so wishes to do so later on in the building lifecycle".

In the same vein, R12 opined that "the enhancement and modification in the current guidelines and design manuals are not sufficient enough to meet the fast-growing demand or expectations of sustainable design requirements in Hong Kong." The interviewee further highlighted that "the concept of sustainability was not originally incorporated in the framework of building regulations from the outset, so, to have a better sustainable design, there is a need to review the whole building regulations and set up a framework of what a sustainable building design is". Thus, while there are many existing building regulations in Hong Kong, they have not been aligned with Hong Kong SAR's targets on Carbon Neutrality. Relatedly, 85% of the interviewed professionals have implemented sustainability concepts in the building development projects in Hong Kong (*see* Figure 2). These include the use of Building Information Modeling (BIM), Virtual Reality (VR) technology, incorporation of Internet of Things (IoTs), etc., in the design, construction and operation of high-rise buildings.

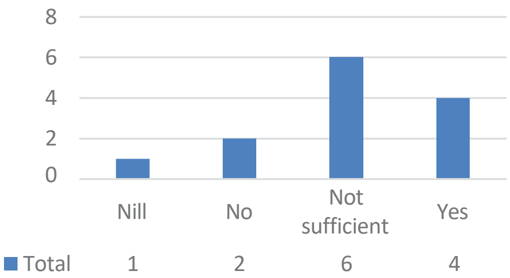

Fig. 1. Professionals' View on Adequacy of the Current Building Code to promote Sustainability in Hong Kong

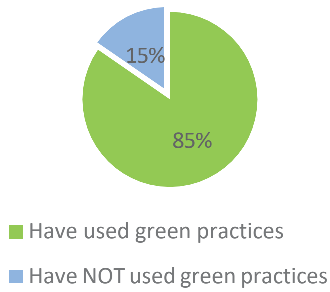

Fig. 2. Sustainability Implementation in Building Development Projects in Hong Kong

The building stakeholders' interviews were in two categories: with building professionals and the building end-users. Sixteen (16) interviews were carried out altogether, including thirteen (13) building professionals and three (3) building end-users. The interviews tried as much as possible to include two representatives from developers, architects, building managers, end-users, maintenance staff and building owners towards obtaining quality information [36]. However, due to time constraints, some of the representatives were replaced with other building professionals with relevant experience in the design and construction of high-rise residential buildings. The interviews included more of the building designer (structural engineers and architects), as revealed in Figure 3.

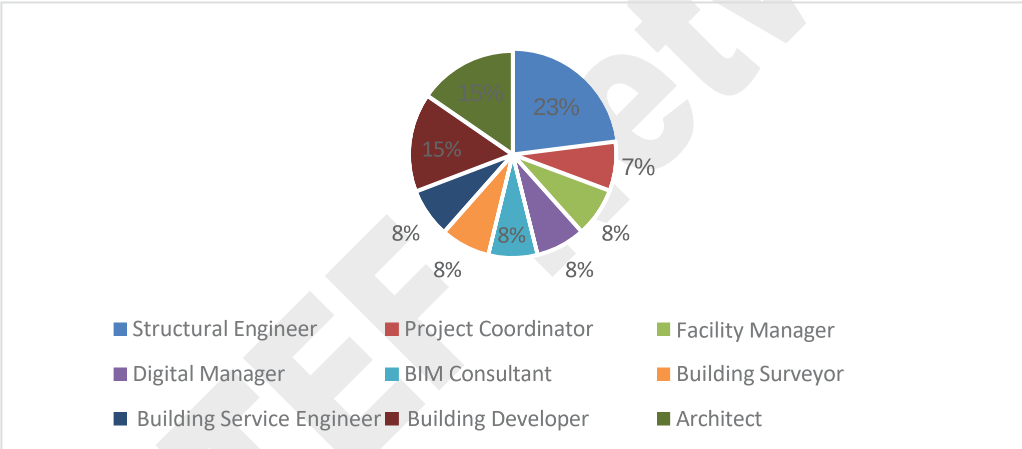

Fig. 3. Professionals' Background

Dehdasht et al. (2020) utilized the Technique for Order of Preferences by Similarity to Ideal Solution (TOPSIS) to identify and classify all the relevant drivers of lean construction implementation [37]. With sustainability as the core focus, the identified drivers were broadly categorized under the three headings of sustainability which are social, economic and environmental. Further classification yielded seven subcategories under which the drivers were grouped. However, these drivers do not specifically address the focus of this research, which is the LPD scheme. The production of buildings is carried out in three phases: design, engineering, and assembly [38]. LPD focuses more on the design and operational stage of the building rather than the manufacturing of the building [39]. Also, since this research focuses on high-rise residential buildings in Hong Kong, there is a need to contextualize the research findings of Ref. [37] for proper application in this research. The drivers were contextualized through the expert interview, as highlighted in Table 2. Relatedly, the barriers to LPD adoption were identified through a literature survey and interviews with relevant building professionals. The identified barriers are detailed in Table 3.

Table 2. Identified Drivers of the Lean Premise Design (LPD) Scheme

S/N	Drivers
Author/Expert Interview	
1	Training for building professionals (e.g., engineers, architects, etc.)
2	Building a communication platform for all stakeholders
3	Government providing bonuses and credits to developers who adopt LPD design, such as extra GFA, achievement awards, etc.
4	Government facilitates buyers to accept LPD design (e.g., reduction in stamp duty)
5	Government promotes public education in LPD
6	Improved circular economy to reduce carbon emission
7	Streamline the design and communication process
8	Clear definition of the LPD objectives in the design stage
9	Shifting from outputs (products) to outcomes philosophy
10	Enhance standardization in building design, construction and management
11	Enhance Modular Integrated Construction (MiC)
12	Easier to adopt offsite construction technology and reduce rework on site
13	Reduction in construction waste
14	Reduction in energy consumption
15	Reduction in material usage
16	Reduction in construction cost
17	Improved efficiency in the design process
18	Reducing workloads in building design and project management by reducing the scope of interior design/provisions
19	Improve company culture
20	Reduction in inventory and spare parts inventory

Table 3. Identified Barriers to the Lean Premise Design (LPD) Scheme Implementation

S/N	Barriers
1	Avoidance of making decisions and taking up responsibility in defining the scope of "lean design"
2	The thought that LPD is only beneficial to developers
3	Developers focus on the ROI (finances, branding, social responsibility, etc.)
4	Compromising profit due to reduction of GFA
5	Implementing LPD may require more time for market research, thus increasing design cost
6	The LPD building units may look less glamourous as compared to traditional design, which renders them less attractive to potential buyers
7	Inability to define peoples' expectations/requirements
8	Expectations of buyers in different price ranges are diverse. (Potential buyers of building pricing from lower to middle range (say 0.5-1000m) may prefer ready-to-move-in conditions while buyers for luxury buildings prefer to renovate by themselves.)
9	Building industry lacks the knowledge and skill of LPD
10	Building professionals are reluctant to new design approach if the current system 'works'
11	End-users' requirements are too diverse
12	Environmental constraints due to differences in site conditions
13	Building industry lacks the knowledge and skill of LPD
14	Lack of support from top management
15	Insufficient management skills of designers and builders
16	Absence of a lean culture in the construction field
17	Lack of communication and feedback among stakeholders
18	Government do not care about sustainability in building development
19	Resistance to change from traditional design practices

S/N	Barriers
20	Lack of communication and feedback from end-users on their requirements in the early stage of the design process
21	Limited application in design-and-build procurement models
22	Stringent requirements and approvals
23	Building designers are not familiar with the concept of LPD

Ref. [40] shows that less than 2.5% of the world's water is fresh, while the rest is seawater. Meanwhile, Hong Kong's total freshwater consumption in 2020 was 1 027 million M^3. Interestingly, 61% was consumed for domestic purposes, as shown in Table 4. This information suggests the potential of conserving a tangible quantity of freshwater resources in high-rise residential buildings in Hong Kong through the LPD scheme.

Table 4. Hong Kong's Annual Fresh Water Consumption by Sector in 2020 [46]

Sector	Quantity (Million m^3 and per cent of total)
Domestic	626 (61.0%)
Industrial	53 (5.2%)
Service Trades	222 (21.6%)
Government Establishment	44 (4.3%)
Construction & Shipping	19 (1.8%)
Flushing	63 (6.1%)

5. CONCLUSION

The research explores the potentiality of implementing the LPD scheme in HRR buildings in Hong Kong. Through literature review and semi-structured interviews with building stakeholders, key barriers, drivers and facilitating factors to LPD implementation were identified. As identified from the stakeholder interviews, there is a need to propose, develop and integrate new sustainable residential building design concepts into the existing codes and guidelines in Hong Kong. Furthermore, there is a need to set up a framework of what sustainable building design is. The LPD scheme considers the users' behaviour patterns, expectations, social needs, green maintenance techniques and government initiatives in the context of residential buildings.

Therefore, these findings have revealed important insights and directions to the next stage of this research project. The next phase of the research will study the end-users' requirements and acceptance of sustainable building design, including water conservation and the developments of the end-user-oriented design (EOD) concepts. Implementing EOD concepts in HRR buildings will promote partnership among stakeholders in order to reduce the consumption of energy and natural resources.

Future Studies. To ensure the study sufficiently cover salient aspects of sustainable building design and water conservations, its next phase investigates areas including but not limited to building end-users' requirements and the willingness and extent to pay on top of the market price for sustainably designed apartments. These aspects include design provision for recycling wastewater from sinks for toilet flushing or watering garden plants; connecting the interior and exterior spaces with balconies, among others. Furthermore, the discussed results will facilitate identifying the latent defects associated with the mismatch of building designs and the end-user's requirements in high-rise residential buildings.

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COMPETING INTERESTS

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Higher Education Institutions in the Sustainable Transition: A Study at the University of Aveiro

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ABSTRACT

In accordance with the United Nations 2030 Agenda for Sustainable Development, Higher Education Institutions (HEIs) play a key role in raising environmental awareness and implementing practices to contribute to the Sustainable Development Goals (SDGs), in a scenario of protecting the environment and promoting innovation and resilience – social, cultural, scientific, and technological. Considering HEIs as active agents of change in the global network for sustainability, the present study aims to present the main strategies and current initiatives of HEIs in the implementation and development of sustainable campus, regarding institutional physical interventions. Using the literature review, it is intended to identify the various contributions of HEIs in the Portuguese and European panorama for this emerging challenge, about the categories evaluation that include energy, greenhouse gas (GHG) emissions, waste, procurement practices and the built environment, mobility, biodiversity, water and food security, also highlighting the initiatives to increase environmental sustainability on the campuses of the University of Aveiro (UA). The conclusions of this research indicate that the HEIs, at the European level are implementing good practices in the management of the campus as a living and evolving laboratory, but there are potential interventions, still little explored, to make the operations of the HEIs more sustainable and effective.

Keywords: Sustainable Campus, Circular Economy, Higher Education Institutions, University of Aveiro.

1. INTRODUCTION

From an operational point of view, large campuses are on a par with small cities in terms of population and urban characteristics [1], with HEIs playing a key role in the transition from a linear economic model to a circular one [2]–[6], through adoption and development of circularity and low carbon principles, in key sectors such as food, building construction and rehabilitation, transport and mobility, energy, waste and water (and wastewater) management [4]–[7], which directly affect the well-being of the academic community, economic growth and environmental quality. This is a process that involves challenges, but above all opportunities, through a socially fair and cohesive transition.

The commitment to transformative initiatives that promote a more sustainable planet has led HEIs in Europe to create action plans to improve internal policies, methodologies and procedures, through teaching, research and the operational management structure of the campuses [8], [9].

With the universities' concern increase about climate change and the development of sustainable campuses grows, so does the demand for HEIs to participate in international sustainability rankings, which assess and classify HEIs' contribution and performance in the incorporation of sustainable development in its key functions [10]. With greater notoriety, in the global panorama, are the Universitas Indonesia Green Metric (UI Green Metric) and Times Higher Education (THE) Impact Ranking [11], [12].

The UI Green Metrics World University Ranking, a pioneer in the assessment of HEIs in the field of sustainability, seeks to assess how universities respond to or deal with sustainability issues through policies, actions and communications, as well as indicators for ranking metrics, represented by: Setting and Infrastructure (15%), Energy and Climate Change (21%), Waste (18%), Water (10%), Transportation (18%), and Education and Research (18%) [13]. Participation in the ranking is voluntary, its coverage is global, and the assessment is carried out at an institutional level. Data collection takes place through online questionnaires sent to university administrators.

Another international ranking, THE Impact Rankings, is the only instrument that uses the United Nations SDGs framework to assess the performance and contributions of HEIs in relation to sustainable development, using the following indicators for analysis: Research, Stewardship, Outreach and Teaching, contribute to global challenges. To participate, universities need to provide performance data on SDG 17 (22%) and at least three other SDGs (26%), thus, HEIs are scored based on a different set of SDGs depending on their focus [14]. Participation in the ranking is voluntary, its coverage is global, and the assessment is carried out at the institutional level in all SDGs, and also individually for each SDG.

The HEIs in Portugal are committed to putting sustainability at the center of their goals, and much is being done to provide the academic community with an environment where all facilities become increasingly sustainable [15], [16]. National Strategy for Development Education 2018-2022, Portugal 2030 Strategy, National Energy and Climate Plan 2030, Recovery and Resilience Plan, are examples of programs and strategies that have contributed for the commitment of educational institutions to sustainability in Portugal, meeting the responsibilities assumed by Portugal within the framework of the European Union – making Europe greener and more resilient. In this context, Portuguese universities and polytechnics are part of a cooperation network – Sustainable Campus Network, where approximately 40 HEIs are committed to the principles and practices of sustainable development, in all its relevant aspects and dimensions – environmental, social and economic [17].

At the national level, the UA has increasingly become a reference when it comes to sustainable campus development [18]. It is worth noting that the UA is in the District of Aveiro, in mainland Portugal, and consists of organic units of the university subsystem (16 departments) and the polytechnic subsystem (4 schools), in addition to 20 research units. The UA is currently attended by around 13600 under-graduate students and 3800 post-graduate students [19]. The UA's Strategic Plan for the 2019-2022 quadrennium includes sustainability from the outset in its values and principles, affirming the search for sustainability from an environmental, economic, and social point of view. In this context, the following objectives are defined [20]: (i) make sustainability a goal for all; (ii) invest in heritage maintenance and rehabilitation; (iii) maximize the use of funding opportunities (national and international); and (iv) promote and evaluate the efficiency, effectiveness, and economy of the application of resources.

At the international level, the UA as European ECIU University – European Consortium of Innovative Universities (established in 1977) is involved in several initiatives to achieve the SDGs and as promoters of synergies to consolidate the ECIU's research and innovation agenda, which emphasizes the development of smarter European regions and seeks to meet the 2030 Agenda – with a focus on energy, circular economy, transport and mobility and resilient communities [21]. As part of the HEI's sustainability policy and strategy, the UA promotes several projects and initiatives on this topic, through the “Grupo para a Sustentabilidade” and the “Campus mais sustentável” actions [18]. Worthy of mention is the UAveiroGreenBuilding project, whose objective is to define an innovative methodology in the logic of the principles of sustainability and circular economy in the construction, rehabilitation, and maintenance of buildings. This European Economic Area Grants (EEA Grants) supported project is led by the UA, in partnership with the Sustainable Habitat Cluster in Portugal and the Icelandic entity EVRIS Foundation [22].

In this context, considering HEIs as strategic agents in supporting sustainable development through their operational activities, and also representing a source of environmental impacts due to the significant consumption of resources and waste generation, this study aims to recognize the main strategies and current initiatives of HEIs in the implementation and development of sustainable campus, with regard to institutional physical interventions. Using a bibliographic review and document analysis, it is intended to identify the contributions of HEIs, in the Portuguese and European panorama, which were highlighted in international assessments for this emerging challenge, in particular the main sustainability actions on the UA campuses.

2. METHODOLOGY

To carry out the study, applied research was carried out with a qualitative approach, around textual elements, and considering the pre-defined content analysis methodology [23], the fundamental phases were respected, in logical sequence, namely: pre-analysis; exploration of the material; and treatment of results, including inference and interpretation. Like any technique, it requires rigor and systematic rules that must be respected to extract the maximum inferences from a phenomenon in research.

A documentary survey was carried out of sustainability programs and initiatives developed in institutional physical interventions, namely disclosed in Strategic and Action Plans, Environmental Sustainability Strategies or Sustainability Reports, made available online by the HEIs. For this analysis, HEIs highly ranked in terms of their environmental credentials in international rankings were selected. Initially, the European universities that obtained positions among the 200 best HEIs worldwide in THE Impact Rankings – 2022 [24] were selected, with a limitation of two universities per country/region, in order to study actions in different places in the European space, which makes it possible to enrich the research due to the diversity of realities. Through the applied methodology, it was possible to analyze 21 HEIs distributed in 12 different locations.

Subsequently, HEIs were highlighted in the Portuguese panorama for their commitment to the SDGs defined by the United Nations, regarding initiatives to increase environmental sustainability in the respective operations of the campuses. In this context, the national HEIs that had globally positive results in the 2022 annual exercise were selected, evaluated by THE Impact Rankings, with the assessment of universities that obtain a place among the 300 best HEIs worldwide. In fact, not only was the presence of two institutions – University of Coimbra (position 26) and University of Trás-os-Montes and Alto Douro (position 78) – in the TOP100 of the global ranking, but four more HEIs were highlighted in positions among the 300 best universities in the international ranking, including the University of Aveiro.

Therefore, the lived experiences and initiatives reported by the HEIs were analyzed and acquired through the investigation of empirical knowledge, and the data collection was carried out through indirect and online observation, seeking information in the published documents of the selected HEIs, which are made available for public access. To facilitate the analysis of the results, the data obtained in the documentary survey were organized into categories of analysis defined based on the guidelines established by the GRI (Global Reporting Initiative), characterized by being a modular system of interconnected standards that aim to collaborate with organizations to understand and communicate their impact on sustainability issues [25]. In view of this, the analysis categories were listed, as follows: (1) Energy; (2) Greenhouse Gas (GHG) Emissions; (3) Waste; (4) Procurement Practices and the Built Environment; (5) Mobility; (6) Biodiversity; (7) Water; and (8) Food Security.

3. RESULTS AND DISCUSSION

This section presents a descriptive analysis of the survey results. The subsections are based on three levels –HEI Europe, HEI Portugal and UA, meeting the initial objectives of the present study.

3.1 HEI - Europa

These institutions are committed to developing, applying and sharing practices in environmental management strategies promoting a sustainable community for education. All HEIs have common goals to make progress on greenhouse gas reduction commitments and transform their Campi into living laboratories for sustainability. The intention is for each of them to explore sustainability solutions in their structures and contribute to the resilience of local and global communities in all cultural contexts, as identified in Table 1.

Table 1. List of HEIs – Europe and sustainable initiatives on campi

Rank(*)	Country | Region	HEIs	Analysis Categories							
			1	2	3	4	5	6	7	8
8	United Kingdom	Newcastle University	X	X	X	X	X	X	X	X
9	United Kingdom	University of Manchester	X	X	X	X	X	X	X	X
26	Portugal	University of Coimbra	X	X	X	X	X	X	X	X
31	Denmark	Aalborg University	X	X	X	X	X	X	X	X
37	Italy	University of Bologna	X	X	X		X	X	X	
42	Sweden	KTH Royal Institute of Technology	X	X	X	X	X	X	X	X
47	Ireland	National University of Ireland, Galway	X	X	X	X	X	X	X	X
62	Ireland	University College Cork	X	X	X	X	X	X	X	X
65	Netherlands	Wageningen University & Research	X	X	X	X	X	X	X	X
76	Italy	University of Padua	X	X	X	X	X	X	X	
78	Portugal	University of Trás-os-Montes and Alto Douro	X	X	X	X	X	X	X	X
92	Denmark	University of Southern Denmark	X	X	X	X	X	X	X	X
101-200	Netherlands	University of Groningen	X	X	X	X	X	X	X	X
	Finland	Lappeenranta-Lahti University of Technology LUT	X	X	X	X	X	X	X	X
	Belgium	Université Libre de Bruxelles	X	X	X	X	X	X	X	X
		Université Catholique de Louvain	X	X	X	X	X	X	X	X
	Latvia	University of Latvia	X	X	X	X	X	X	X	X
	Spain	University of Barcelona	X	X	X	X	X	X	X	X
		University of Girona	X	X	X	X	X	X	X	X
	France	Montpellier University	X	X	X	X	X	X	X	X
	Sweden	University of Gothenburg	X	X	X	X	X	X		

(*) Rank – THE Impact Rankings 2022
(1) Energy; (2) GHG Emissions; (3) Waste; (4) Procurement Practices and the Built Environment; (5) Mobility; (6) Biodiversity; (7) Water; and (8) Food Security.

Universities have been striving to gain efficiency and reduce the impact on natural resources and to adopt sustainable practices in the areas of energy and water consumption, improving biodiversity and waste generation. More recently, areas of sustainable procurement management and associated campus transportation have been receiving attention in promoting sustainable practices, with emphasis on the operations indicated below:

Net-zero carbon and near-zero energy buildings: replacement of existing lighting and equipment with more efficient ones; installation of intelligent control and automation sensors; upgrades to insulation systems; use of renewable energy sources, such as photovoltaic panels and wind turbines; decarbonization of heating/cooling systems by thermal energy storage systems.

Optimization in sustainable waste management: behavioral change in the reduction of waste per capita and the respective organic fraction (elimination of plastics, paper and food waste); sustainable waste sorting; increased reuse and recycling of waste on campuses – promotion of the circular economy; generation of heat and energy through the burning of non-recycled materials; removal of the disposal of municipal waste in landfills.

Sustainable practices in procurement and in the built environment: adoption of a policy and strategy for sustainable and responsible procurement and contracting (promoting decarbonization in the investment chain for new circular products and services, and valuing local suppliers); implementation of environmentally sustainable practices in the construction of new buildings and in the rehabilitation of buildings, with the promotion of passive building projects with zero energy balance (NZB: Net Zero-Energy Buildings); management and constant monitoring of the built environment (performance indicators).

Reduction of mobility impacts: reduced use of individual transport and incentives to promote smoother and more sustainable modes of transport, such as bicycles, electric vehicles, carpooling or public transport; improvement and increase of support infrastructures; greater travel management by the academic community; incentives and ease of use of digital tools for remote participation (reduction of air and land travel).

Promotion and maintenance of biodiversity: creation of habitats for wild pollinators, through the inclusion of nests and planting of wildflowers on campus; landscape management, with planting of native species and control of exotic plants; elimination of chemicals, such as fertilizers, pesticides, and herbicides, in community gardens and campuses.

Promotion of water efficiency: implementation of the 5R's principle, namely replacement of usage devices by more efficient ones and rainwater harvesting systems; management of water consumption, through control and monitoring systems in campus's buildings.

Strengthening food sustainability: redistribution of meals to people with food insecurity, avoiding waste; promotion of new eating habits and lifestyles (greater nutritional adequacy of the academic community's diet); use of fresh vegetables grown on campus, reduction of meat consumption, availability of meals based on organic ingredients; construction of horizontal and vertical community gardens on campus.

It is evident, in general, that HEIs recognize their importance in the dissemination of environmental awareness, functioning as a reference to the academic community and society through their own actions. Continuous improvements contribute to the relevant positions of HEIs in classification rankings, and in the articulation of an Environmental Management System subject to certification and acquisition of environmental seals, based on the guidelines of ISO 14001 (Environmental Management Systems) and 50001 (Energy Management Systems).

3.2 HEI - Portugal

In relation to national HEIs, in recent years measures have been taken to reduce the carbon footprint (commitments in line with the National Energy and Climate Plan 2030), adoption of sustainable practices in the areas of energy and water consumption, preservation of biodiversity, waste management and sustainable procurement, with programs and initiatives aligned with the HEIs in Europe (Table 2).

Table 2. List of HEIs – Portugal and sustainable initiatives on campi

Rank(*)	HEIs	Analysis Categories							
		1	2	3	4	5	6	7	8
26	University of Coimbra	X	X	X	X	X	X	X	X
78	University of Trás-os-Montes and Alto Douro	X	X	X	X	X	X	X	X
101-200	NOVA University of Lisbon	X	X	X	X	X	X	X	X
201-300	University of Algarve	X	X	X	X	X	X	X	X
	University of Aveiro	X	X	X	X	X	X	X	X
	University of Minho	X	X	X	X	X	X	X	X

(*) Rank – THE Impact Rankings 2022

(1) Energy; (2) GHG Emissions; (3) Waste; (4) Procurement Practices and the Built Environment; (5) Mobility; (6) Biodiversity; (7) Water; and (8) Food Security.

It is worth mentioning that all Portuguese HEIs, object of this analysis, are part of the Sustainable Campus Network, reflecting the positive aspects of the influence of alliances in sustainability actions on university campuses.

3.3 UA - University of Aveiro

Affirming the commitment and dedication of the academic community to sustainable development, the UA, through the “Grupo para a Sustentabilidade” and the “Campus mais sustentável” actions, has been developing actions equivalent to European and Portuguese HEIs. These continuous improvements contribute to the relevant positioning of the UA in international rankings, inherent to this theme, as in the participation of UI Green Metrics and THE Impact Rankings (Figure 1).

THE IMPACT RANKINGS 2022	1 NO POVERTY	13 CLIMATE ACTION	14 LIFE BELOW WATER
RANKED 201-300	62 th	76 th	98 th
UI GreenMetric World University Rankings 2021	Energy and Climate Change	Waste	Education & Research
RANKED 177	46-52 th	81-121 th	75-87 th

Fig. 1. UA position in *THE Impact Rankings* 2022 and *UI Green Metric* 2021

In the latest edition (2021) of the UI Green Metric World University Ranking, the UA has the highest score compared to previous years (up 13 positions compared to 2020), occupying the 177th position in 956 evaluated HEIs from 80 countries. At the national level, the UA ranks second in seven institutions evaluated. However, considering the Education and Research criterion, the UA leads the group of national institutions. At a global level, the UA was also highlighted in the Energy and Climate Change and Waste categories.

The UA was evaluated on all 17 SDGs, demonstrating its commitment to meeting international goals and reaching positions between 201-300 worldwide, in a total of 1406 participating HEIs from 106 different countries/regions, through THE Impact Rankings. The UA obtained the highest scores on the SDGs: No poverty (SDG 1) – 1st position nationally and 62nd worldwide; Sustainable cities and communities (SDG 11) – 2nd position at national level; Responsible consumption and production (SDG 12) – 1st position at national level; Climate Action (SDG 13) – 3rd nationally and 76th globally; Life below water (SDG 14) – 98th worldwide; and Partnerships for the goals (SDG 17).

It is noted that there is a great involvement of the entire academic community regarding sustainability on the UA campuses, which has been decisive for the existence of a significant increase in actions in favor of emerging challenges. Below are some examples of initiatives and programs involving sustainable campus operations at UA:

(1) Energy: intervention in terms of lighting (replacement of equipment, installation of control systems and improvements to existing infrastructure); improvements in the efficiency of Heating, Ventilating and Air Conditioning systems; installation of a telemetering system (automated accounting).

(2) GHG Emissions: production of energy from renewable sources; installation of thermal solar panels; application of a geothermal and biothermal solution for building air conditioning; building thermal comfort improvements; actions of the other categories that reduce the carbon footprint.

(3) Waste: Recycling and Refunding of Aluminum and Polyethylene Terephthalate (PET) Packaging - REAP project, a recycling system that allows the academic community to deposit aluminum and PET packaging in machines distributed across campuses, generating credits; platform for reuse and sharing of goods and equipment between UA units/services - "Vamos Reutilizar"; “CicloCompost” project at HortUA Lab, which consists of offering a service for the collection of bio-waste at home and its recovery through the composting process, in addition to training offers for the local community; implementation of a centralized waste system; monitoring and recording of waste produced.

(4) Procurement Practices and the Built Environment: adoption of sustainable practices and principles of circularity in the construction and rehabilitation of buildings - “UAveiroGreenBuilding” project; acquisitions systematically evaluated having as a reference, in addition to the usual criteria, sustainability criteria.

(5) Mobility: encouragement of more sustainable forms of mobility and the use of non-polluting forms of locomotion, for example, the creation of closed parking for bicycles and the UAUbike project (offer of conventional and electric bicycles).

(6) Biodiversity: planting trees on campuses and region; actions by HortUA - Sustainability Living Lab; installation of urban furniture to promote outdoor spaces.

(7) Water: replacement of devices with a more efficient model and installation of timers; installation of rainwater harvesting systems; water efficiency audits of campus buildings.

(8) Food Security: FairFood project, aimed at restoring healthier eating habits, based on traditional cuisine and local and seasonal products; promotion of Appointments on Nutrition and Food Health for students, through the Social Action Services.

According to the Account Management Report (University of Aveiro Group 2021), the results of the work carried out, regarding sustainability on campuses, are evident. As an example, the results of the plan indicators for the strategic objective – “Uma UA sustentável”, with the corresponding goals achieved in 2021 activities [26]: recycling and reuse rate (increase of 39.58%); efficient and ecological use of water (21.51% reduction); paper consumption (7.79% reduction); number of bicycle users - Smooth Mobility Plan (170 users); improvement in the thermal, acoustic and ventilation comfort of the buildings (5 spaces); creation of maintenance plans (2 units); and dematerialization of processes (process designs: 13 and process implementation: 6 units). These results reflect a more sustainable UA in terms of energy, water, paper, waste and mobility on campuses.

4. CONCLUSION

HEIs play an important role as strategic agents in supporting sustainable development through teaching, research, and social extension activities. These also represent a source of environmental impacts due to the significant consumption of resources and generation of waste. Thus, an HEI must ensure that its policies and practices minimize negative impacts on the environment, promoting integrated and effective strategies in the short and long term.

This work contributes to increasing HEIs' awareness of their responsibility in promoting sustainable development. At the European level, HEIs are implementing good practices in managing the campus as a living and evolving laboratory.

In total, the HEIs studied are committed to the reduction of greenhouse gases in their operations (Category 2) and the promotion of almost zero energy buildings (Category 1). With evidence for the use of energy from renewable sources, including the generation of energy by wind turbines installed on university campuses, and the development of new clean energy solutions, such as Power-to-X technologies (conversion technologies that transform electricity into carbon-neutral synthetic fuels, known as electrofuels or e-fuels). Another category that is directly related to the decarbonization of activities on campuses, becoming a priority in HEIs in recent years, is the reduction of the impacts of mobility (Category 5), with initiatives to promote smoother and more sustainable modes of transport, identifying measures effective in 100% of the institutions that were the focus of this research. Recognizing the inevitability of the energy transition given the climate emergency and the need to change the economic paradigm, in particular, regarding fossil fuels, European HEIs (highlights in international rankings for sustainability) clearly undertake to decarbonization of the economy, with the aim of reducing its greenhouse gas emissions.

As identified in the previous section, practices for optimizing sustainable waste management (Category 3) and promoting and maintaining biodiversity (Category 6) are also included in the goals for sustainable development of the HEIs studied. The implementation of sustainable waste management, with awareness of the transition to a more circular economy, is part of the policies adopted by the 25 analyzed HEIs, in which the value of products, materials and resources remains in the economy for as long as possible and production of waste is reduced to a minimum. This transition is an opportunity to transform the economy of institutions and create new and sustainable competitive advantages for Europe. As far as the universities' biodiversity policies are concerned, they mainly include the promotion of urban wildlife and the improvement of green spaces on campuses, by encouraging habitats for animals in vulnerability to extinction, planting native species, creating orchards and community gardens, and the development of green roofs and living walls in university buildings. These measures aim at the European Union's biodiversity strategy to ensure that ecosystems are restored, resilient and properly protected.

In Category 4 – Procurement Practices and the Built Environment – close to 90% of the HEIs under analysis incorporate in their purchasing processes, policies or methodologies for the organization's sustainable acquisitions, enhancing environmental and social criteria. The adoption of measures for sustainable procurement drives the appreciation of local suppliers and, also, the supply chains to be more transparent, sustainable and circular. About sustainable construction, a little less than 75% of institutions mention in the media, process strategies to guarantee sustainability in construction, maintenance and renovation of their buildings. Regarding the strengthening of food sustainability at universities, approximately 90% of HEIs have a healthy and sustainable food policy or promote food security programs to the academic community.

It is possible to observe that all the analyzed HEIs have common goals of transforming their campuses into living laboratories for sustainability. However, there are opportunities for improvement, in the recommendation of efficient technical solutions, referring to the adoption of measures for the efficient use of water (Category 7). In this context and in an attempt to systematize, it is assumed that the ways to achieve a more efficient use of water in buildings are related to the 5R principle [27]: Reduce consumption, Reduce losses and wastes, Reuse water, Recycle water and Resort to alternative sources.

On average, 12% of the 25 HEIs analyzed, and which were highlighted in international rankings, are very consistent in adopting circularity and the principle of water recycling. With emphasis on the treatment of effluents from the disposal of gray water, reintroducing the water in the circuit and constituting an alternative resource in non-potable consumption. The other HEIs are consistent in an effort to consciously and monitored the reduction of the water they use, where, on average, 50% of the HEIs refer to interventions in their water management strategies on campuses, through the use of alternative sources, such as rainwater harvesting. Regarding the efficient management of water in the operations of the HEIs, it is noted that current efforts are aimed at reducing losses and consumption. However, sustainable campus strategies should, whenever possible, include the integration of solutions with alternative origins and the design of innovative systems, aiming at the reuse and recycling of water and incorporating the principles of circularity in the efficient management of this resource in the operations of the HEIs.

Water efficiency translates into economic, environmental and strategic benefits, and contributes to social awareness of the need to preserve water as a natural resource, through the simple and planned implementation of systems that contribute to improving sustainability. In general, these systems act as an alternative supply mechanism for buildings, irrigation of green spaces and as a water reserve in dry seasons, a frequent scenario in the Mediterranean region, with a high risk of water stress or even water scarcity.

The University of Aveiro has assumed sustainability as a foundation in its strategic plan, and aims to integrate, to a greater extent, themes related to sustainable development in the institutional physical operations on its campuses, as well as the assumption of decarbonization objectives in line with the national and European purposes.

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#4. Water efficiency and demand management

Nearly zero water buildings: contribution to adaptation and mitigation processes in urban environments

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ABSTRACT

Extreme weather events related to heat waves and heavy rainfall are expected to intensify in the coming decades in various regions of the planet as a result of climate change. The impacts of these events on urban environments will be particularly significant, given that, according to the United Nations, around two-thirds of the world's population will live in cities by 2050. Buildings and other urban infrastructures therefore have an important role to play in the processes of mitigation and adaptation to climate change. The implementation of nearly zero energy buildings (NZEB), for example, can contribute to very significant reductions in greenhouse gas emissions, which is why the European Union has established special requirements in new buildings. However, “nearly zero buildings” for all resources, not just energy, should employ integrated and enhanced construction solutions in the future, not only contributing to increasing environmental sustainability in urban areas, but also playing an important role in adaptation and mitigation in relation to climate change. In the case of “nearly zero water buildings”, they can increase the resilience of urban environments in the face of extreme events such as prolonged droughts or extreme precipitation and, considering the water–energy nexus, can also contribute to a significant reduction in emissions. The “zero building” concept is not, however, similar for all resources. In the case of energy, the usual concept of NZEB does not mean a circular use of the resource, but rather that the total amount of resource used by the building is approximately equal to the amount of renewable resource produced or available on the site. In the case of water, part of the resource can be used in a circular way (water recycling), but renewable local sources alternative to the supply from the public network can also be considered. The design of “nearly zero water buildings” should be based on the 5R principle of water efficiency: Reduce consumption; Reduce losses and waste; Reuse water; Recycle water; and Resort to alternative sources (rainwater, salt water, etc.). It is obvious that water efficiency is of the utmost importance in the face of prolonged droughts, but the use of rainwater in urban areas has an additional known effect in dampening flood peaks. Considering the water–energy nexus, reducing water consumption in a building (the 1st R) also produces significant energy savings. This is a result of reducing the energy needs for domestic hot water, to pressurise water in buildings, and also in public systems, in pumping and treatment of water and wastewater

Keywords: climate change, water efficiency, zero building

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1. INTRODUCTION

Extreme weather events related to heat waves and heavy rainfall are expected to intensify in the coming decades in various regions of the planet as a result of climate change. The impacts of these events on urban environments will be particularly significant, given that, according to the United Nations, around two-thirds of the world's population will live in cities by 2050 [1,2]. Buildings and other urban infrastructures therefore have an important role to play in the processes of mitigation and adaptation to climate change. The implementation of nearly zero energy buildings (NZEB), for example, can contribute to very significant reductions in greenhouse gas emissions, which is why the European Union has established special requirements for new buildings.

Currently, it is recognised that water, energy and nutrients are interconnected and constitute the essential connection for the sustainable development of humanity, so they should not be considered separately [3]. Water is essential for the production of many forms of energy, and energy is needed for almost all water uses. Water and energy are also associated with food production [4]. Agriculture currently represents, on average, 70% of the world's water consumption and 30% of global energy consumption, considering the supply chains [5].

This water–energy–nutrients nexus is also valid at the scale of buildings. In their use phase, buildings are places of high consumption of resources and, therefore, “nearly zero buildings” for all these resources, not just energy, must employ integrated and enhanced construction solutions in the future. Their role is essential not only for increasing environmental sustainability in urban areas but also as a contribution to the adaptation and mitigation of climate change. In the case of “nearly zero water buildings”, in addition to their evident contribution to water efficiency in urban areas, they can increase the resilience of urban environments in the face of extreme events such as prolonged droughts or extreme rainfall and, considering the water–energy nexus, can also contribute to a significant reduction in emissions [6].

The “zero building” concept is not, however, similar for all resources. In the case of energy, the usual concept of NZEB does not mean a circular use of the resource, but rather that the total amount of resource used by the building is approximately equal to the amount of renewable resource produced or available on the site. In the case of water, part of the resource can be used in a circular way (water recycling), but renewable local sources alternative to the supply from the public network can also be considered. Regarding “nearly zero nutrients’ buildings”, the resource can be considered circular. Nutrients such as phosphorus, for example, can be recovered from urine in buildings and used as fertilizer in green roofs or in urban agriculture in public spaces [7-9].

2. METHODOLOGY

This paper analyses the contribution of nearly zero water buildings to adaptation and mitigation processes in urban environments, based on the 5R principle of water efficiency in buildings [10,11]: Reduce consumption; Reduce losses and wastes; Reuse water; Recycle water; and Resort to alternative sources (rainwater, salt water, etc.).

The first R - Reducing consumption, includes the adoption of efficient products and devices, without compromising user comfort, public health or the performance of building networks. The second R - Reduction of losses and waste, may involve interventions such as monitoring losses in building networks and usage devices (flushing cisterns, garden irrigation systems, etc.) or the installation of circulation and return circuits of sanitary hot water.

The third and fourth Rs – wastewater reuse and recycling, are important measures for the design of near-zero water buildings, which are distinguished by the fact that reuse is a series use and recycling is a reintroduction of the same water into the beginning of the building circuit (after treatment). Reuse has undergone a certain development in recent years in relation to greywater, in particular the reuse of effluents from baths and washbasins for flushing toilets and irrigation.

The last R - Resort to alternative sources, may involve rainwater harvesting or desalinisation, for example. This concept of alternative sources refers to water that does not come from the public distribution network and which, in some cases, can be used for purposes that do not require drinking water.

For each of these “Rs”, its contribution to the processes of adaptation and/or mitigation of climate change is analysed. Here, the water–energy nexus clearly plays a fundamental role in terms of reducing emissions (mitigation), insofar as, by reducing water consumption, energy consumption is reduced in the building, in the production of domestic hot water and in eventual pressurisations, and also in public systems, in the pumping and treatment of water and effluents.

But all the interventions considered in the 5R principle can have a significant importance in adaptation processes. It is obvious that water efficiency in general is of the utmost importance in the face of prolonged droughts and that, for example, rainwater harvesting in urban areas has an additional known effect on dampening flood peaks, which may occur in the context of extreme precipitation events, driven by climate change.

3. RESULTS AND DISCUSSION

3.1. Reducing consumption

Several studies, including by the European Commission [10], show that the adoption of efficient water-using products (WuP) in buildings can reduce consumption by about 30% compared to the current scenario, a very relevant value and with significant implications also for the level of energy consumption and mitigation of emissions, as mentioned above. Two essential measures with a view to reducing consumption in buildings (1st R) are water efficiency audits, for existing buildings, and the water efficiency labelling of WuP, which makes it possible to raise awareness and support consumers in choosing the most efficient products in new buildings and renovations.

In the case of Europe, energy labelling has been mandatory for some years, under the so-called "Energy Directive", but there is still no obligation in terms of water efficiency labelling, despite the fact that the countries of the South, with a Mediterranean climate, are suffering increasing water stress or even water scarcity as a result of climate change. Interestingly, the European Energy Directive provides for mandatory labelling of products that use hot water (taps and showers), but only in terms of their energy efficiency, although the relationship between water consumption and energy consumption in these products is straightforward.

In the absence of a mandatory European system of labelling the water efficiency of products, this has been promoted since 2007 on a voluntary basis, in countries or by sector associations more sensitive to this problem. Currently, there are three voluntary water efficiency labels in Europe, namely the German WELL label, the European Water Label (EWL), supported by the European Industry for Taps and Valves (CEIR), and the Portuguese ANQIP label (Figure 1) [12], which was the first scheme to be launched in Europe. As the European Commission has not yet implemented the energy label for taps and showers, two countries where energy issues are very relevant (Sweden and Switzerland) have meanwhile adopted voluntary energy labels for WuP.

In view of the current situation, with a wide dispersion of national labelling schemes, harming information to consumers and the free trade of products, ANQIP, EWL and the Swedish and Swiss labels decided to come together, creating a single label of water and energy efficiency for products – the Unified Water Label (UWL) (Figure 2) [13]. For the launch and management of this unified label, a specific association has already been set up, based in Brussels - the Unified Water Label Association (UWLA). The European Commission is supporting this process, providing for the possibility of establishing a “Voluntary Agreement” with the UWLA and waiving the energy label for taps and showers provided for in the Energy Directive.

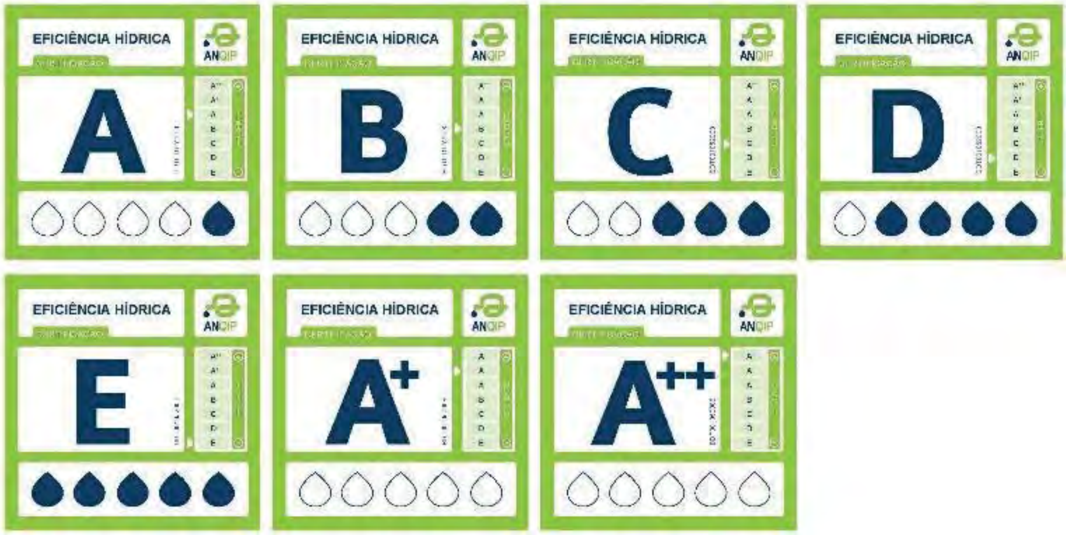

Fig. 1 - Water efficiency labels for products (ANQIP) [12]

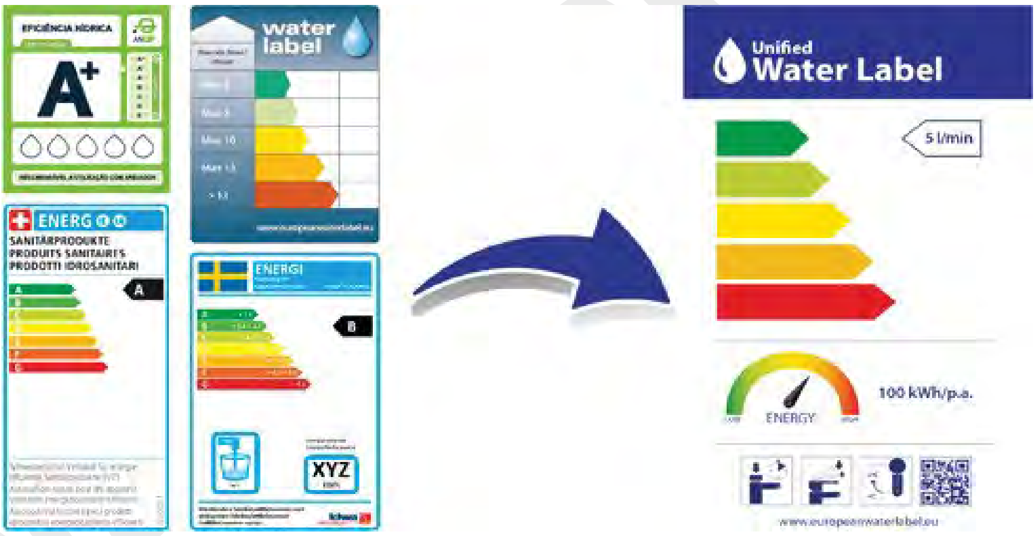

Fig. 2 – European Unified Water Label [13]

A study promoted in Aveiro (a medium-sized Portuguese city) by ANQIP - the Portuguese association for quality and efficiency in building installations - found that energy savings due to the use of efficient products (classified in category "A" of the ANQIP's water efficiency labelling) allows for a reduction in emissions of more than 100 kg of CO_2 per capita and per year, compared to the current scenario, considering the heating of domestic hot water in buildings and energy consumption in public networks [12]. It should be noted that, in Portugal, energy consumption for heating domestic hot water represents more than 30% of the total energy consumption in homes. Based on the results of the study, it is possible to determine that the discharge of a 6-litre cistern implied an energy consumption close to 12 Wh in the public water supply and drainage networks, equivalent to a common 3 W LED lamp turned on for 4 hours [13].

This study also made it possible to demonstrate the significant contribution made by the efficiency of sanitary products to the reduction of greenhouse gas emissions and, consequently, to the mitigation of climate change. Water efficiency also contributes to adapting to situations of prolonged droughts or water scarcity intensified by climate change, through the conservation of drinking water that it promotes.

3.2. Reduction of losses and waste

The concept of waste basically corresponds to an unnecessary (excessive) consumption of the resource in the "production" process, for example, negligent use of water in a building. In other words, water waste is, in essence, a set of actions and processes by which water is wasted without benefit, or simply misused.

In buildings, the most common waste situation results from the lack of circulation and return circuits for sanitary hot water (SHW), resulting in long waiting times for hot water in showers or taps. Therefore, the circulation and return of sanitary hot water, or equivalent solutions, have been increasingly considered in regulation in many countries. In Europe, for example, most building regulations already require the installation of SHW circulation and return circuits when the distance between the WuP and the hot water producing or accumulating equipment is relevant (10 or 15 metres, depending on the country) [14].

The water savings resulting from installing SHW circulation and return circuits can be significant. Considering, for example, a shower with a flow rate of 20 l/min and a distance of 20 metres between the device that produces or accumulates hot water and the shower, the waiting time will be approximately 40 seconds, so the possible savings will be of about 13.5 litres, equivalent to three flushes from a toilet with an average volume of 4.5 litres.

Regarding the energy balance in an installation with return circuits, the balance can be positive or negative, depending on the characteristics of the solution adopted for the return, the consumption diagram in the building, etc. There is an additional energy consumption when the return pumps are installed, but there is also an energy saving corresponding to the energy "imbibed" in the saved water.

3.3. Reuse and recycling of water

In fact, all the water we use has been recycled by nature. However, total recycling at the level of buildings is still not considered a priority technology, due to its costs, health risks and existing alternatives, although partial solutions already exist, with water recovery for non-potable purposes. The exception is, of course, isolated habitats such as space missions.

Israel and Singapore, for example, already carry out significant water recycling, although the uses for these waters are primarily agricultural and industrial. However, the National Water Agency of Singapore states that the regenerated water is already within the WHO (World Health Organization) and US-EPA (United States Environmental Protection Agency) requirements for drinking water [15].

Recycling is the only measure that, by itself, would allow, in theoretical terms, the design of nearly zero water buildings. In practice, however, 100% recycling of the water consumed is never feasible, as there is consumption for food purposes, irrigation, evaporation, etc., which do not allow the recovery of all the water consumed in the building. Even at the International Space Station (ISS) the recycling percentage varies between 85% and 93%, according to data from NASA (US National Aeronautics and Space Administration) and the Canadian Space Agency [16,17].

However, the reuse in buildings of water without faecal contamination, coming from discharges from bathtubs, showers, washbasins and, in certain situations, from washing clothes and kitchens, which are generically called greywater, is starting to become generalised. In general, the water from showers, baths and washbasins, called “light grey water”, is not very polluted. Water from washing machines usually has a higher pollutant level and water from the kitchen (sink and dishwasher) even greater (dark grey waters).

It is considered that greywater can be used for flushing toilets, washes and watering gardens, after adequate treatment in the case of systems with a long retention time. In systems with direct reuse or short retention time, with a single user or users from the same family, it is accepted that treatments can be simplified or even dispensed with, as long as there are adequate maintenance routines.

The total amount of greywater produced in a building can vary considerably depending on the habits of residents and their living standards. In the absence of specific studies, it is estimated that the production of greywater corresponds to about 70% of consumption, with 40% of light grey water and 30% of dark grey water [17,18]. For a capitation of 110 l/(inhabitant and day), the production of light greywater will correspond, therefore, to about 44 l/(inhabitant and day). This production is sufficient for toilet flushing, which corresponds to a value close to 35% of total consumption. At the limit, it is assumed that the use of greywater can reduce the consumption of drinking water by up to 50%.

With regard to the water–energy nexus in the reuse of greywater, it can be said that compact installations, with a short retention period and without treatment, also reflect energy savings, since the reduction in water consumption from the public network also corresponds to lower energy consumption in the urban water cycle. With regard to installations with a long retention time, with a "conventional" treatment for this type of water, the energy consumed in the treatment makes the system "neutral" from the energy point of view; that is, the energy spent in greywater treatment, about 1.8 kWh/m^3, is equivalent to the energy saved in the urban water cycle.

In the case of installations with a long retention time, with the usual treatment for this type of water, the energy consumed in the treatment makes the system approximately “neutral” from the energy point of view. In fact, considering that the aforementioned study carried out by ANQIP in the city of Aveiro led to an energy consumption in public networks of about 1.8 kWh/m^3, it appears that this value is equivalent to the energy spent in a conventional treatment of greywater. However, since the temperature of greywater coming from showers, for example, is generally above 30 °C, the use of this thermal energy for preheating domestic hot water allows savings of around 3 kWh/m^3, which can make these installations advantageous not only in terms of water efficiency, but also in terms of energy efficiency [18,19].

3.4. Resorting to alternative sources

The use of alternative local sources may include, for example, desalinated water or rainwater. For some uses, these non-potable waters can be used directly, without treatment, as is the case in Hong Kong, where salt water is used to flush toilets. Desalination can be an interesting solution in places with high freshwater scarcity and located next to salt water bodies. However, it is still not a generalisable solution in all cases, even on the coast, as it has some known drawbacks, such as the environmental impact of the brines (and sometimes chemicals) discharged to the sea and the high energy consumption [20].

As far as brines are concerned, they have a high concentration of salt and a higher density (and possibly also a higher temperature, particularly in thermal processes), which implies environmental risks at the point of discharge. Regarding energy consumption, values above 3 kWh/m^3 are generally indicated for the available reverse osmosis technologies, values that are still high when compared to the usual energy consumption in conventional systems (in the aforementioned study carried out for the city of Aveiro, Portugal, total energy consumption in the water supply network was 0.838 kWh/m^3).

The use of rainwater in buildings is a technology that responds not only to concerns about reducing the consumption of fresh water from traditional sources, but also to other situations that climate change makes more intense and frequent, such as, for example, exceptional precipitation events. In the latter case, rainwater harvesting systems in buildings contribute to a dampening of the flood peak, reducing floods in urban areas. Due to their numerous advantages, rainwater harvesting systems have seen a great increase in many countries, even being mandatory in some cities for some urban areas or for certain buildings (S. Paulo, for example).

In buildings, rainwater can be used for various non-potable purposes, such as flushing toilets, washing machines, outdoor irrigation, washing floors, etc. In general, these are competing uses with the reuse of greywater, but with fewer health risks and lower treatment requirements. Note that these water efficiency measures can be used cumulatively. It is assumed that rainwater can supply between 30% and 80% of consumption in buildings, without any complex treatment (apart from simple filtration and natural decantation in the tank) [21-23].

However, there are already individual equipments on the market for the potabilisation of these waters, so, in theory, a total supply of rainwater to the building will be feasible, completely replacing the supply from the public network, provided that the local precipitation is sufficient. Water potabilisation equipment for rainwater can also be used for water from local sources that are less polluted, such as wells and boreholes.

Rainwater harvesting systems, by reducing the consumption of drinking water in buildings, also reduce energy consumption in public networks. Although rainwater harvesting systems often require a pressurisation system in the building, the corresponding energy consumption is equal to or lower than when the supply comes from the public network.

4. CONCLUSIONS

Environmental sustainability reasons increasingly impose the construction of buildings that minimise the excessive consumption of resources that is observed today, through the use of local renewable sources, a circular use of resources, the reduction of waste, etc. The concept of “nearly zero buildings” is gaining added importance in this context.

Currently, “nearly zero energy buildings” are already a reality in many places on the planet, but it is necessary to move towards “nearly zero resources buildings” and to “nearly zero water buildings”, bearing in mind that these are the fundamental and interrelated resources for the sustainable development of humanity. These zero buildings, in addition to their contribution to sustainability in urban environments, also play an important role in mitigating and adapting to climate change.

With regard to “nearly zero water buildings”, the technologies currently available allow them to be implemented, although, from an economic point of view, their viability is still low in most situations. In any case, the application of different water efficiency measures in buildings, in accordance with the 5Rs principle, makes it possible to significantly reduce the consumption of fresh water from traditional sources and thus contribute to the conservation of this resource, essential to face the extreme drought situations, whose intensity and frequency has increased with climate change in many parts of the planet.

Another known effect of climate change is the increase in the frequency and intensity of extreme precipitation in some areas. However, some water efficiency solutions in buildings, such as the use of rainwater, also allow for lessening the effects of extreme precipitation, dampening flood peaks.

In general, all water efficiency measures in buildings contribute also to a reduction in energy consumption, which is reflected, as a rule, in a reduction of greenhouse gas emissions, such that nearly zero water buildings, in addition to their important role in environmental sustainability, also contribute significantly and in different ways to the mitigation and adaptation to climate change.

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Interpreting the technical versus the physical as drivers for shower water use

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ABSTRACT

Showering is informed by the person, their preferences and the affordances provided by the showering device and supporting systems. People shower for different reasons, including reasons of health, hygiene, wellbeing, leisure, relaxation among others. Studies have explored water use and efficiencies in showering in homes and other buildings. The lack of granularity of showering data can however mean that findings and deductions are typically made at the household or building level, rather than at the individual, shower end-user level. This study helps to address this gap about showering. This paper presents further analysis from an in-home trial with 12 adult participants: 6 male and 6 female, with a particular focus on interpreting the shower performance factors against the shower user. The findings highlight the contextual and physical characteristics e.g., shower positioning and user anthropometrics, and the showering needs (perceived shower functionality) as important drivers of water use.

Keywords: end-users, showerhead, showering, time, purpose, water efficiency

1. INTRODUCTION

Showerheads are commonly described as eco/low-flow, low-pressure gravity fed showers, power showers, electric showers, mixer showers, or pumped showers (Sadr *et al.* 2016). Water consumption in showers depends on the product design, type of shower control fixed/adjustable), spray pattern, hot water source and delivery mode, and even the pressure of the water droplets on the skin (Sadr *et al.* 2016; Adeyeye *et al.* 2020). The main water-saving shower technologies are efficient showerheads; alarm and time control devices, which also support the greatest energy saving; and digital feedback-based interventions. However, efficient technology does not guarantee a more sustainable use of water and can lead to a rebound effect (González-Gómez, López-Ruiz and Tortajada 2022).

Research studies increasingly show that socio-technical factors affect personal water use. Bathing (shower and baths) could account for up to 40% of domestic water use (Guven and Tanik 2018; Moslehi *et al.* 2020). Around 840 billion liters of water are used for showers each year, and people spend around £2.3 billion on heating water for these showers (EST 2015 in González-Gómez, López-Ruiz and Tortajada 2022), and the demand for separate shower enclosures continues to rise. Further, Makki *et al.* (2013); Ananga et al., (2019) highlighted that multiple individual factors determine shower end use consumption. However, research studies are often limited for reasons including privacy issues, variability of data sources, quality of

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feedback. Therefore, an efficient shower system combined with good showering attitudes, habits and behaviours can have significant impact on the domestic water and energy consumption.

This paper builds on previously published research on the Demystifying the Showering Experience Project (Adeyeye and She 2015 a & b; Adeyeye, She & Baïri 2017; Sousa et al., 2018; Ip, She & Adeyeye 2018; Adeyeye, She & Meireles 2018; 2020 etc.). This project was a comprehensive multi-stage study aimed at understanding the socio-technical aspects of the shower, including showering products, habits, behavior and perceptions. The empirical approach and experimental methods were used to understand design and user factors pertaining and influenced by water-efficient shower products by documenting and Analysing the physical, physiological, and sociological needs and requirements of the individuals involved.

The objectives were:

- To define and document the ambient conditions and characteristic variations in water efficient showerheads
- To investigate the possible correlations between these factors and the time spent in the shower i.e., shorter showers save water, and the amount of water used
- To start to define the conditions and range of acceptability of water efficient showerheads using physical and socio-psychological factors, which in turn can be used to determine the effectiveness of the product to promote sustained water efficiency practises.

2. MATERIAL AND METHODS

The research design is shown in Figure 1.

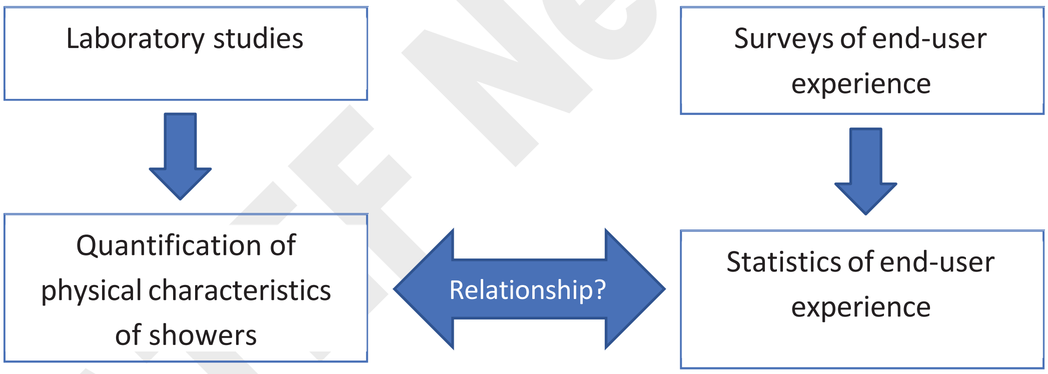

Fig. 1. Research design for the demystifying the showering experience project

The project was undertaken in stages:

1. **Laboratory experiments:** During this stage, controlled measurements of the physical characteristics and performance parameters of the showerhead at the point of water delivery e.g., on the human body were undertaken, in addition to commonly measured parameters (PRESSURE, flow rates etc.) at the point of supply.
2. **In-home trials to gauge performance experience and feedback:** A sampled set of 12 users identifying as 6 male and 6 females trialled 12 showerheads over 12 weeks; 10 eco-showerheads and 2 control showerheads. They were also provided with shower timers and tape measures. In addition to their feedback on their showering experience, data was obtained to evaluate the influence of anthropometrics, physical performance characteristics and psycho-social response. The user feedback was obtained through a showering activity record, and feedback sheets.
3. **Pre- and post-trial workshops:** Two workshops were held with each group at the beginning and end of the in-home trials. The first to introduce the goals of the project, discuss ethics, the study program and logistics, and obtain initial views and feedback. The second workshop was aimed at debriefing the findings, compared against the initial feedback, and understanding any issues and problems. Participants had the option of

a gift of one of the types of showerheads or a gift voucher. In all cases, all participants chose a showerhead.

The profile of participants is shown in Table 1. The showerhead profiles in Table 2**.**

Table 1. Demographic representation of participants. Source: Adeyeye, K., & She, K. (2015a)

		Gender				Gender	
		Female	Male			Female	Male
Adults in Household (Age_18+)	1	3	4	Children in Household	1	0	1
	2	2	1		2	1	2
Education	Bachelor Degree	1	0	Income	£20,000 - £29,999	3	2
	Currently studying	0	1		£30,000 - £39,999	1	0
	Further Education/ College	0	1		£40,000 - £49,999	1	0
	Posgraduate degree, Doctorate	5	2		£50,000 - £59,999	0	1
	Professional qualification	0	2		£60,000 or more	1	2
Employment_	Employed (full-time)	4	5	Relationship_	Divorced	1	0
	Employed (part -time)	1	0		Married or domestic partnership	3	5
	Student	0	1		Single, never married	1	1
	employed and student	1	0		cohabiting	1	0
Ethnicity	Asian/Asian British	0	1	Religion	All	1	0
	Mixed/Multiple ethnic groups	1	1		Christian (all denominations)	0	2
	Polish Catholic & Jewish	1	0		No religion	5	4
	White	4	4				

Table 2. Attributes of the 10 main eco-showerheads Source: Adeyeye, She and Bairi (2017)

Table 1
Attributes of the 10 sample showerheads.

Ref no.	S-01	S-02	S-03	S-04	S-05	S-06	S-07	S-08	S-09	S-10
Shape	Round	Oblong	Round	Round	Round	Round	Round	Rectangle	Curved rectangle	Round
Height	90	157	106	100	100	106	135	67	65	135
Width	90	82	106	100	100	106	135	182	120	135
Height incl. handle	215	270	239	230	230	239	246	227	219	246
Construction	ABS plastic with grey hard plastic faceplate	ABS plastic with grey soft plastic faceplate	ABS plastic with grey hard plastic faceplate	ABS plastic	ABS plastic	ABS plastic with grey, hard plastic faceplate	ABS plastic with grey, hard plastic faceplate	ABS plastic with grey, hard plastic faceplate	ABS plastic with grey, hard plastic faceplate	ABS plastic with white, hard plastic faceplate
Colour	Grey and chrome	Grey and chrome	Grey and chrome	Chrome	Chrome	Grey and chrome	Grey and chrome	Grey and chrome	Grey and chrome	White and chrome
Sprout type	Recessed twin	Recessed twin	Recessed twin	Protruding single soft rubber	Protruding single soft rubber	Recessed twin	Recessed twin	Triple central, recessed twin	Recessed twin	Recessed twin
Sprout layout	3 × 3 double sprout clusters	Two long double-sprout oval rows	Two concentric double sprout circles	Central core and radial rows	Central core and radial rows	3 × 3 double sprout clusters	Random x 3 clusters	Central triple clusters, random rows	Random	Random x 3 clusters
Inlet pipe connection (inch)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Working pressure (bar)	0.3–5.0	1.5–5.0	0.35–5.0	0.3–5.0	0.3–5.0	1.0–5.0	0.35–5.0	1.5–5.0	0.35–5.0	0.35–5.0
Measured regulated flow rate @ 2 bar pressure	10.3	7.2	7.2	9.2	8.7	5.1	11.3	7.2	8.1	9.6
Regulated flow rate @ 2 bar pressure	8.7	8.7	7.9	13.2	12.9	5.1	7.6	7.4	8.3	7.6
Unregulated flow rate @ 2 bar pressure	14.5	14.5	23.9	N/A	N/A	N/A	23.3	13.8	21	23.3
Number of functions	1	4	1	3	1	1	2	2	1	2
Mode of operation	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets	With Air	With Air	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets	Colliding twin jets that turn into thousands of tiny droplets
Additional comments		Supplied with 9 l/min flow regulator, includes 1 × vitamin C cartridge to neutralise chlorine	Supplied with 9 l/min flow regulator	Rub clean nozzles	Rub clean nozzles	Supplied with 5.7 l/min flow regulator fitted	Supplied with 9 l/min flow regulator, Two types of spray – Satin jet body shower or massage	Ergonomic slider function selection on handle, Supplied with 9 l/min flow regulator	Supplied with 9 l/min flow regulator	Supplied with 9 l/min flow regulator, Two types of spray – Satin jet body shower or massage
Image										

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3. RESULTS AND DISCUSSION

3.1 Results

The results presented in this paper focuses on the lowest (01 and 06) and highest ranked (04 and 07/10) showerheads. All analyses were conducted in SPSS. It is worth nothing that headline findings are presented. As with all studies of this nature, there are many explanations behind the headline data which cannot be fully covered within the scope of one paper.

1.1.1 Shower design and characteristics

Most technical standards are based on controlling flow rates as the significant factor for achieving water efficiency. However, this study found that the flow rate of efficient showerheads is not always directly proportional to shower durations or higher water use. More important are the perceived performance of the showerhead, typically on the body. First was the nature and mode of water delivery; showerheads that deliver water as a fine mist or spray, delivered asymmetrical spray distribution, had low flow intensity and significant heat loss all resulted in a comparatively higher shower duration, and lower user feedback. The feedback for the higher rated showerheads (04, 07/10), and lower rated showerheads (01, 06) are shown in Figure 2, and summarised below:

- Showerheads 01, 06 have similar design, same water delivery strategy (tiny droplets spray). The lab tests show an asymmetrical spray area, low flow intensity and significant heat loss. The users responded with comparatively higher average shower duration and lower feedback especially for spray coverage, enjoyability, ease of clean.
- Showerheads 04, 7/10 were preferred as they produced a well-defined area of high flow intensity and temperature distribution, were multi-mode showerheads – dual or three functions. The average shower duration was comparatively less. User feedback indicated it was because they could clean quicker and easier. Although, both showers were identical showerheads apart from the colour, the data returned different average shower durations. There are explanations which were explored during and after the study but falls outside the scope of this paper.

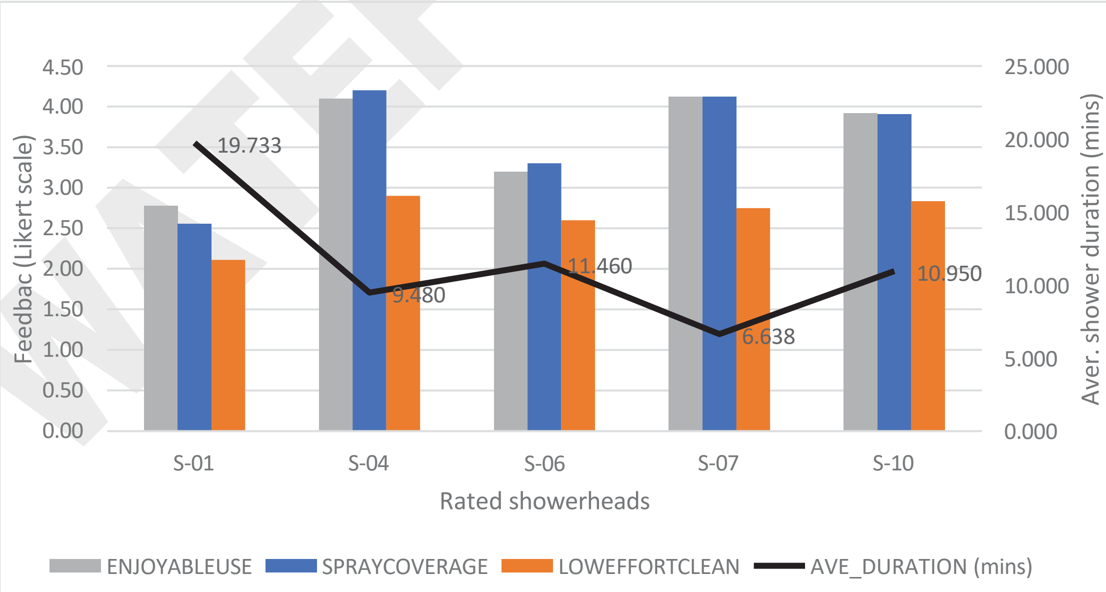

Fig. 2. A sample of showerhead duration and user feedback.

1.1.2 End-user characteristics and physical adaptation

Anthropometry, the measurement of body size, weight, and proportions, is an intrinsic part of health studies and is relevant as showering is a beneficial indicator for health, hygiene, and wellbeing. One of the main aims of this survey is to investigate the extent to which anthropometric data of this sample of adults is a dependent variable for showerhead positioning, performance preferences, shower duration and water consumption. In this study, anthropometric indices include the height and weight of the participants. This was compared with the positional height and standing distances to the showerhead. The anthropometric data were measured and declared by participants and were not verified by the researchers.

The non-parametric independent samples test showed that the distribution of showerhead height and distance are the same across the showerhead types (Figure 3). However, the opposite was the case when showerhead height and distance were compared against user height and weight. This suggests that the positioning of the showerheads in the shower enclosure is typically informed by the user's height and weight (n = 124; degree of freedom = 5; asymptotic sig. (2-sided test) < .001 in both instances) (Figure 4). Excluding factors pertaining to the physical environment e.g., the type and size of shower enclosure, the data suggests that the user height and weight, as well as the perceived performance in terms of flow, spray and distribution influences the positioning of showerhead, and to some extent the shower duration / water consumption (Figure 5). Some trends were observed for user heights of > 160cm and < 180 cm, and user weights of > 55kg, and < 75kg. However, no generalisations could be deduced.

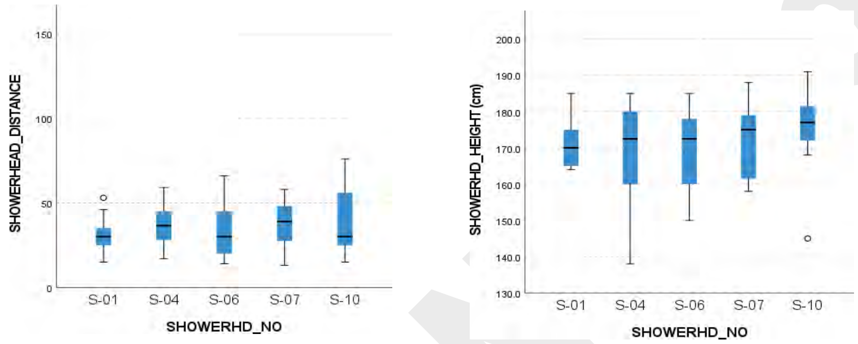

Fig. 3. Mean distribution of user-defined showerhead heights and standing distances (in cm).

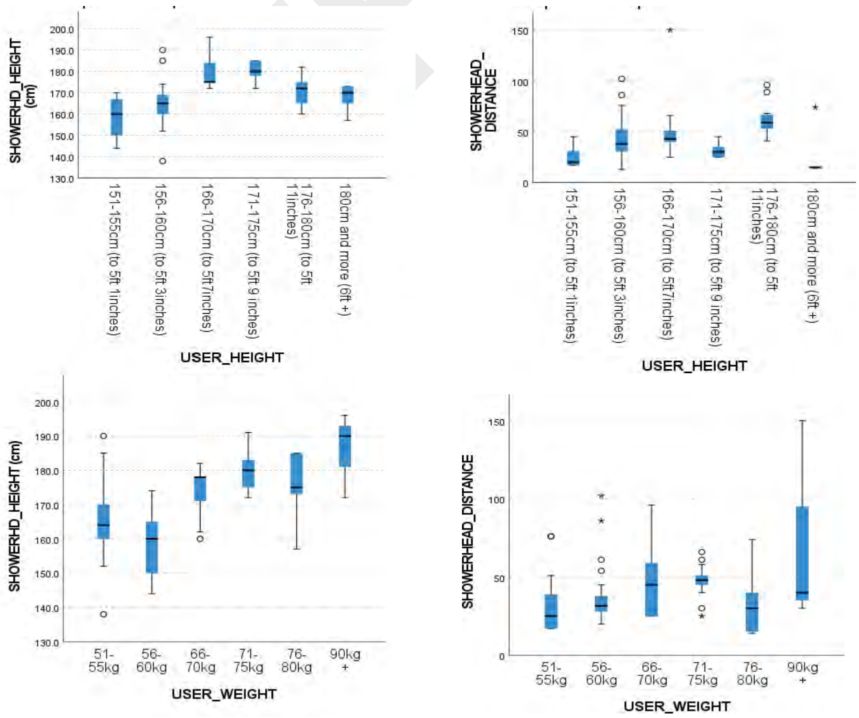

Fig. 4. Mean distribution of user defined showerhead heights and standing distances (in cm) compared with user-declared height and weight.

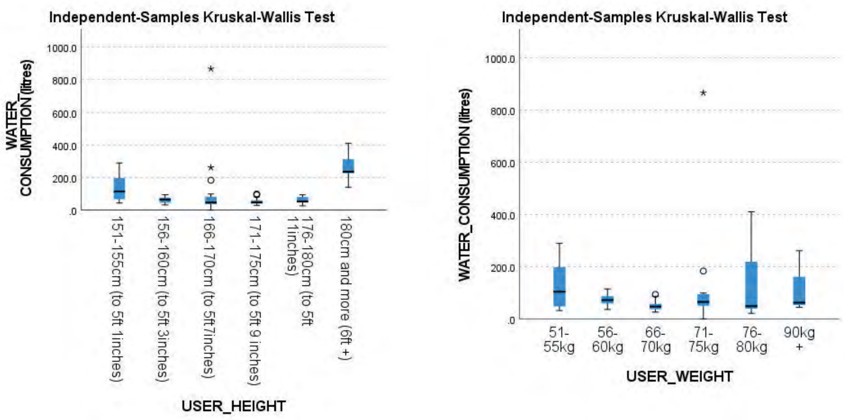

Fig. 5. Mean distribution of water consumption against user-declared height and weight.

1.1.3 Time, location, and purpose of the shower activity

This study found that the time of day, and week, as well as where showering takes can influence shower duration (Adeyeye and She 2015b). Further analysis was undertaken to understand the significance of the purpose of the shower – whether for hygiene, to refresh e.g., after physical exercise, or to relax e.g., after busy day at work. Unsurprisingly, an unhurried shower in the evening was more water intensive. However, this depended on the demographic. Figure 6 shows the instances according to age and declared gender. But this variation was also apparent where for instance, whether the participant was employed or student, or even the household make-up. Further investigations suggested that the evening groups were mostly female participants but again this depended on the forementioned conditions. It was also interesting to note that some participants undertook their main shower activity i.e., for hygiene in the evenings. Contrary to the common view that most people undertook hygiene showers in the morning. Therefore, these findings could have implications for water efficiency messaging.

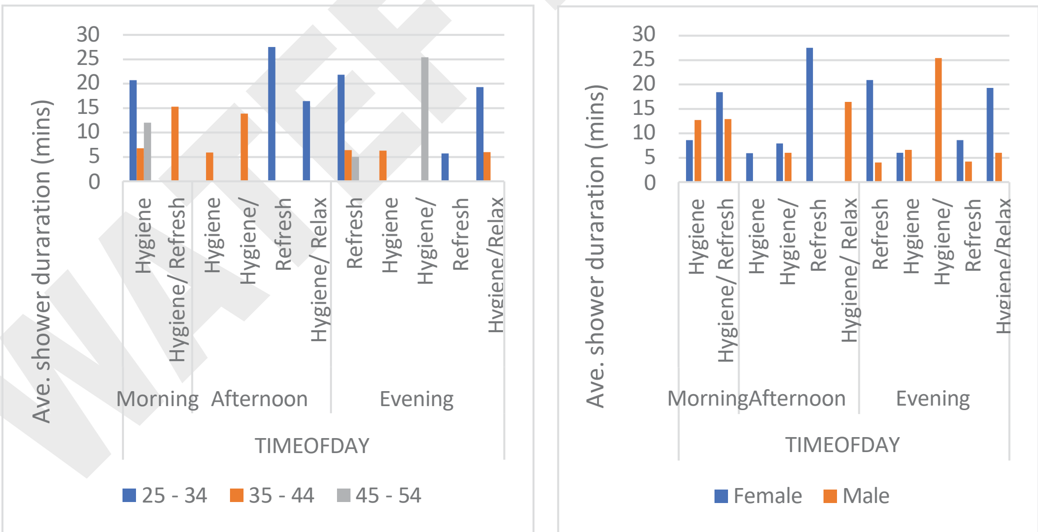

Fig. 6. Characteristic mean shower duration based on the time and purpose of showering activity.

3.2 Discussion

4. DISCUSSION AND CONCLUSION

The findings provide some interesting insights into the functionalities of showerheads versus the individual, their needs, preferences, experiences in the shower. Further analysis of the dataset from this study suggests that anthropometrics, the physical location of the shower, as well as the purpose of the shower could provide further insights into the social-technical understanding of showering and showering water use. Dissatisfaction in any physical or technical performance factor affects the shower experience and can result in longer showers and more water consumed in the process. However, several questions remain, and multiple contradictions are notable and more insights from larger scale, longitudinal studies are essential to be able to scale up the findings.

In the meantime, it is recommended that future water use, water efficiency studies or trials as a matter of routine include the collection and meta-analysis of these factors to create a more holistic picture of showering and general water use in buildings.

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COMPETING INTERESTS

Nothing to declare.

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Water Distribution Challenges in Northeast Trinidad and Tobago

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ABSTRACT

Provision of a reliable supply of water with adequate pressures is a paramount challenge for the water distribution network in Northeast Trinidad and Tobago. Customers in this region, served by the Water and Sewerage Authority of Trinidad and Tobago often have trouble in obtaining a pipe borne supply of water as they reside on areas of high elevation or at the farthest end of the distribution system. These areas where water supply challenges are a problem are Oropoune Gardens Piarco, Upper Five Rivers Arouca, Windy Hill Arouca, Edna Hill Arouca, Lillian Heights Arouca, The Foothills Arouca and Bon Air North that is made up of Pineridge Heights Housing Development and Ridgeview Heights Housing Development. Sources of supply for these impacted areas are the North Oropouche Water Treatment Plant, the Hollis Water Treatment Plant, Tacarigua Highlift Station and the Arouca Wells. Challenges in the water supply are a consequence of obsolete pipelines, numerous leaks, failure of mechanical equipment and the shortened duration of supply to the respective areas. The aim of this paper is to identify the inefficiencies and challenges of the water distribution system in this region by simulating flow through pipes using ANSYS Fluent simulation and the Hazen-Williams equation and advocate prospective solutions that can alleviate or even eliminate the inadequate water supply experiences faced by customers in these highly elevated areas.

Keywords: ANSYS Fluent, Distribution pipelines, Hard-hit areas, Hazen-Williams Equation, Northeast Trinidad and Tobago, Transmission pipelines, Water distribution

1. INTRODUCTION

Distribution systems are a series of interconnected components pipes and storage facilities responsible for transporting drinking water directly from the water treatment plants to the customers or from the source of supply to customers (U.S EPA 2021). The water transported is used for drinking and fire protection. It is also used for commercially and industrially purposes (AWWA 2003, 1). Generally, there are four types of water systems namely surface water system, groundwater system, purchased water system and rural water systems. In surface water systems, there is a water treatment plant and distributed system operated by a public utility that is supervised. The surface water is treated to remove particles and other contaminants and injected with a disinfectant. The treated surface water is transported using large transmission mains. In groundwater systems, the water may or may not be treated before entering the distribution system. Treated groundwater is an indication that the water contains iron and excessive hardness, making it unpotable. Additionally, groundwater systems that consist of several wells spaced around a distribution system eliminates the need for a large transmission system. Likewise, if there is a large quantity of groundwater in one area, a well field is installed. Well fields allow water to be collected and treated in one designated place allowing the water quality to be more easily controlled and in doing so, costs are reduced. On the other hand, purchased water systems entail one utility purchasing water from another utility. The advantage of purchased water systems is that good quality water is produced, and the water is accounted for using a bulk meter. Bulk metering incorporates the cost of water passing through the mains and includes that of leaks and wasted water. Purchased water systems consists of a greater-than-average storage capacity, as the source of water is a single source. Further to this, storage is practiced if there is either a limited rate of drawing water or if it is cheaper to purchase water during the night (AWWA 2003, 3-4). Conversely, rural water systems are a water district serving communities and rural homes widely spread

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where there is no groundwater, or the water quality is very poor. Water is drawn from a surface water source or a groundwater source and provided to customers living many miles away. The disadvantage to this type of system is that the pipelines are not large enough to supply water to fight fire requirements.

The problem of water distribution is a major challenge worldwide, including Trinidad and Tobago. Water is a precondition for human existence and sustainability for the planet (UN 2021). Trinidad and Tobago is one of the most developed countries in the Caribbean (European Commission 2020) and just as many other regions in the country, Northeast Trinidad and Tobago is quickly developing both socially and economically in the areas of industry, commerce and domestically. These developments result in population growth as there is a migration of people to the Northeast region. Migration and population growth go ‘hand in hand’ and in turn results in the increased demand for water, and in some cases the demand also exceeds the supply.

The Water and Sewerage Authority (WASA) of Trinidad and Tobago is exclusively responsible for extracting, treating and distributing potable water to customers throughout the country. Water is extracted from surface water sources, groundwater sources, and rural intakes. Northeast of the island consists of two large surface water sources; the North Oropouche River and the Hollis Dam. Water is withdrawn from the North Oropouche River by the North Oropouche Water Treatment Plant, is treated and distributed to customers. The water from this plant supplies most of the areas in Northeast Trinidad from as far as Valencia to Five Rivers, Arouca. The Hollis WTP on the other hand withdraws water from the Hollis Dam, treats the water and then distributes it to customers from areas as far as Arima to Bon Air North Arouca.

2. AIM AND OBJECTIVES

The aim of this paper is to identify the inefficiencies and challenges of the water distribution system in specified areas of the Northeast Region most affected. These areas are those of the highest elevation and are at the farthest end of the transmission and distribution system. The objectives are:

1. To investigate the flow of water in pipes using ANSYS Fluent Fluid software.
2. To investigate the frictional pressure drops occurring in pipes using the Hazen-Williams equation.
3. To recommend prospective solutions that can alleviate or eliminate the inadequate supply of water from observations after actual troubleshooting of the system.

3. SITE DESCRIPTION

The selected study sites, all located Northeast of the island are:

1. Oropune Gardens, Piarco
2. Upper Fiver Rivers, Arouca
3. Windy Hil, Arouca
4. Edna Hill, Arouca
5. Pineridge Heights Housing Development, Bon Air North, Arouca
6. Ridgeview Heights Housing Development, Bon Air North, Arouca
7. Lillian Heights, Arouca
8. The Foothills, Arouca

All eight areas are located at the farthest end of the distribution system and include customers residing on extremely high elevations where water must be supplied and is expected to reach.

3.1.1 Oropune Gardens Piarco

Oropune Gardens Piarco, is a housing development, south of the Churchill Roosevelt Highway and Northwest of the Piarco International Airport. It was established by the Housing Development Corporation of Trinidad and Tobago. The source of water to this area is the North Oropouche Water Treatment Plant. Customers reside both in low rise buildings and mid-rise buildings in the area. The challenge experienced with this development is the supply of water to suffice customers, especially those residing on the top floors of the mid-rise buildings that are located at the farthest end of the Oropune distribution system.

3.1.2 Upper Five Rivers Arouca

Five Rivers consist of a few areas of extremely high elevation, referred to as ‘Upper Five Rivers’. These highly elevated areas are:

1) Manimore Road
2) Bertie Road (inclusive of Bertie Road Extension)
3) Manoram Road
4) Mission Road
5) William Hill
6) Spring Road (inclusive of Spring Road Extension)

The challenge with this area is that it is extremely difficult to supply water to these areas on a scheduled basis because of inadequate water supply pressures, as the respective areas are at located at the farthest end of the distribution system.

3.1.3 Windy Hill and Edna Hill Arouca

Windy Hill and Edna Hill, Arouca is supplied by the Arouca High Lift Station. The Arouca High Lift Station is a mixture of water from Hollis Water Treatment Plant and the Arouca wells. Both areas are located off the Arima Old Road and are highly elevated.

3.1.4 Pineridge Heights and Ridgeview Heights Housing Developments, Arouca

The Pineridge Heights Housing Development (HD) and Ridgeview Heights Housing Development, both establishments of the Housing Development Corporation of Trinidad and Tobago are located Bon Air North, Arouca. The source of supply to this area is strictly the Hollis Water Treatment Plant. Like Oropune, this area also consists of low rise and mid-rise buildings, in addition to the area being highly elevated.

3.1.4 Lillian Heights and The Foothills, Arouca

Lillian Heights and The Foothills are both nearby residential communities also receiving water from the Arouca High Lift Station and are also off the Arima Old Road. Lillian Heights is a mixture of both low-rise and mid-rise buildings and is still being developed as similar buildings and developments within the area, are being constructed.

4. METHODOLOGY

4.1 Customer Statistics, Sources of Supply and Production

A site visit to the respective hard-hit areas was made and the numbers of houses were physically counted. Then, the total number of customers for each area was calculated by multiplying the number of houses by 3.3, the average household size (Trinidad and Tobago CSO 2011, 1). See Table 1. Following this, terrain on Google maps was used to determine the elevations for each hard-hit area and the required distribution pressure for the respective area was calculated by multiplying the distribution pressure by 0.7 m head of water, the equivalent unit of measuring water pressure (RIC 2020, 1). See also Table 1. The water demand was then calculated by multiplying the total number of customers by the per capita water demand used by WASA, that is 82 U.S GPD and depicted in Figure 1. In essence, for each of the hard-hit areas, the following was determined and listed in Table 2.

(i) The source of supply.
(ii) The distance of the hard-hit area from the source of supply.
(iii) Size and type of distribution main.
(iv) Minimum pressures in the water service connection to reach customer.
(v) The type of supply.
(vi) Scheduled supply
(vii) The class of supply
(viii) Customer class

4.2 ANSYS Fluent

ANSYS (Fluent) Fluid software is used for modeling fluid flow (ANSYS 2022). In this study it was used for simulating the flow of potable water through crucial transmission and distribution pipelines supplying water directly to the hard-hit areas. Pipelines ranging from sizes 1100 mm, 200 mm and 150 mm were drawn using ANSYS Fluent 2021 R1 Fluid workbench and meshed (ANSYS 2016, 1). The operating conditions selected for simulation was atmospheric pressure including the selection standard reference values for water. Images illustrating the simulation of wall shear stress and total pressure inside the each of the respective pipelines were generated. See Figures 2 to 7.

4.3 Pressure Drop in Pipes using Hazen-Williams Equation

The Hazen-Williams equation is used for designing of water distribution lines (Menon 2004, 50). It was used for calculating the frictional pressure loss of water in the crucial pipelines. The Hazen-Williams equation is:

$$P = [4.52Q^{1.85} \div (C^{1.85}\ x\ d^{4.87})]$$

Where: P= Friction loss per foot of pipe (psi); Q= Flow rate (gpm); d= Internal Pipe Diameter (inches); C= Hazen-Williams coefficient (Menon 2004, 50).
Google Maps was used to measure the distance of the hard-hit areas from the source of supply as 'the crow flies'. Using the distances measured, along with the crucial pipe diameters of 1100 mm, 200 mm, 150 mm, the Hazen-Williams constant of 150 for PVC pipes, and the flow rate from the source of supply, the friction loss in the respective pipes was calculated. The results were compared to the elevations and displayed in Figure 8.

5. RESULTS

Table 1 Customer statistics

Street Name	Population	Elevation (m)	Required distribution main pressure (kPa)
Manimore Road	366	80	386
Bertie Road	92	90	434
Bertie Road Extension	50	50	241
Manoram Road	581	90	434
Mission Road	155	100	483
William Road	208	90	434
Spring Road	228	120	579
Spring Road Extension	168	120	579
Windy Hill & Edna Hill	1389	70	338
Oropune Gardens	403	0	0
Pineridge Heights Housing Development (HD)	750	80	386
Ridgeview Heights Housing Development (HD)	750	90	434
Lillian Heights Housing Development (HD)	248	90	434
The Foothills Housing Development (HD)	175	80	386

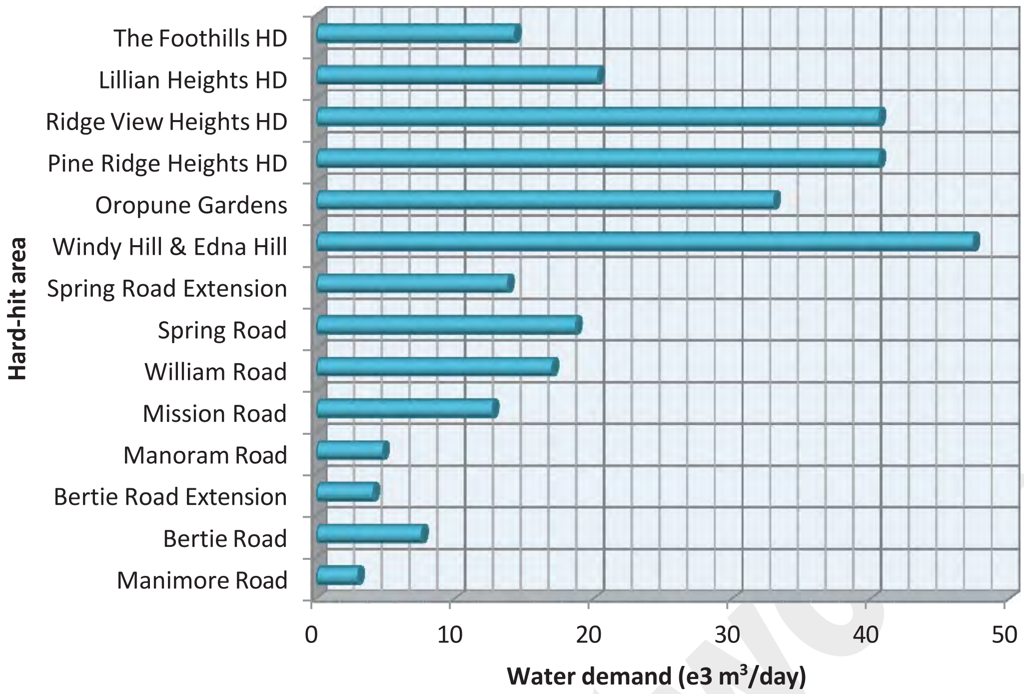

Figure 1 Water demand for hard-hit area

Table 2 Areas in Northeast Trinidad and Tobago affected by an inadequate supply of water

Essential Parameters	**Impacted Area**					
	Oropune Gardens, Piarco	**Upper Five Rivers, Arouca**	**Windy Hill & Edna Hill, Arouca**	**Pineridge Heights & Ridgeview Heights Housing Developments**	**Lillian Heights, Arouca**	**The Foothills, Arouca**
Source of supply	North Oropouche Water Treatment Plant	Tacarigua High Lift Station	Arouca Well field & Hollis Water Treatment Plant	Hollis Water Treatment Plant	Arouca Well field & Hollis Water Treatment Plant	Arouca Well field & Hollis Water Treatment Plant
Type of Source of Supply	Surface water	Groundwater	Impounding reservoir and groundwater	Impounding reservoir	Impounding reservoir and groundwater	Impounding reservoir and groundwater
Water Production of Source of Supply (U.S MGD)	26.4	3.5	0.52	10.2	0.52	0.52
Distance of source of supply from Hard-hit area (km)	29	30.5	1.11	0.736	2.9	2.6
Size and Type of Distribution Main	200 mm PVC	150 mm PVC	150 mm PVC	200 mm PVC	150 mm PVC	150 mm PVC

Booster Station required to boost pressures	No; Gravity flow	Yes; Khandahar Booster Station	Yes; Arouca High Lift Station	Yes; Limpet Booster Station	Yes: Arouca High Lift Station	Yes; Arouca High Lift Station
Minimum discharge pressure on distribution main required to reach customers water service connection (kPa)	414	1724	345	414	276	965
Minimum water pressure required at point of water service connection (supply pressure) (kPa)	138	138	138	138	138	138
Type of Supply	Intermittent	Intermittent	Intermittent	Intermittent	Intermittent	Intermittent
Scheduled supply (hr./day)	12/3	12/3	12/3	12/3	12/3	12/3
Class of supply	4	4	4	4	4	4
Customer Class	A4	A3	A3	A4	A4	A4

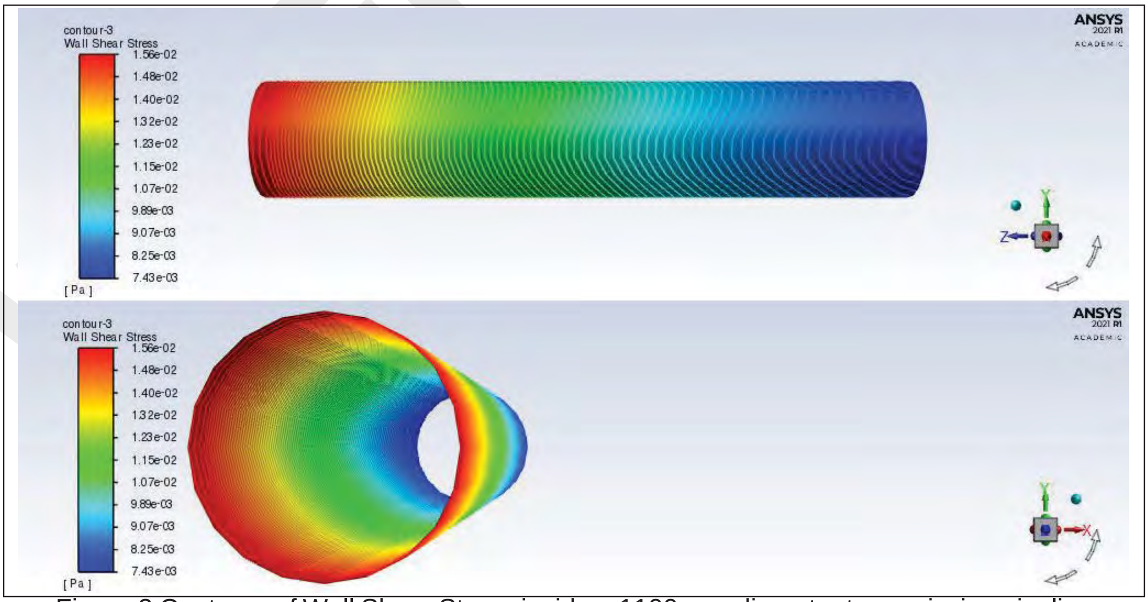

Figure 2 Contours of Wall Shear Stress inside a 1100 mm diameter transmission pipeline

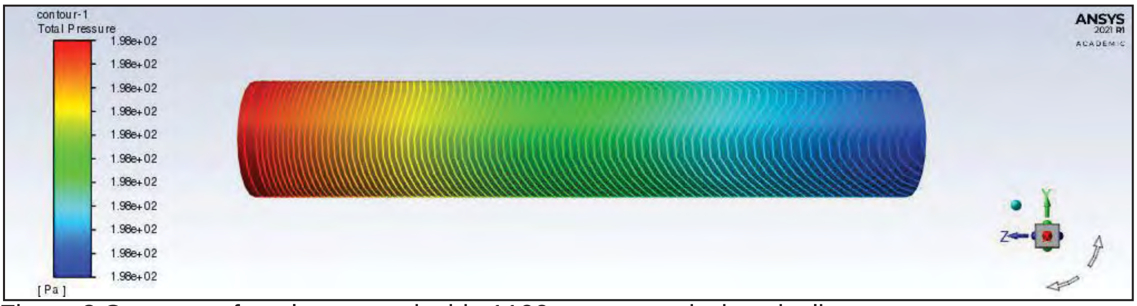

Figure 3 Contours of total pressure inside 1100 mm transmission pipeline

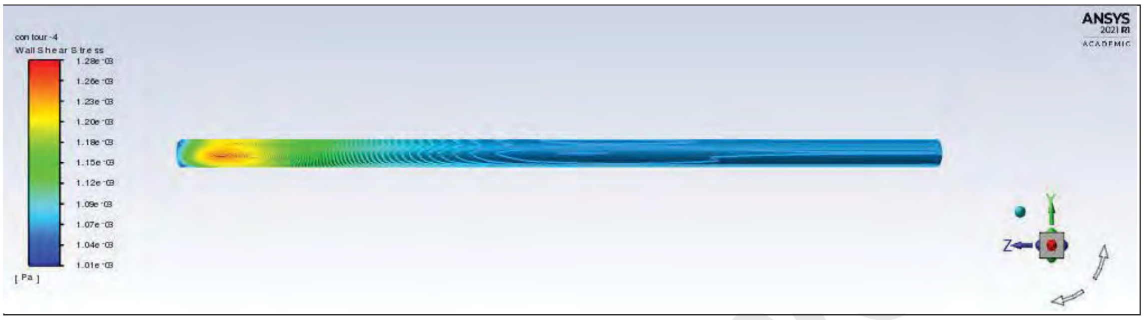

Figure 4 Contours of wall shear stress inside a 200 mm diameter distribution pipeline

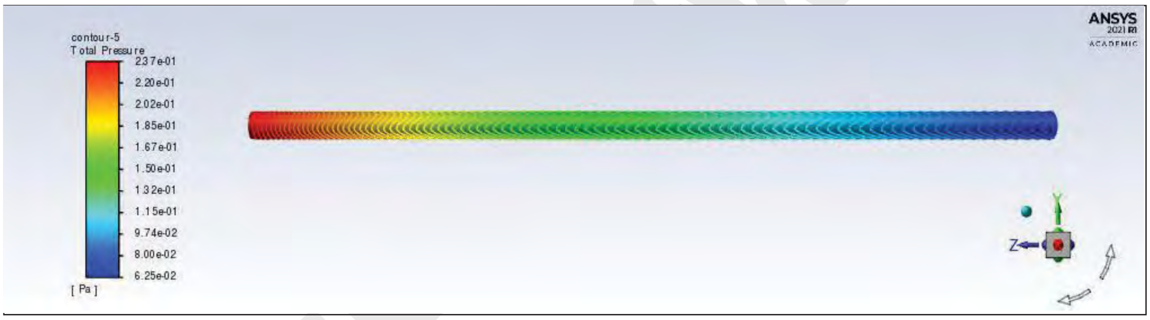

Figure 5 Contours of total pressure inside a 200 mm diameter distribution pipeline

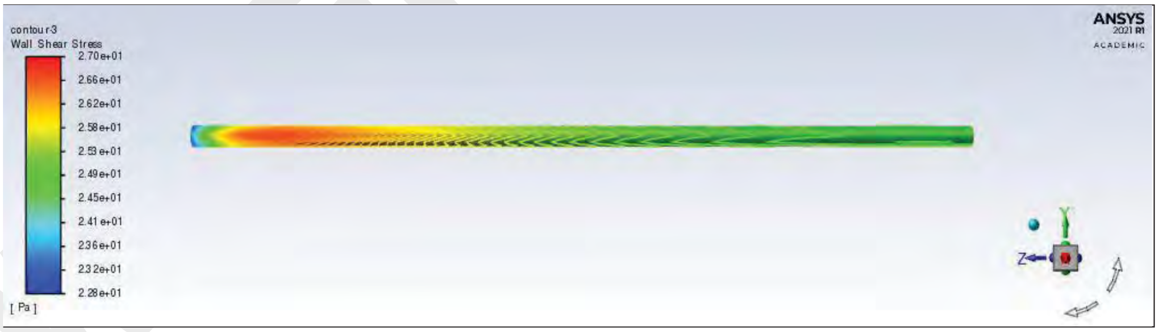

Figure 6 Contours of wall shear stress inside a 150 mm diameter distribution pipeline

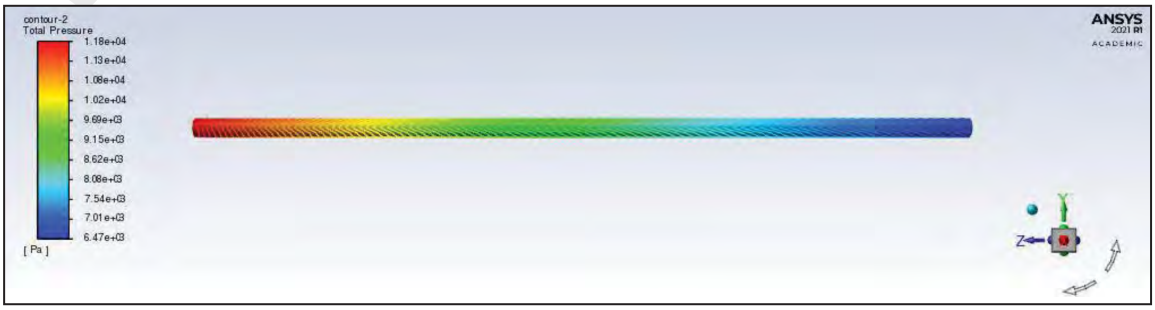

Figure 7 Contours of total pressure inside a 150 mm distribution pipeline

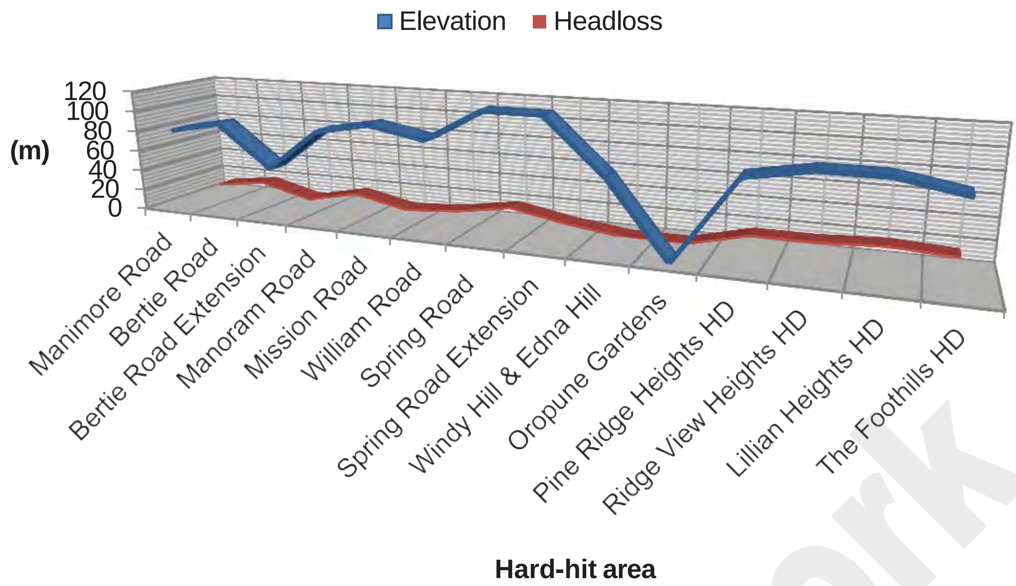

Figure 8 Head loss compared to elevation for hard-hit area

6. DISCUSSION

The areas focused on in this research are deemed as 'hard-hit' areas. 'Hard-hit' areas, as perceived by the author are areas severely impacted by a deficient supply of water because they are either highly elevated or are located at the farthest end of the transmission and distribution system. The water distribution networks investigated in this study are looped networks. Water distribution difficulties to the hard-hit areas all experience one common dilemma that is, inadequate pressures. Inadequate pressures are water pressures in the distribution system that is too low causing the supply to not reach the customers in the hard-hit areas. In other words, the pressures occurring in the respective pipelines are not the standard pressures or design pressures required to supply customers. In Table 1, the number of customers ranged from 50 to 1389 respectively, whereas elevation ranged from 0 m to 120 m respectively and require minimum pressures ranging from 0 kPa to 579 kPa respectively. Furthermore, the per-capita water demand illustrated in Figure 1 for each hard-hit area ranged from 3 e3 m^3/day to 47 e3 m^3/day. Per capita demand and water supply goes 'hand in hand' as the supply must be sufficient to satisfy the demand. For this reason, the author found it necessary for it to be included in the study.
Moreover, Table 2 provided a more detailed analysis of the hard-hit areas, including identification of customer class. 'A3' customers are domestic customers whose premises are fitted with internal plumbing whereas 'A4' customers are domestic customers whose premises are also fitted with internal plumbing but are metered (Public Utilities Commission of Trinidad and Tobago 2008, 3-5). Table 2 also stated the source of supply and the total water production supplied to said areas. The volume of water available for supply to customers are dependent on the latter two. Adding to the importance of supplying water to the hard-hit areas is also dependent on the distance of the customer from the source of supply, the diameter of the distribution mains and the importance of a booster station. Pipe diameter plays an important role in the flows and pressures occurring in the crucial pipelines. The minimum water pressure required at the point of water service connection is 20 psi (Regulated Industries Commission 2020, 3). This is approximately 138 kPa. 'psi' (pounds per square inch) is a unit of measuring water pressure and is equivalent to 0.7 m head of water. If the required pressures listed in Table 1 are attained in the distribution pipelines, then it is guaranteed that a minimum 138 kPa would be provided to customers.
A critical contributing factor that must be taken into consideration for the highly elevated areas, is that the required discharge pressures can only be attained when the low-lying areas are first fully saturated. 'Fully saturated' low-lying areas means customers on the low elevations, in particular elevations of 0 m must first receive an adequate water supply at 138 kPa.

Continuation of investigating the water distribution challenges in the Northeast region was done using simulation. Simulation of flow in the major transmission and distribution pipelines using ANSYS FLUENT (Fluid) were performed for the author to have a better sapience of the flow of water in the pipelines. The 1100 mm diameter transmission pipeline transporting treated water directly from the NOWTP tank was analysed and shown in Figure 3. The depiction of wall shear stress in Figure 3 was seen to vary along the length of the pipeline. Closer to the source of supply, the wall shear stress was the highest, but as the water travels farther away from the source, shear stress decreases. Shear stress was indicated by the colour change from blue to red. The red colour indicates a lower shear stress in the pipes as the distance from the source of supply increases. In addition, it proves that the farther away from the source of supply, shear stress tends to decrease. Similar occurrences were observed from simulating total pressure in the pipes. This occurs when the water travels farther along the transmission and distribution pipeline system, hence the reason why customers at the farthest end of the distribution system tend to experience water supply challenges. In some instances, to aid in combating this problem, high lift stations and booster stations are strategically placed along the transmission and distribution systems with the aim of boosting pressures. The same were depicted in Figures 4 to 7 for pipelines of diameters 200 mm and 150 mm respectively. Pressure drops due to friction in the pipelines occur as this transmission pipeline is a long distance one (Menon 2004, 57). Once the respective pressures are attained, then the flow will be achieved as pressure and flow are directly proportional to one another.

Oropune Gardens, unlike Upper Five Rivers is a low-lying area with an elevation of 0 m. Once the supply is switched on and the tank height at the source of supply is maintained, the water reaches the customer at the farthest end of the distribution system. Ridgeview Heights and Pineridge Heights Housing Development on the other hand, receive a supply from the Hollis WTP. Once the adequate supply reaches the Limpet Booster Station, only then will the booster attain the designed suction and discharge pressures.

The Hazen Williams equation was used to calculate the head loss occurring in the distribution pipelines in the respective areas. Elevation ranged from 0 m to as high as 120 m whereas the head loss occurring in the pipes ranged from 0 m to 17.4 m respectively. Figure 8 that depicts the area elevation compared to head loss. A maximum head loss of 17.4 m in the pipes is indicative and proof that friction losses in the pipes can be considered minor. This usually occurs in long-distance pipelines in which pressure drop due to friction in the pipeline contribute to a significant portion of total fricitional pressure drop (Menon 2004, 57). Once there is friction losses occurring in pipes, and then there will be a loss of energy of the water flowing in the pipes. Friction losses in other words, are an indication of resistance in pipes. Figure 8 also shows that head loss is greater in the pipeline of smaller diameter. Friction losses vary according to the pipe diameter and is due to the converging and diverging boundaries in pipes. The distance of the source of supply to the pipe is also a major factor affecting friction loss in pipes. During gravity flow, friction loss will be greater and flow rate will be smaller. However, as seen in Figure 8, friction loss in pipes is extremely small and can be considered negligible.

All in all, actual system trouble shooting and observation, in addition to flows, pressures and frictional loss in pipes, revealed that the following mitigation measures are critical to ensuring an efficient supply of water is provided to customers:

1. Maintaining an adequate tank height from the source of supply as it would affect flow and pressures in the pipelines.
2. Closing all off-takes or crucial valves along the distribution system and in particular those on the transmission system. This would also enable respective targeted flows and pressures to be achieved.
3. Though not complex, the water distribution networks in this study are a mixture of ageing infrastructure and customer demands varies according to customer type. Obsolete infrastructure results in leaks as there are often ‘weak points’ on the transmission and distribution pipelines. ‘Weak points’ are areas on the pipeline that are unable to withstand the flows and pressures in the pipelines and thereby break or crack gradually or immediately, according to the material type of the pipeline, resulting in leaks. Currently, the region has a total of 1018 leaks (WASA 2022). The number of leaks is dynamic and changes on a daily and weekly basis and add to the volume of unaccounted-for water. The solution is to maintain the number of leaks existing by excessive leak repair exercises taking into the consideration and not forgetting the

order of priority leaks is to be repaired. Therefore, leakage is significant reason for an inadequate supply of water to the highly elevated customers as well as those farthest on the distribution system. With less leaks, the pressures and flows in pipes will be greater.

4. Preventative maintenance practices of new and existing booster stations and highlift stations.
5. In addition, another possibility is the consideration, identification and utilization of an alternative source of supply.

7. CONCLUSION

In conclusion, the dilemma of an inadequate supply experienced by customers on highly elevated areas and those at the farthest end of the transmission and distribution system, although continues to occur, can be solved. Maintaining the flows and pressures begins from the source of supply and can continue along the transmission and distribution system if all mitigation measures mentioned earlier are adhered to. Also, water distribution challenge is a 'work in progress' but close monitoring and maintenance of the transmission and distribution system can definitely alleviate and even eliminate water supply problems to hard-hit areas in Northeast Trinidad and Tobago.

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Impact of climate changes on domestic hot water consumption

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ABSTRACT

Domestic hot water consumption is the second largest source of energy consumption in residential buildings. Moreover, the growing trend towards more energy efficient buildings, from a thermal performance point of view, is increasing its relevance. As such, more accurate modeling of domestic hot water consumption is required, including the consideration of the impact of climate change. A new approach that accounts for the variability of the domestic hot water consumption and cold-water temperature throughout the months is used to forecast the amount of energy and carbon emissions associated. It was found that in climate change context, despite the forecasted increase in air temperature in the summer months, the decrease in the remaining leads to higher domestic hot water consumption and, proportionally, higher energy and carbon emissions.

Keywords: domestic hot water consumption, energy consumption, carbon emissions, climate change

1. INTRODUCTION

Climate changes have resulted in increases in globally-averaged mean annual air temperature and variations in precipitation, and these changes are expected to continue and intensify in the future [1]. According to the Intergovernmental Panel on Climate Change (IPCC), each of the last four decades has been successively warmer than any decade that preceded it since 1850. Global surface temperature in the first two decades of the 21st century (2001–2020) was 0.84 to 1.10°C higher than 1850–1900. The report also projected that global surface temperature will continue to increase until at least mid-century under all emissions scenarios considered, and global warming of 1.5°C and 2°C will be exceeded during the 21st century [2].

The impacts of climate changes will be felt mostly through changes in intensity and frequency of extreme events, both in relation to precipitation and temperature. Heavy precipitation events will intensify and become more frequent in most regions. At the global scale, extreme daily precipitation events are projected to intensify by about 7% for each 1°C of global warming [2]. Despite the increase of heavy precipitation, a reduction in the wet day's probability is expected, which implies higher probability of droughts [3]. Extreme hot days in mid latitudes will warm by up to about 3°C, at global warming of 1.5°C, and about 4°C at 2°C, and extreme cold nights in high latitudes warm by up to about 4.5°C at 1.5°C and about 6°C at 2°C [2].

Regarding extreme temperature related events in the Iberian Peninsula and Mediterranean region, the frequency of "heatwave days" is projected to increase from an average of about two

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days per summer for the period 1961–1990 to around 13 days for 2021–2050 (and 40 days for 2071–2100) [4, 5].
Extreme weather conditions, such as floods and droughts, are the main impacts on water availability for human consumption. In addition to these quantitative impacts, climate changes can also affect water quality, as temperature is known to affect chemical and microbiological processes within the distribution phase [6, 7].
Climate change will also play a role on water end-use within the buildings, namely in terms of domestic hot water (DHW) consumption and related energy use and carbon emissions.
Meireles et al. [8] found that the proportion of DHW consumption in residential buildings is proportional to the air temperature. Furthermore, cold-water temperature in public water distribution networks is also affected by the air temperature. Consequently, the energy used to produce DHW and the corresponding emissions are expected to be affected by climate changes. This is explored in the present communication for the region of Algarve, located in the south of Portugal.

2. MATERIAL AND METHODS

The estimation of the amount of DHW consumption and corresponding energy use and carbon emissions is done by simulation. The approach used is depicted in **Fig. 1**.

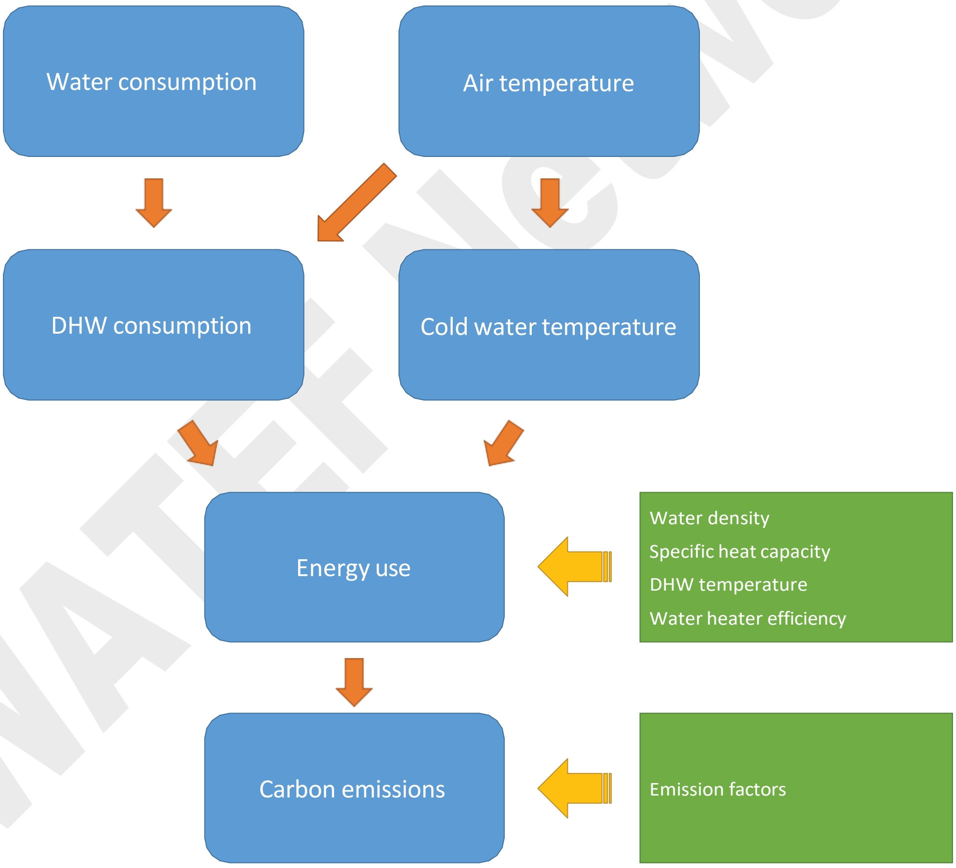

Fig. 1 – Calculation methodology

The monthly water consumption was provided by the water utility for the years of 2017 and 2018, with the average value used herein (**Fig. 2**). Overall, the water consumption in 2018 was only 5.6% lower than in 2017 and it is assumed to be a good approximation of the consumption in the future.

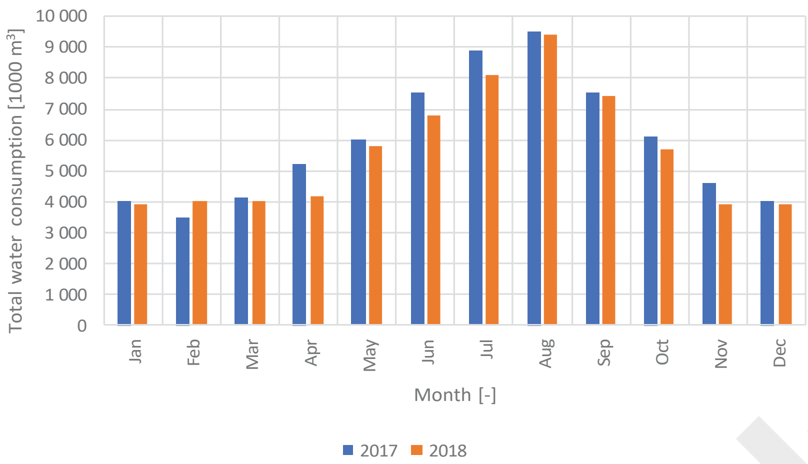

Fig. 2 – Observed total water consumption

The monthly air temperature was obtained from the Portuguese meteorological services, IPMA (Instituto Português do Mar e da Atmosfera). Long series of observed air temperatures (1966 to 2018) and forecasted air temperatures (2041-2070) were retrieved and averaged (**Fig. 3**). The observed trend in observed and forecasted data is neglected in this study. The forecasted air temperatures correspond to the ensemble results of both the Global and Regional models for the 8.5 Representative Concentration Pathway scenario obtained on the scope of the AdaPT program [9].

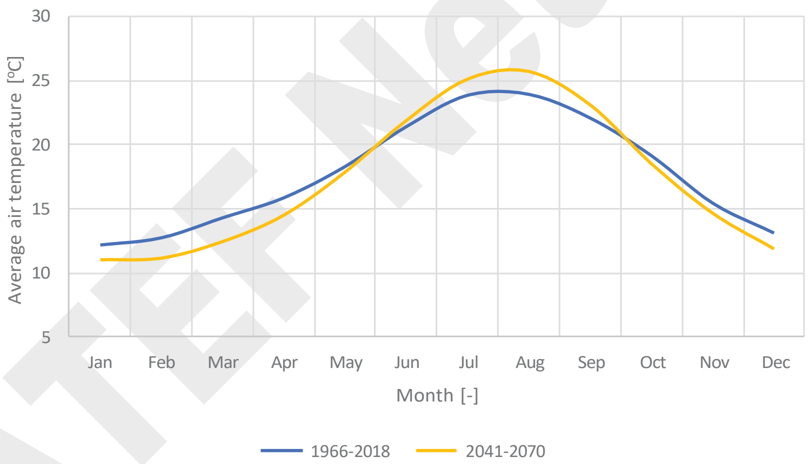

Fig. 3 – Observed and forecasted monthly average air temperature

The proportion of DHW consumption was estimated using the model developed by Meireles et al. [8] and the calculation of the corresponding energy and carbon emissions followed the approach detailed in Sousa and Meireles [10]. The cold-water temperature was estimated using the model developed by Hendron et al. [11].

3. RESULTS AND DISCUSSION

Despite the higher temperatures forecasted for the summer months, the lower temperatures in most of the other months leads to an estimate of higher DHW consumption in the future. The monthly DHW consumptions are detailed in **Fig. 4**, resulting in an annual increase of 106×10^3 m^3.

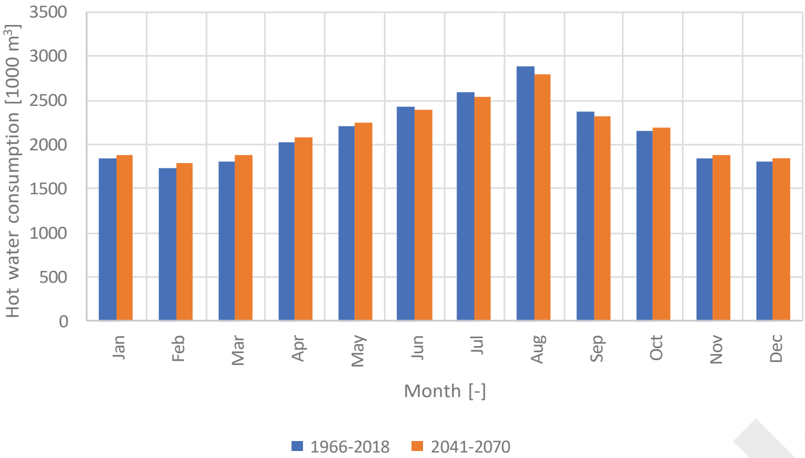

Fig. 4 – Estimated DHW consumption presently and in the future

Fig. **5** presents the energy use and carbon emissions estimations for the present and future, with a forecasted increase of 11 613 MWh and 2 825 tons, respectively. The combined difference in DHW consumption and cold-water temperatures results in a 2% increase, significantly more than the 0.4% in terms of DHW.

Fig. 5 – Estimated energy use (top) and carbon emissions (bottom) presently and in the future

4. CONCLUSION

Considering that DHW production is already the second largest contributor for the total energy consumption in buildings, and that its relevance will tend to growth with the increasing thermal performance of new and refurbished buildings, its assessment is of outmost importance. The energy consumption for DHW production depends on the cold-water temperature. Therefore, air temperature changes expected due to climate changes, in the future, will have an impact on the energy and carbon emissions associated to the DHW consumption.

In this work, an estimation of the energy and carbon emissions due to DHW production in the region of Algarve (Portugal) for a climate change scenario is determined. Analysis of the air temperature shows that higher temperatures are expected in summer months. However, lower temperatures are predicted in most of the other months. Therefore, a higher DHW consumption is expected in the future, and consequently the energy used and the carbon emissions.

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Water Demand Modelling and Analysis United Kingdom, North American and Sri Lankan Data

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ABSTRACT

The aim of this research is to study real water consumption data to determine the most appropriate statistical distribution function to address the peak water demand. Furthermore, the study is expected to contribute to finding a better fitting water demand model, which could apply to any water network. To achieve the objective, this study selected water usage of three different countries with diverse socio-economic backgrounds, climate, and geography to get an overall picture of water usage patterns. The countries selected for the analysis were United Kingdom, North America, and Sri Lanka. The most widely used probability distribution functions to represent a continuous random variable such as normal, log-normal, exponential, logistic, log-logistic, 3- parameter log-logistic and Weibull were applied to comprehend the suitability of fitting. The normal, log-logistic and 3- parameter log-logistic distributions are suitable to represent demand data with lower and high demand values and were selected for further analysis and is described here to provide their suitability for modelling water demand.

***Key words**: uncertainty in water demand, log-logistic distribution, 3- parameter log-logistic distribution, probability distribution function, sustainable design*

1. INTRODUCTION

Water demand varies due to various factors, such as time of the day, social background, availability of sources, weather conditions and firefighting. More often water demand comprises of extreme demand values and simply cannot be applied normal distribution to model demand data. Techniques and methodologies for water demand modelling are widely available in the literature for developed countries; nevertheless, information based on real data is often unavailable for designers and planners to get confidence of deciding on future demand. Methods to estimate future water demand affecting the water distribution systems are fundamental, if robust and resilient management are desired. Future water demand is the main driving force in water system management [1]. Urban demand modelling is applied through a variety of methodologies such as multivariate regression, Bayesian maximal entropy and ordinary least square regression [2]. Furthermore, some water demand modelling applies time-series analysis, which use only historical records and reflect the inherent auto-correlation structure of the water use pattern over time [3] [4].

A study to investigate the performance of Artificial Neural Networks (ANNs) to establish the best model to apply in water demand has been previously carried out [5]. The multiple linear regression, time series analysis, and ANNs were used as techniques for peak daily summer water demand forecast modelling. The study used peak daily water demand data from 1993 to 2002 and meteorological variables for the summer months of May to August of each year for an area of high outdoor water usage in the city of Ottawa, Canada. Thirty-nine multiple linear regression models, nine time series models, and thirty-nine ANN models were developed, and their relative performance was compared. The study confirmed that the ANN approach is shown to provide a better prediction of peak daily summer water demand than multiple linear regression and time series analysis.

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Most models are not coupled with the real water network systems, and they do not specify where in the system water demand occurs and are thus less applicable to specific local water demand system modelling. Another study, attempted to address this by applying water use in a specific area in Kathmandu Valley in Nepal [6]. They used multiple regression analysis to select a daily water demand function and relevant explanatory variables. The results indicate that the number of connections, water pricing, public education level, and average annual rainfall are significant variables of domestic water demand.

Water demand modelling plays a key role in water infrastructure planning, design, and development, hence quantifying the effects of uncertainty on water demand estimates is critically important [7]. Theoretical and empirical methodologies for representing, modelling, and simulating complex system behavior for water demand over the last 30 years were reviewed previously [2]. The results concluded that although tangible progress has been made in improving the capabilities of water demand modelling, significant limitations still remain. Furthermore, the authors concluded that the original water demand models were fundamentally spatial, ignoring variations in water consumption over time in analysis.

This study attempted to address these issues by focussing on the variations in water demand over the time. It is important to study water usage in different countries with diverse socio-economic backgrounds, climate, and geography to get an overall picture of water usage patterns. The United Kingdom, North American and Sri Lankan data were used for the analysis. The demand data were obtained from four water companies for the duration of five to seven years for use in the study.

The focus was given to identifying the most suitable probability distribution function to model water demand data. The study is expected to contribute to finding a better fit to model peak water demand, which could apply to any water network.

2. METHODOLOGY

2.1 Data collection

2.1.1 United Kingdom (UK) Water Demand Data

To analyse the UK's water consumption patterns, daily water consumption data were obtained from two UK water utility companies to use in this research. The data were collected using data loggers at 15-minute intervals by the water companies and averaged and transferred to daily data using a software. The two companies were named as Company 1 and Company 2 for purpose of this study. Company 1 delivers water to approximately 6.7 million households and businesses in Northwest England, UK and covered an area of 6300 square kilometres. The data was received for 5 years from April 2009 to April 2013. Company 2 supplies drinking water to South-Eastern England, UK and serves 868 square kilometres covering a population of 0.7 million. The datasets were received for 7 years from January 2008 to December 2014, respectively.

2.1.2 North American Water Demand Data

Daily water demand data from pump houses at McPhillips, Maclean and Hurst were obtained from January 2009 to December 2014. The water company collected the daily data using data loggers at the water treatment plant, by the water services division. This Water Works system delivers an average of 225 million litres of water to approximately 270,000 households and businesses across approximately 297 square kilometers (114 square miles) of the developed portion of Canada.

2.1.3 Sri Lankan Water Demand Data

The data sets were collected from a treatment plant which delivers the water to several urban and semi urban towns in Sri Lanka. All towns are located about 19 km Southeast of the capital Colombo and is located on the wet zone of the country. Data were collected using data loggers at 15-minute intervals at the main distribution outlet pipe and converted to daily data using a software attached to the data loggers. Water from this treatment plant delivers water to approximately 6 million households and businesses in Southeast Colombo and covered an area of 6240 square kilometres. The data was collected from 01.01.2013 to 31.12.2018, through the main meter at the outlet pipe, where readings were taken daily.

2.1.4 Data validation - Goodness of fit test

Different researchers have considered different goodness of fit tests, in order to confirm the best fitting statistical distribution. The Anderson Darling goodness of fit parameter gives more weight to the tails than the Kolmogorov-Smirnov (K-S) test and it was selected to examine the appropriateness of the 3- parameter Log logistic distribution for water consumption data together with normal distribution (normal distribution is standard distribution used in most of the modelling studies). The Anderson Darling (AD) statistical method is a measure of how far the plot points fall from the fitted line in a probability plot. It is a modification of the K-S test and provides more accuracy for data which lie at the tails. A smaller AD value indicates that the distribution fits the data better [8] stated that as a guideline, the large sample 5% point is 2.492 and the 1% point is 3.857 could be used to assess the data.

3. RESULTS AND DISCUSSION

The main focus of this study was given to identify the most suitable probability distribution function to model water consumption data. The following sections will describe the methods applied to confirm the suitability of each distribution and the findings from the analysis.The Log normal, log logistic, 3-parameter Log logistic distributions were selected for the analysis as those probability distributions are more suitable to model positively skewed data (Table 1).

Table 1: Suitability for using different probability functions in water demand modelling

Distribution	Description	Suitability for water demand data modelling	PDF
Normal	Has the familiar symmetrical bell shape and its sample space extends from minus to plus infinity.	The water consumption data admit only positive values and since the data typically show skewed frequency patterns, the normal distribution must be approached with caution, particularly if inferences will focus on the tails of the distribution.	$f(x)=\frac{1}{\sigma\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]$ where μ is mean of the distribution and α is the standard deviation.
Log-normal	Logarithmically related to normal distribution and shows considerable flexibility of shape, which is always skewed to the right.	In log-normal distribution its sample space admits only positive values and suitable to use in analysing water consumption data.	$f(x) \quad \frac{1}{\alpha x\sqrt{2v}}\exp \frac{1}{2\alpha^2}\ln x \quad \mu^{2}$ $x,\mu,\alpha\ !\ 0$ Where μ is the mean and α is the standard deviation.

Log-logistic	The log-logistic distribution resembles the log-normal in shape, has a more tractable form. It can cope well with outliers in the upper tail	It is an uni-model, defined only for positive random variables and positively skewed which is best representing the water consumption pattern.	$f(t)=\frac{AK(At)^{K\ 1}}{[1+(At)^{K}]^{2}}$ $t \succ 0$ Where K is called a shape parameter, as K increases the density become more peaked. The parameter A is a scale parameter.
3 Log-logistic	The 3-parameter Log logistic distribution is a generalisation of 2-parameter distribution and is also known as shifted Log logistic distribution	The 3-parameter Log logistic distribution has an additional parameter called the shift parameter, which adds a location parameter to the scale and shape parameters of the (unshifted) Log logistic and adjusts the data on the tail to represent a better fit to demand data than the Log logistic distribution.	$f(x;\mu,\sigma,\xi)=\frac{\left(1+\frac{\xi(x-}{\sigma}\right.}{\sigma\left[1+\left(1+\frac{\xi(x}{}\right.\right.}$ Where μ is the location parameter, σ scale and ξ is the shape parameter
Weibull	Depending on the values of the parameters, the Weibull distribution can be used to model a variety of life behaviours and provides better distribution for life length data.	To use Weibull distribution in analysis, it is essential to have a particularly good justifiable estimate for the shape parameter to replicate the accurate distribution pattern. This distribution is not suitable to model water demand data.	$f(t)=\frac{\beta}{\eta}\left(\frac{t}{\eta}\right)^{\beta-1}\exp\left[-\left(\frac{t}{\eta}\right.\right.$ Where β is shape parameter and η is the scale parameter.

3.1 Probability Plots for UK, North American and Sri Lankan Data

There are various numerical and graphical methods that are used to estimate parameters and identify suitable probability distribution. To establish the most appropriate model in this research, a maximum likelihood method which is a graphical method used to create the probability plots. The data were fitted to normal, log logistic and 3- parameter Log logistic distributions and analysed using a 95% confidence interval (5% significance level) to establish the parameters of the distributions. The middle line in the probability plot shows the normal line and the other two lines in either side of the middle line show the 95% confidence interval. [9] stated that normal probability plots are useful in identifying distributions that fit to the normal distribution and the distributions which have tails heavier and lighter.

The normal, log logistic and 3-parameter Log logistic probability plots for Company 1 water demand data are shown in

Figure 21. The **Error! Reference source not found.** shows the North American data and Figure 3 shows Sri Lankan data. The best fit corresponds to the case where the dots fall closest to the line overall and those deviations of the data from a straight line suggest minor departures from the normal and the Log normal distributions. The 3-parameter Log logistic graphs tend to have some deviations from a straight line, similar to normal and Log normal distributions shown in the graphs, nevertheless all-data points lie within the 95% confidence limit. This suggests 3-parameter Log logistic distribution is marginally better than normal and Log logistic distributions to model the water demand data. Additionally, to the other distributions stated above, the Weibull distribution used in the North American data demonstrates its unsuitability in water demand modelling (Figure 2).

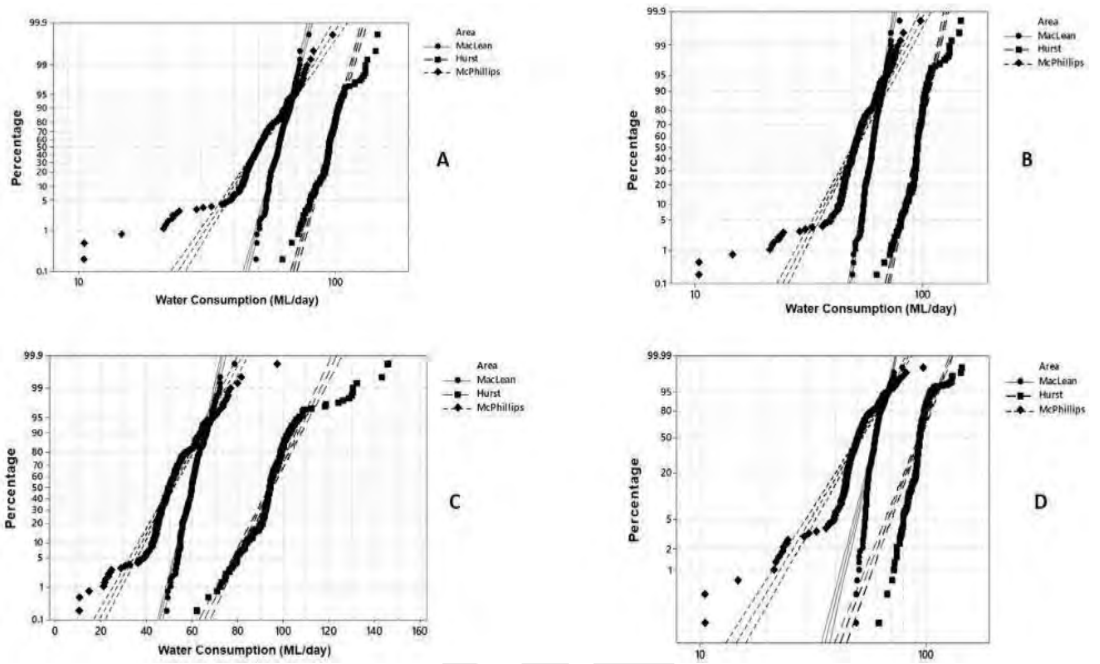

Figure 1: Comparison of A: Log logistic, C: Normal, D: Weibull - ***North American***

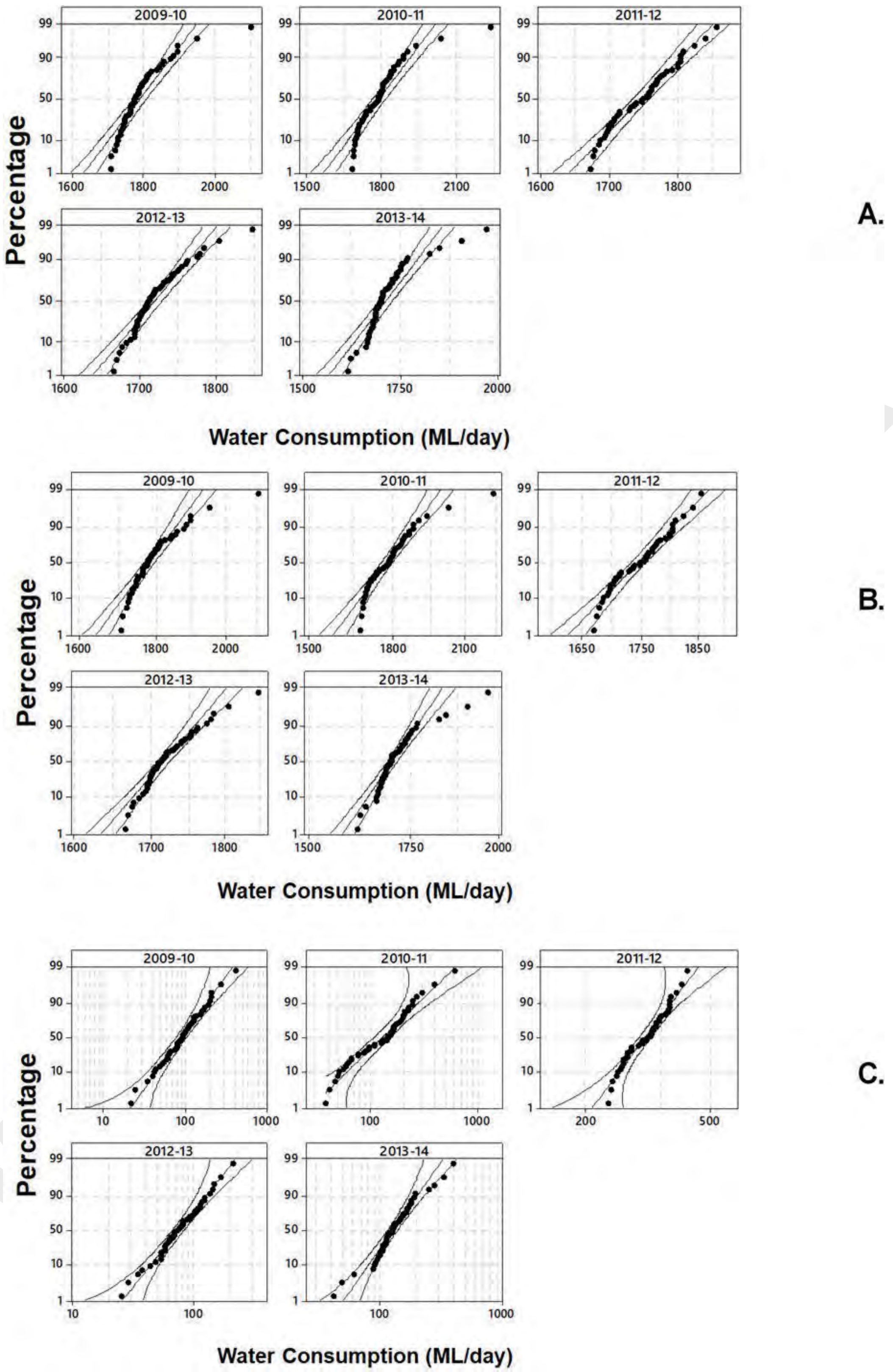

Figure 2: Comparison of A) Normal, B) Log logistic and C) 3-parameter Log logistic distribution for UK Company 1

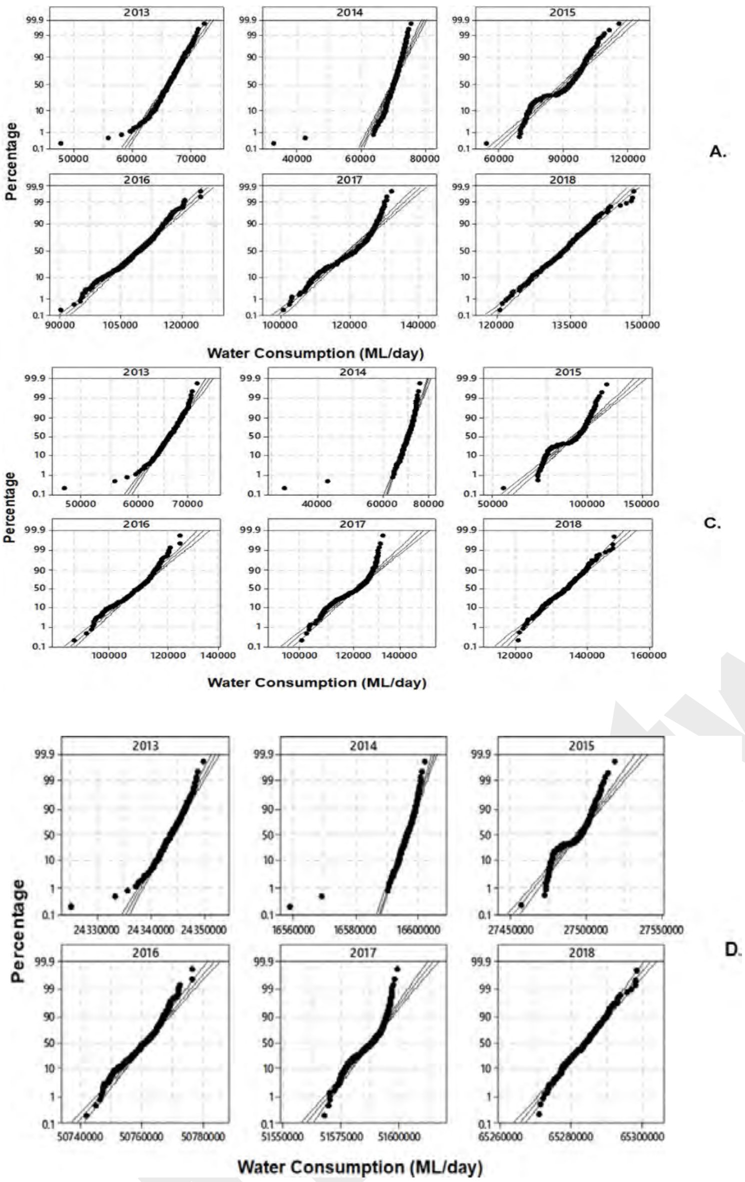

Figure 3: Comparison of A: Normal, C: Log logistic, D:3 D log logistic

3.2 The Goodness of Fitness Test

The Anderson Darling goodness of fit parameter was selected to examine the appropriateness of the 3-parameter Log logistic distribution for water consumption data. Analysis shows that 3-parameter Log logistic has the lowest AD values and it is suitable to model the water demand data. Anderson Darling (AD) values for normal, log logistic and 3-parameter Log logistic distributions for North American data shown in **Error! Reference source not found.**.

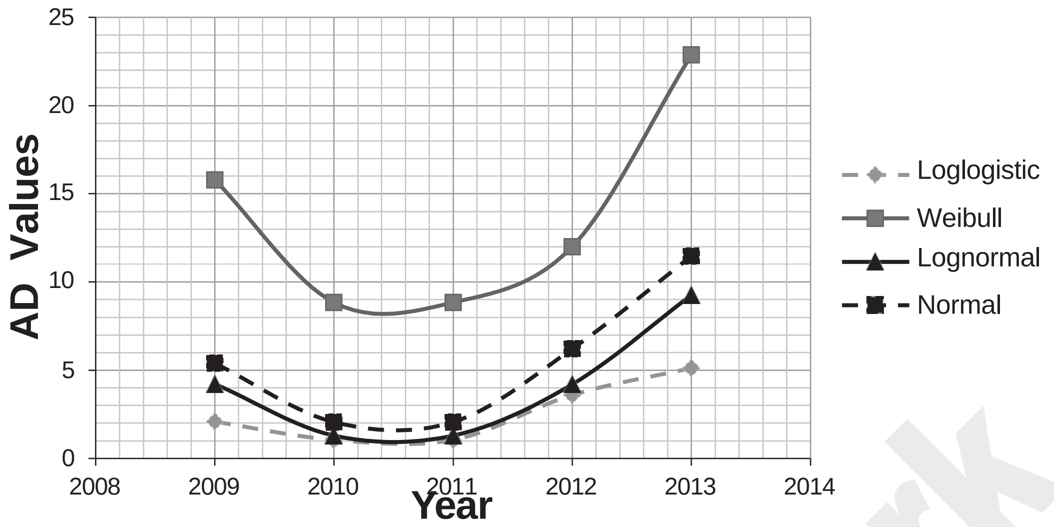

Figure 4: AD values for North American Water demand

The study shows that from the three selected distribution patterns studied, (the normal, log logistic and 3- parameter Log logistic) all distributions produced acceptable AD values and proved to be effective to model the peak demands. The 3-parameter Log logistic distribution adjusts the data in the tail to represent a better fit to data than a Log logistic distribution and it was observed that 3-parameter Log logistic provided a slightly better fit than Log logistic or normal distributions. The AD values obtained for the Weibull distribution have higher values than the other 3 distributions and is not suitable to model water demand data.

4. CONCLUSION

The findings show the distribution patterns for North America and Sri Lanka are remarkably similar to UK studies completed in 2002 [10]. Furthermore, the findings from analysing the Sri Lankan water demand data, has shown that water consumption patterns are almost similar to the UK and North America, although the lifestyles of the investigated users are completely different, the proposed models fitted very well to all datasets. It can be concluded that there may be less water per capita consumption in developing countries, however, there is no difference to the water use pattern, based on the country's economic and social status. In fact, the three distributions 3- Log logistic proved to be more effective when modelling the water demand for all the investigated time periods.

These findings would lead to improved modelling techniques to apply globally for water use estimation and to a greater capacity to link water use with its impact on future forecast. The results of this study could be considered of practical application, allowing the use of the hereby proposed probabilistic model to obtain a good estimation of the peak water demand.

The work also has the potential to provide a statistically proven method to select appropriate peaking factors to forecast the future water demand to address the extreme demand values and help policymakers make decisions with confidence.

ACKNOWLEDGEMENTS

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COMPETING INTERESTS

We declare no competing interests.

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#5. Water quality

Microbial Activity in Potable Water Storage Tanks of Barbados

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ABSTRACT

As water supplies are strained, distributed infrastructure can often help stabilize a water distribution system. Recently, the Barbados Water Authority, in collaboration with Caribbean Community Climate Change Center, installed 400, 450, and 1000-gallon potable water storage tanks to residential and school properties to increase the reliability of distributed drinking water to residents. However, there is minimal knowledge on the potential microbial impact of these storage tanks on water delivered to the tap. Preliminary data from this project confirmed that temperatures within these tanks can exceed 25 degrees Celsius, the lower threshold for increased growth of the premise plumbing pathogen Legionella. Inhalation and ingestion of Legionella pneumophila is known to cause Legionnaire's disease, a life-threatening lung disease with pneumonia-like symptoms and tends to grow with long water stagnation periods, low disinfectant residuals and elevated temperatures.
Seven sites located in the northern parishes of Barbados and one site located in a western parish were tested for temporal fluctuations of temperature, nitrate, total chlorine, total coliforms, and Escherichia coli (E. coli) with Legionella tested at select times of the day. Five of these sites were installed within the past year and three were installed up to four years prior to this study. All tanks showed values below the nitrate recommended range of less than 10 mg/L given by the USEPA. Only three tanks maintained the minimum chlorine residual of 0.2 mg/L given by the USEPA. Five sites showed positive total coliform tests and three sites showed positive E. coli tests and Legionella tests. With these results in mind, more quality assurance testing must be performed to ensure the true activity inside these tanks at various times of the year, location on the island and with continually flushed and finished systems. Unintended consequences from infrastructure upgrades are a threat, especially as climate change will continue to strain drinking water source supplies.

Keywords: Legionella pneumophila, water scarcity, potable, temperature, Barbados

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Biomass-based sorbents for stormwater treatment

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ABSTRACT

Rapid urbanisation coupled with climate change necessitates the use of bespoke facilities to treat waste and stormwater to prevent contamination of ground and surface water. The contaminants of interest are wide-ranging including heavy metals, organic pollutants and micro plastics. There is mounting interest in the management of significant amounts of biomass waste available around the world and the exploration of utilisation potential. For example, large amounts of filter media including 'active' materials such as charcoal and plant-based fibres are used. Nevertheless, the use of biomass- based sorbents and filter materials are of importance as they can offer both technical and economic advantages over traditional treatments. For example, studies are on-going on biomass-based sorbents coupled with conventional filter media such as graded sand and gravel gravity filters. The present work reviews developments in this area and introduces new potential materials for consideration, including biowaste from crustaceans, shells, and agriculturally derived biomass waste. With reference to example water supply and wastewater treatment facilities in the SE of England, the potential scale and benefits for resource recovery of biomass waste whilst also harnessing its water remediation potential will be discussed.

Renewable energy-powered reverse osmosis desalination: Solutions and opportunities for large-scale implementation

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ABSTRACT

Operating reverse osmosis (RO) systems using renewable energy (RE) is fundamental for meeting water security challenges, especially for small islands and developing states. Their integration requires RO systems to accommodate variations from RE sources to avoid reliance on backup systems. This study presents the outcomes of a research project that aimed to optimise the operation of RE powered RO by improving their variable-speed and modular operation for handling a wide range of RE variations. An industrial-scale pilot RO plant with 3.2 m^3/h production capacity was designed, commissioned and tested at Aston University, UK, to be the basis for this project. It includes an isobaric pressure exchanger and delivers similar performance to large RO systems to develop solutions suitable to such scale. Several operation strategies were investigated for operating RO systems using RE. An advanced control system using Model predictive control was developed to control the RO power consumption based on RE variation. RE availability prediction using neural networks was developed for scheduling the startup/shutdown cycles of RO units during modular operation. The project concluded that operation at variable recovery and constant brine flowrate delivered the lowest specific energy consumption and widest operation range for systems using an isobaric energy recovery device. Model predictive control enhanced energy utilization compared to a proportional-integral controller leading to a 2.35% improvement in permeate production for a defined power input. Overall, the solutions developed showed that RO systems can operate efficiently by direct RE using variable operation, which showcased the opportunities for further testing and development towards large-scale implementation.

Keywords: Reverse osmosis; renewable energy; variable operation; model predictive control; wind speed prediction.

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1 INTRODUCTION

Water consumption is increasing at twice the rate of population growth. Arid regions, especially small islands and developing states, suffer from water challenges due to the lack of resources. Renewable energy (RE) powered Reverse Osmosis (RO) has been suggested, however, the technology is only available for small-scale applications, if not dependent on grid power [1]. Variable operation is promising for operating RO directly using RE. It is structured on two techniques: a) variable-speed operation is operating RO systems at varying production capacity according to fluctuation in RE, and b) modular operation is connecting/ disconnecting RO units according to RE intermittency. Both techniques have been used in previous studies, however, due to technical challenges, they have not been used for large-scale RO applications [1, 2].

Variable-speed operation requires an operational strategy to vary the plant production capacity within a wide operation range while operating at the lowest Specific Energy Consumption (SEC) [3-7]. Another challenge is developing a control system for implementing the operation strategy. Advanced control systems were recommended for their fast response in adjusting controlled variables, despite RE fluctuations and RO system inertia [3]. As for modular operation, the RO system can be subject to multiple start-up/shutdown cycles when directly operating using RE. This is unusual for RO systems as they require adhering to certain guidelines during startup or shutdown to guarantee safe operation. This emphasises the importance of planning the start-up and shutdown of RO units to be operated safely while using RE.

This abstract summarises the outcomes of a research project between Aston University, UK, and the University of Bahrain, Bahrain, which aims to optimise the operation of RE-powered RO by developing solutions for large-scale applications. The objectives of the project are:

- Develop a RO test rig with similar design and performance to large-scale systems.
- Analyse and select an operation strategy for variable operation of RO systems.
- Design and implement an advanced control system that delivers enhanced performance with respect to RE variation.
- Enhance the modular operation of RO using operation scheduling and wind speed prediction.

2 METHODOLOGY

2.1 RO test rig

The RO test rig, shown in Fig.1, was designed to deliver similar performance to large-scale RO systems. The feed flow is split between a High-Pressure Pump (HPP) and an isobaric Energy Recovery Device (ERD). It includes two pressure vessels, each with three 8-inch RO membranes connected in series. The test rig delivers 3.2 m^3/h from seawater (35,000 mg/l) at 7.5 kW rated power consumption and 2.34 kWh/m^3 SEC. The HPP and ERD are equipped with Variable Frequency Drives (VFD) to allow for independent control of permeate and brine flowrates, respectively. A detailed description of the RO test rig can be found in [8].

Fig. 1. RO unit installed at Aston University, UK [8].

2.2 Control system design

Developing a control system to operate a RO system under varying conditions requires two elements: a) an operation window to define the safe limits of operation, and b) an operation strategy to systematically change the operating parameters according to available power. An operation window, shown in Fig. 2, was defined based on the hydraulic limitation of the membranes across the feed pressure and flowrate using the Reverse Osmosis System Analysis (ROSA) software. The operation window was used during the RO system development, system design and component selection to reflect the operational constraints. In addition, several operation strategies were investigated including a) operation at constant recovery, b) operation at constant brine flowrate, and c) operation at constant feed flow. The operating parameters for the optimum strategy were then mapped and embedded into the control system.

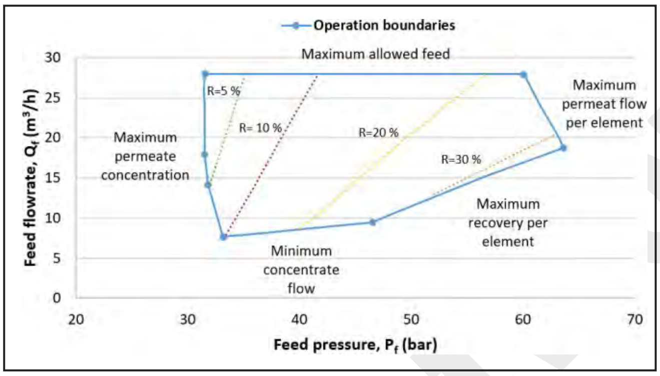

Fig. 2. Safe operational window [8].

A fast and robust controller is crucial when implementing the variable-speed operation technique. An advanced Model Predictive Controller (MPC) was designed to control the RO system according to RE variation and compared to a conventional Proportional-Integral-Differential (PID) controller. As shown in Fig. 3, the MPC includes a prediction model to assess the plant response to a generated control signal and an optimiser that modifies the control signal according to the predicted plant output.

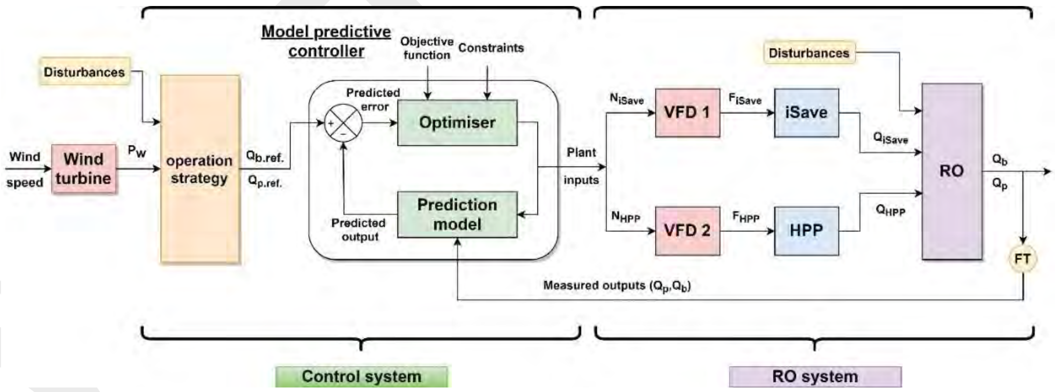

Fig. 3. Structure of the model predictive controller [8].

2.3 Modular operation and scheduling

Operation scheduling for the modular operation of RO systems can provide a stable operation similar to using grid power by reducing the repeated start-up/shutdown cycles. A case study was assumed that includes three RO units connected to the same wind turbine to simulate the modularity of large-scale RO systems and investigate the operation improvement when using scheduling compared to unscheduled operation. A modular operation strategy was developed, and a wind speed prediction Neural Network (NN) was designed to predict wind speeds 24 hours ahead. The predicted time range allows for scheduling the daily operation of the RO units.

2.3.1 Modular operation

The modular operation technique was developed to execute the RO units connection/disconnection according to power availability. It defines the number of operating units, their power allocation, and order of operation. It also includes a Standard Operating Procedure (SOP) for each unit, which is based on the guidelines of the membranes and pumps manufacturers.

2.3.2 Wind speed prediction

A feed-forward backpropagation NN was developed for predicting wind speeds. As presented in Fig. 4, the network consists of three layers: an input layer, a hidden layer, and an output layer. The layers are connected by weighted connections, in which the neurons receive the inputs from the previous layer and produce an output based on an activation function. The input and output layers include 24 neurons, which is equivalent to the network inputs and outputs. The Levenberg-Marquardt backpropagation algorithm was used as a training algorithm to modify the network weights [9, 10].

The wind speed data were collected at the University of Bahrain with one-minute resolution for a period spanning six months. The wind data were converted to hourly averages, to match the selected prediction range, before network training or testing.

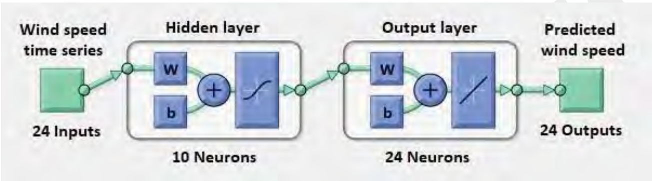

Fig. 4. Structure of the neural network [11].

3 RESULTS AND DISCUSSION

3.1 Selection of an optimum operation strategy

Three operation strategies for variable-speed operation were investigated including operation at constant recovery (15, 20, and 24%), constant brine flowrate or constant feed flowrate. For operation at constant brine flowrate, the permeate flow was controlled using the VFD for the HPP, while mainting the brine at the minimum flowrate of 9.8 m^3/h to allow for pressure higher than 1 bar at the iSave brine discharge [12]. As for operation at constant feed flow, the brine and permeate flowrates were varied to maintain a constant feed flow at 13.3 m^3/h, which is the sum of the minimum brine flowrate and maximum permeate flowrate.

Fig. 5. shows that operation at variable recovery delivers a wider operation range than operation at constant recovery. It also shows that variable recovery guarantees lower SEC compared to constant recovery due to lower power consumption by the ERD. Maintaining a constant brine flowrate achieved lower SEC consumption compared to maintaining a constant feed flow due to minimised brine flowrate and operation at higher recovery ratios. This concluded that operation with variable permeate recovery and constant brine flowrate was the optimum operational strategy for systems with split feed flow configuration and an isobaric ERD.

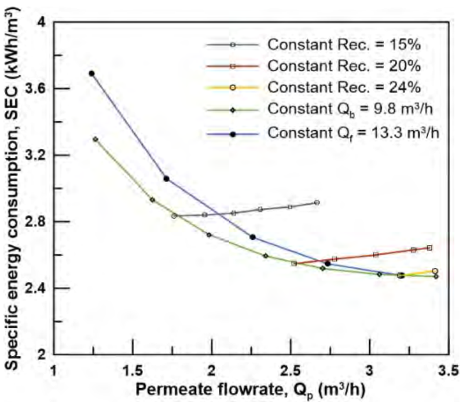

Fig. 5. Comparison of potential strategies for variable-speed operation [8].

3.2 Control system performance analysis

The PID and MPC controllers were compared by analysing their ability to maintain the permeate flowrate at the reference value and comparing their permeate production for the same wind power. Three wind speed signals were used that are based on a random signal with Gaussian distribution about a mean wind speed of 6.5 m/s with a standard deviation of 5, 10 and 15% of the mean wind speed. The sample time was set to 10s and a high variation was introduced for feed concentration and temperature with a 2% standard deviation around a mean of 35,000 mg/l and 25^{o}C, respectively.

Each controller was tested for one hour of operation with same wind speed signal and feed disturbances. A sample (15 minutes) of the performance for wind speed signal with 10% standard deviation is shown in Figs. 6 and 7 for the PID and MPC controllers, respectively. Fig. 6 shows that the PID controller caused an evident mismatch (delay or overshoot) between the actual and reference permeate flowrate in cases of high-amplitude wind speed variation. As for the MPC, shown in Fig. 7, it provided significant improvement by matching the actual permeate flowrate to the reference signal. This translated to minimal overshoot and less delay compared to the PID. This improvement is due to the MPC's ability to predict the plant response to the control signal and optimise the control sequence before execution.

In general, the MPC controller achieved higher permeate production per hour compared to the PID during testing, with varying quantities according to the wind speed fluctuations. The improvements in permeate production per hour reached 0.31%, 1.76% and 2.35% for the three wind speed time series.

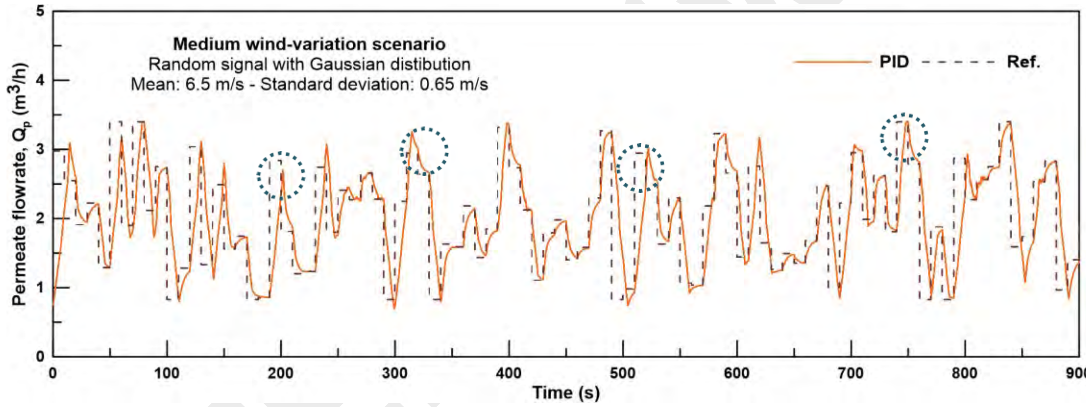

Fig. 6. Performance of the PID controller for tracking a reference signal for a defined wind-speed variation [8].

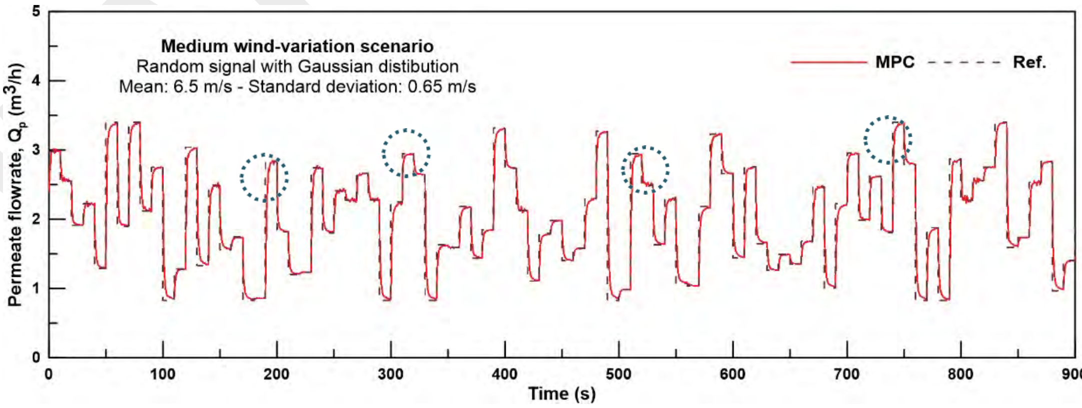

Fig. 7. Performance of the MPC controller for tracking a reference signal for a defined wind-speed variation [8].

3.3 Wind speed prediction and operation scheduling

Wind speed prediction was simulated for the 6-month dataset and the correlation between the target and predicted values was analysed using linear regression, in which the NN outputs reached a correlation of (R^2 = 0.59) with the target data, which showed that wind speed prediction has the potential for use for RO units scheduling. The wind speed accuracy would vary based on the wind variation intensity during the day. In the example shown in Fig. 8, the NN predicted wind speed with an accuracy of 1.54 m/s and R^2 of 0.76.

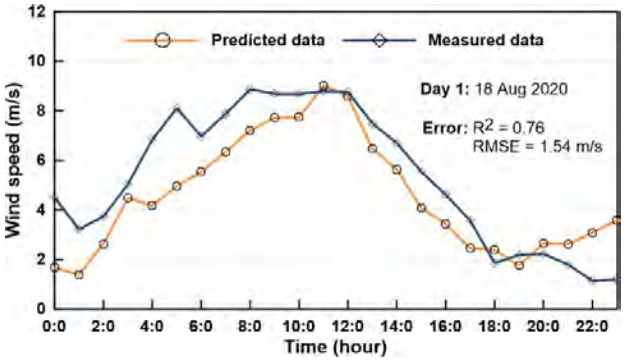

Fig. 8. Predicted hourly average wind speeds on 18 August 2020 compared to measured data [11].

For scheduling the RO units' operation, the number of possible operating RO units was estimated based on the predicted wind speed. Then, the RO units operation was scheduled according to two scheduling approachs; a high-production approach that maximises permeate production by assuming that there are no prediction errors, and a low-production approach that minimises unexpected shutdowns by reducing the number of operational units by one compared to the predicted possible, as shown in Fig. 9.

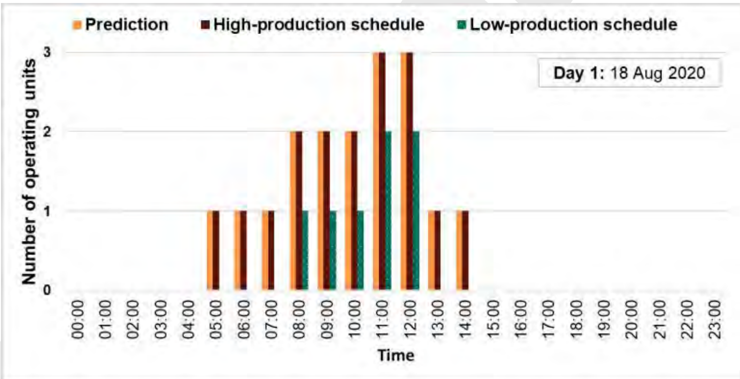

Fig. 9. Schedule of the RO units' operation based on the high-and low-production scheduling techniques [11].

4 CONCLUSION

Operating large-scale RO systems directly using RE without energy storage or grid power is essential for decarbonising water production and allowing access to RO for small islands and developing states. This project has aimed to provide solutions based on advanced control and machine learning techniques to improve the efficiency of variable-speed and modular operation. The following are the key findings of the project:

- Operation at variable recovery and constant brine flowrate is optimum for the RE-powered large-scale RO systems that include an isobaric ERD.
- MPC provides superior control and higher permeate production compared to PID for the variable operation of RO systems.
- Wind speed prediction using NN for 24 hours ahead is applicable for estimating the possible number of operating units and scheduling the RO system operation.
- With further investigation, the combination of variable-speed operation, modular operation and wind speed prediction can lead to a breakthrough for RE-powered RO.

ACKNOWLEDGEMENTS

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COMPETING INTERESTS

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Comparative Study of Solar-Enhanced Advanced Oxidation Processes for Water Treatment

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ABSTRACT

Water is an essential resource for human survival but in the 21st Century there is a lack of potable water in many parts of the world. In several developing countries people resort to consuming heavily polluted water obtained from rivers which contain life threatening diseases. The study used two methods, Photo Fenton and photocatalytic semiconductor as an advanced oxidation process to eliminate various water contaminants and provide an effective water treatment solution. Experiments were performed on polluted river water. Both methods were assessed by evaluating the physiochemical parameters that define the characteristics of safe water. The two methods successfully eradicated about 80-100% of pollutants that was measured in the river water samples. This shows the technology has potential in eradication of the contaminants.

Keywords: Titanium Dioxide, Solar Photo Fenton, Solar photocatalytic semiconductor, Photocatalysis (Arial, inclined, 10 font, justified)

1. INTRODUCTION

The world has advanced hugely with new technology, nevertheless there are still countries in today's society where the need for water is massive. Water is a fundamental for existence, from industry to agriculture, however nature also needs water to thrive. So it is essential that the water provided is free of any contaminants. In the history of mankind, a clean water supply has been an effective way to eliminate many types of diseases. In many countries of the world, clean water sources are becoming scant while simultaneously facing a significant rise in demand due to reasons such as industrial and agricultural development, population growth, increase human life expectancy, and long term drought caused by global warming. An increased capacity for rainwater harvesting is one example of a short term solution being used at the moment to deal with this matter.

1.1 Solar Photo-catalyst properties

There are many types of AOPs but the best one so far is photocatalytic oxidation (PCO). This involves the destruction of organic compounds by the shining of ultraviolet light or sun light onto a catalyst such as Titanium Dioxide (TiO_2), Zinc Oxide (ZnO), Iron Oxide (FeO_3), while at the same time avoiding the introduction of other chemicals. solar light and/or UV lamps are used to excite the catalyst thereby destroying the fungus and bacteria into non-harmful products [1]. Titanium Dioxide (TiO_2) is by far the best for water treatment at present. TiO_2 is stable, affordable, environmentally-friendly, sustainable and nontoxic. Also, no additives are needed assists the photo degradation of a wide variety of substances that pollute water at ambient temperatures [1].

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TiO2 has been the most promising catalyst due to the following reasons:

- TiO_2 catalyst does not output any toxic products unlike other semiconductor catalysts.
- TiO_2 operates at ambient pressure and temperature.
- The mineralisation of parents is complete leaving no secondary pollution.
- Low operation cost.

Disadvantages of TiO_2 are the post recovery particles or separate ability of particles, and low capacity of adsorption [1].

1.2 TiO_2 Water Disinfection processes

Being a composite semiconductor, Titanium dioxide (TiO_2) is automatically a strong oxidant when exposed to any form of light, like ultraviolet radiation or the sun. Due to TiO_2 peak oxidation properties, it is ideal for the degeneration of a majority of natural or man-made compounds even in low concentrations [2]. Through this interaction, various organic substances that approach the titanium dioxide are decomposed continuously until they become harmless. The influence of photo catalysis is almost permanent.

In semiconducting composites or compounds, a large number of electrons are located at the bound region. An energy level of not less than 3.2 electron volt (eV) is required to excite these electrons. A quantum of light (hv) with its wavelength less than or equal to 390 nm can supply the energy (3.2 eV). Exposure to UV light and sunlight causes particles of TiO_2 catalyst to produce a hole (h+) and charged coupled electrons that are separated from the ion. These are known as free electrons (e-).

Once exposed to ultraviolet light (UV), TiO_2 valence band (vb) electrons (e-) get excited and make their way to the conduction band (cb) thereby abandoning the hole (h+) which has a positive charge [3].
The excitement of valence band (vb) electrons (e-) under UV light causing their movement from the hole (h+) to the conduction band (cb) is termed photo-excitation and can be expressed as this reaction

TiO_2 + hv >>> TiO_2 + e- + h+ (1)

Coupled e- and h+ from the process can join again to reseal the energy soaked in as heat. Holes (h+) get into reactions with water (H_2O) forming hydroxyl radicals (OH) and simultaneously natural compounds are oxidised. Electrons also react with oxygen (O_2) to generate superoxide radicals (O_2) and carry out oxidation and the breakdown of organic matter [3].

The reactions that form TiO_2 superoxide and hydroxyl radicals can be expressed as:
TiO_2 (h+vb) + H_2O >>> OH + H+ (Reaction 1)
TiO_2 (h+vb) + OH- >>> OH (Reaction 2)
TiO_2 (e-cb) + O_2 >>> O_2- (Reaction 3)
Where: vb is the valence band and cb is the conduction band.
Other superoxide and hydroxyl radicals are created, react with contaminates and also undergo oxidation by breaking down into water (H_2O) and Carbon dioxide (CO_2) [4]. The TiO_2 photo catalytic process is a productive technique for eliminating bacteria and breaking down organic contaminates due to the strong oxidising agents. The addition of TiO_2 to wastewater decreases microorganisms much better and faster since its total surface area for the reaction is larger. The only drawback to this process is the necessity of removing more TiO_2 particles from the water. This can be time consuming and complex, but this could be done by filtration [5].

1.3 Solar Photo Fenton properties

1.3.1 Levels of Fe^{2+}

Previous studies examined the optimal levels of Iron needed for the SPF reaction to be more effective. The studies tested the levels of iron in industrial wastewater, which contained pesticides. The tests were implemented with various doses of the compound $7H_2O.FeSO_4$. The results of these tests revealed that raising the amount of Fe^{2+} to 2.4 mg/L was the optimum dosage with a 10:1 suitability of Hydrogen Peroxide [6].

1.3.2 Hydrogen Peroxide and the Fenton Process

Degradation activity when adding Hydrogen Peroxide from 30 mg to 180 mg to pollutant samples were examined. The removal activity was most significant a ratio of 10:1 where a dosage of 120 mg dosage removed most pollutants in 90 minutes. The reaction found that no additional H_2O was necessary as an excessive quantity of water will reduce the activity of the reaction. The benefits of this research showed that using Hydrogen Peroxide in the reaction can minimise the quantity of water required for the reaction, which is highly useful when the aim is to be more economical [6].

However, there are many conditions where Hydrogen Peroxide hasn't been practical under a UV situation. This is because water has a very low molar extinction coefficient and increasing the depth of the water quickly decreases the activity of the UV light energy. The effectiveness of degradation is generally dependent on the level of pollutants in the water. If the concentration levels of the pollutants are small, then under a UV situation the hydrogen peroxide will be considered effective [7].

1.3.3 Time

The effect of stirring the sample in comparison with using solar energy to degrade the pollutants was also examined. The results indicated that when stirring 85% of COD was extracted within half an hour and 91%of COD removal rate was reached by solar energy, however, the duration was 4 times longer. The Chloropyrifos removal was also more effective in solar light in comparison to stirring, with removal rates reaching 83% after 180 minutes of solar exposure compared to the 60% was achieved after 1hour of stirring [8].

1.3.4 Improving the SPFP reaction

The effects of adding particular compounds indicated that adding Titanium Dioxide to the sample made the degradation rate of pollutants better by acting as a catalyst. The use of TiO_2 improved degradation of Chloropyrifos in the sample water by 27%, which is a 10% increase from the standard SPF reaction. When experimenting with the levels of this catalyst, it was found that 1.5-2 mg/l was the optimum level of Titanium Dioxide required [9].

1.3.5 Cost Effective

Due to the technology being used in developing countries where significant expenditure into water treatment process is not an option, this reaction needs to be extremely cost effective. Studies carried out on cost evaluation of the entire reaction and ways to cut the expenditure were extremely promising as a reduction of 83.33% in reagent costs was achieved [10].

Due to the level of pollution in the water, relying on Advanced Oxidations Processes (AOPs) alone to treat the water will be quite expensive. In order to decrease cost, the SPF reaction will be the initial stage of the degradation process before using cheaper biological treatments to remove the remaining pollutants. Therefore, in the preliminary oxidative procedure the non-biodegradable and toxic compounds within the water will be removed [10].

2. MATERIAL AND METHODS

2.1 Experiment Procedure

- River water samples was collected twice weekly from the Couva River, Trinidad, West Indies.
- Perform Photo-Fenton reaction with solar light at various flowrates within the solar reactor (Figure 4).
- Measure the levels of iron and sulphates and observe the difference in results.
- Perform photocatalytic reaction under solar light with TiO_2.
- Measure the rate of removal of contaminants found in surface water system and observe which is the most effective method.
- Implement improvement to increase efficiency.

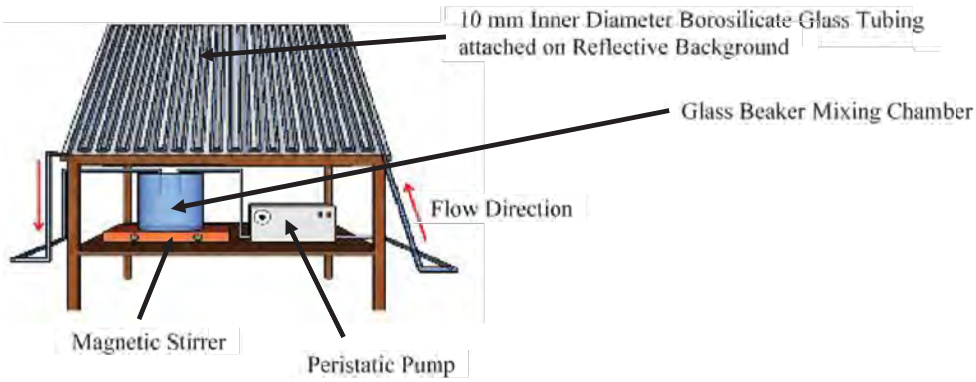


Figure 1: Experimental setup for Solar Photochemical Reactor for Water and Wastewater Treatment

2.2 Water Collecting & Sampling

The study involved using sample water from the Couva River, Trinidad, West Indies. The use of technology was a very important issue and the major pros of using this technology is that the collected water can be compared with a wider timeline range. Therefore, any unexpected readings from any potential external factors can be acknowledged and eliminated. External factors such as chemicals which may have been dumped upstream could influence the water properties at any given time.

2.3 Water Parameters

After carrying out the SPF and Photocatalytic reaction specific parameters were calculated to show how effective the reaction has on removing the pollutants and to assess the integrity of the sample against standards established by the authorities. These parameters measure different elements of the sample to see if there were problems in the sample compared to the standardised water.

2.3.1 Phosphates

Phosphate is a crucial element in regard to the growth of living things such as plants and animals, and is essential for nutrition. However, it is important that this physiochemical parameter is kept in moderation since a high concentration of phosphate can lead to an increase in mineral and organic nutrients, which in turn leads to a reduction in the amount of dissolved oxygen in the water.

2.3.2 Turbidity

Turbidity is the measure of light-transmitting properties from a sample of water. These experiments measured the quality of the water discharged with respect to the residual and colloidal suspension issue. The study of the intensity of light that is scattered on the water sample will show the level of colloidal issue within the sample as it will absorb the light that is shone upon the sample and so when there is a clear water sample under the same conditions the difference becomes visible.

2.3.3 Colour

Colour may be used to determine the age of the sample water that is collected from the two water sources. By evaluating the odour and the colour of the water sample, specific characteristics can be found with regard to the age of the water. When a sample of wastewater was fresh it displayed a light grey shade [11].

2.3.4 Temperature

Temperature is an important factor when taking hygiene into account. In terms of biochemical reactions, a rise in temperature of 10^0C can approximately double the reaction rate of bacteria with the uptake of oxygen, while cooling to low temperatures will destroy living bacteria as without heat the bacteria cannot survive. A key factor of temperature in water is specific constituents such as the concentration of dissolved oxygen can be changed at different temperatures due to the ionisation of ammonia elements [12].

2.3.5 Ammonia NH_3

There are usually small quantities of ammonia in natural water but excessive amounts are incredibly harmful to aquatic life in fresh water.

2.3.6 Biochemical Oxygen Demand

Biochemical Oxygen Demand or Biological Oxygen Demand (BOD) is a parameter that is routinely used in all forms of containments for surface and wastewater. This parameter measures the amount of dissolved oxygen that is being consumed by the microorganisms within the wastewater sample. This occurs when the organic matter within the sample has undergone biochemical oxidation [13].

2.3.7 Chemical Oxygen Demand

The Chemical Oxygen Demand (COD) represents the amount of organic issue within a pollutant sample which is very sensitive to the oxidation process when undergoing SPFP. The definition of COD is the 'total quantity of oxygen required to chemically oxidize the non- biodegradable and bio degradable organic matter'. The organic matter within the sample will be eliminated and transformed into water and carbon dioxide [14].

2.3.8 pH

The pH is classified from the negative logarithm of the hydrogen ion concentration that is used to determine whether the liquid is acid or alkaline [15].

2.4 EXPERIMENT

2.4.1 River Water Collection

Water was collected from the Couva River, Trinidad, West Indies. Initially a risk assessment was carried out to consider the potential risks and ensure the safety of all the members collecting the river water.

2.4.2 Solar Photo Fenton experiment

Test 1

Sunlight test: 10:1 ratio (Flat); River Water: 12L; Temperature: 22-24 degrees;
Hydrogen Peroxide (30%): 24 ml; Iron: 2.4 g (Table 1)

Test 2

Sunlight test: 10:1 ratio (raised); River Water: 12L; Temperature: 22-24 degrees
Hydrogen Peroxide (30%): 24 ml; Iron: 2.4 g (Table 2)

Test 3

Sunlight test: 5:1 ratio; River Water: 12L; Temperature: 19-20 degrees
Hydrogen Peroxide (30%): 24 ml; Iron: 2.4 g (Table 3)

Test 4

Sunlight test: 20:1 ratio; River Water: 12L; Temperature: 17-20 degrees
Hydrogen Peroxide (30%): 24 ml; Iron: 2.4 g (Table 4)

Table 1: Physiochemical Parameter Readings at different stages of the 10:1 experiment under sunlight conditions - Flat

Minutes	0	30	60	90	120
Colour	290	45	14	6	5
BOD	56.00				4.81
Turbidity	29.25	14.61	7.52	4.73	2.64
pH	6.1	6.2	6.6	6.9	6.9
Phosphates	0.474	0.265	0.232	0.176	0.079
COD	51.31	25.35	18.69	8.51	4.74
Ammonium	9.21	7.55	5.63	4.41	3.64

Table 2: Physiochemical Parameter Readings at different stages of the 10:1 experiment under sunlight conditions - Raised

Minutes	0	30	60	90	120
Colour	290	38	12	6	3
BOD	56.00				4.55
Turbidity	29.25	12.61	6.54	7.10	7.10
pH	6.1	6.3	6.7	7.1	7.1
Phosphates	0.474	0.233	0.212	0.176	0.039
COD	51.31	25.35	18.69	8.51	4.74
Ammonium	9.21	7.15	5.32	4.11	3.61

Table 3: Physiochemical Parameter Readings at different stages of the 5:1 experiment under sunlight conditions

Minutes	0	30	60	90	120
Colour	290	51	27	17	12
BOD	48.00				5.95
Turbidity	28.35	15.21	8.54	6.43	3.44
pH	6.1	6.4	6.7	7.2	7.2
Phosphates	0.474	0.213	0.192	0.186	0.139
COD	51.31	26.32	19.69	9.51	5.74
Ammonium	9.21	6.95	5.12	3.71	3.21

Table 4: Physiochemical Parameter Readings at different stages of the 20:1 experiment under sunlight conditions

Minutes	0	30	60	90	120
Colour	317	61	47	19	16
BOD	46.00				5.15
Turbidity	28.35	8.27	4.24	1.43	0.42
pH	6.3	6.4	6.7	6.8	6.8
Phosphates	0.274	0.213	0.182	0.086	0.049
COD	50.31	38.32	22.69	18.51	11.77
Ammonium	8.21	6.95	5.62	3.91	3.41

Test 5
Sunlight test: 10:1 ratio; River Water: 12L; Hydrogen Peroxide (6%): 24 ml; Iron: 2.4 g (Table 5)

Test 6
Sunlight test: 5:1 ratio; River Water: 12L; Hydrogen Peroxide (6%): 12 ml; Iron: 2.4 g (Table 6)

Table 5: The results from the 10:1 experiment under sunlight conditions

Minutes	0	30	60	90	120
Colour	314	32	13	8	2
BOD	45.00				6.35
Turbidity	27.35	13.27	8.21	4.43	2.42
pH	6.2	6.2	6.3	6.3	6.4
Phosphates	0.574	0.213	0.180	0.146	0.089
COD	50.71	35.32	24.69	17.51	12.77
Ammonium	8.31	7.95	5.82	4.91	3.11

Table 6: The results from the 5:1 experiment under sunlight conditions

Minutes	0	30	60	90	120
Colour	324	82	53	38	28
BOD	42.00				8.32
Turbidity	26.35	19.27	14.21	10.43	8.42
pH	6	6.1	6.2	6.3	6.4
Phosphates	0.474	,363	0.318	0.246	0.219
COD	48.71	37.31	26.69	22.51	19.77
Ammonium	9.31	8.95	7.22	6.91	5.11

Test 7
Sunlight test: 20:1 ratio; River Water: 12L; Hydrogen Peroxide (6%): 48 ml; Iron: 2.4 g (Table 7)

Test 8
Sunlight test: 2:1 ratio (Iron: Alum); River Water: 12L; Hydrogen Peroxide (6%): 48 ml Iron: 2.4 g (Table 8)

Table 7: The results from the 20:1 experiment under sunlight conditions.

Minutes	0	30	60	90	120
Colour	364	80	58	35	26
BOD	44.00				9.32
Turbidity	27.35	19.57	14.41	10.93	6.42
pH	6.1	6.2	6.2	6.3	6.4
Phosphates	0.454	0.373	0.308	0.226	0.149
COD	46.71	36.31	27.69	20.51	11.77
Ammonium	8.31	7.95	6.22	4.91	4.11

Table 8: Results from the 2:1 ratio of Aluminium Sulphate under sunlight condition

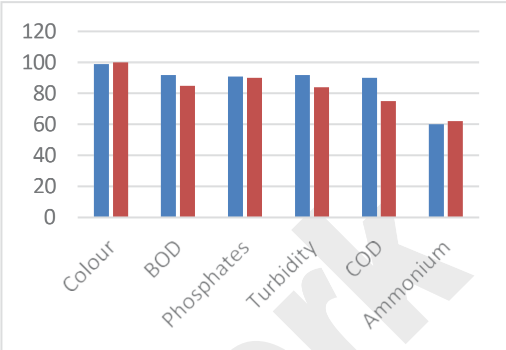
Minutes	0	30	60	90	120
Colour	343	24	15	7	4
BOD	42.00				2.52
Turbidity	28.35	12.27	7.29	2.41	0.72
pH	6.3	6.4	6.7	7.0	7.0
Phosphates	0.474	0.263	0.118	0.046	0.059
COD	52.71	16.31	9.69	2.51	0.47
Ammonium	9.31	8.93	7.25	5.91	5.16

Test 9
Sunlight test: 5:1 ratio (Iron: Alum); River Water: 12L; Aluminium Sulphate: 12 g
Hydrogen Peroxide (6%): 48 ml; Iron: 2.4 g (Table 9)

Table 9: Results from the 5:1 ratio of Aluminium Sulphate under sunlight condition

Minutes	0	30	60	90	120
Colour	343	25	15	10	8
BOD	44.00				3.52
Turbidity	28.35	15.27	13.29	11.41	8.72
pH	6.3	6.4	6.7	6.9	7.0
Phosphates	0.474	0.163	0.108	0.086	0.057
COD	52.71	17.31	9.69	2.91	0.87
Ammonium	9.31	8.92	7.24	5.71	5.06

Figure 2. Percentage Removal: 10:1 Ratio 6% sunlight and 30% sunlight.



2.4.3 Solar photocatalytic semiconductor experiment

A 1.5 m long tubular translucent photo-reactor was used to run the test using solar energy. The setup was placed on a trolley and wheeled outside and located in the direction of the sunlight.

Test 1
Sunlight test; River Water: 10 L; Temperature 18-20; 0.2 g/L TiO_2 (Table 10)

Test 2
Sunlight test; River Water: 10 L; Temperature 18-19; 0.5 g/L TiO_2 (Table 11)

Test 3
Sunlight test; River Water: 10 L; Temperature 18-20; 1.0 g/L TiO_2 (Table12)

Test 4
Sunlight test ;River Water: 10 L; Temperature 18-20; 2.0 g/L TiO_2 (Table 13)

Table 10: Results from Sunlight + 0. 2 g/L TiO_2 test

Minutes	0	60	120	180	240
Colour	500	197	155	45	35
BOD	42.00				1.52
Turbidity	61.35	36.27	28.29	23.95	17.42
pH	7.2	7.4	7.5	7.6	7.7
Phosphates	0.413	0.165	0.117	0.086	0.050
COD	82.41	14.31	5.69	5.51	4.47
Nitrite (NO_3)	0.870	0.116	0.125	0.090	0.070
TDS (ppm)	935	881	875	868	860

Table 11: Results from Sunlight + 0. 5 g/L TiO_2 test

Minutes	0	60	120	180	240
Colour	500	135	32	25	15
BOD	43.00				1.63
Turbidity	61.35	18.27	16.29	14.95	11.22
pH	7.2	7.5	7.8	7.9	7.9
Phosphates	0.413	0.164	0.116	0.085	0.050
COD	82.41	12.31	11.69	9.51	6.47
Nitrite (NO_3)	0.870	0.760	0.620	0.590	0.470
TDS (ppm)	935	929	910	901	814

Table 12: Results from Sunlight + 1 g/L TiO_2 test

Minutes	0	60	120	180	240
Colour	500	85	18	16	7
BOD	43.00				1.68
Turbidity	61.35	16.28	15.25	11.95	7.50
pH	7.2	7.6	7.8	7.9	7.9
Phosphates	0.413	0.360	0.310	0.280	0.260
COD	82.41	16.38	14.69	12.51	10.17
Nitrite (NO_3)	0.870	0.720	0.650	0.580	0.360
TDS (ppm)	935	852	848	785	630

Table 13: Results from Sunlight + 2 g/L TiO_2 test

Minutes	0	60	120	180	240
Colour	500	16	12	10	8
BOD	43.00				2.23
Turbidity	61.35	13.68	11.25	8.75	6.50
pH	7.2	7.6	7.7	7.9	7.9
Phosphates	0.413	0.380	0.350	0.310	0.290
COD	82.41	16.23	14.29	10.41	9.13
Nitrite (NO_3)	0.870	0.520	0.450	0.380	0.250
TDS (ppm)	935	710	667	645	607

3. RESULTS AND DISCUSSION

3.1 SOLAR PHOTO FENTON

Based on published research, 30% concentration of hydrogen peroxide would be capable of eliminating the pollutants found within the sample even in the climate experienced. However, the source of light had a major effect on the efficiency of the reaction. When testing, it was apparent that the best ratio for removing the degradants was 10:1 as previous studies had suggested. The 30% concentration of hydrogen peroxide was able to remove the majority of pollutants when used under sun light conditions. However, when repeating the same experiment with a 6% concentration of hydrogen peroxide, it proved less effective with significant concentrations of physiochemical parameters still remaining within the sample. Taking into account the cost difference, it was found that, the optimum concentration of hydrogen peroxide was more than twice the price of the 6% concentration. This therefore makes the technology expensive. From the summary of results shown on Table 14 below, that the removal of degradants between the 30% and the 6% with aluminium sulphate is similar on specific physiochemical parameters. The difference that the coagulant made when analysing the peak results between the original 6% and the 6% with aluminium sulphate is very large.

When raising the concentration ratio between Iron Sulphate and Aluminium Sulphate, it seemed less active in comparison to the 2:1 concentration. The reduction of the pollutants was significantly decreased when increasing the dosage to 5:1 and 10:1.

When experimenting with external conditions, the results of coagulation supplementation were very promising with high removal activity despite the varying exposure to sunlight experienced. Another element that needs to be taken into account is the settlement period of the entire experiment. This is due to iron being a product of the reaction. The iron with the contaminants attached must be separated from the sample itself.

Table 14: The best results of all chemical concentrations

Physiochemical Parameters	Time (Minutes)				
	0	**30**	**60**	**90**	**120**
Ammonium 6%	8.31	7.95	5.82	4.91	3.11
pH	6.2	6.2	6.3	6.3	6.4
COD 6%	50.71	35.32	24.69	17.51	12.77
Phosphates 6%	0.574	0.213	0.180	0.146	0.089
Turbidity 6%	27.35	13.27	8.21	4.43	2.42
Colour	314	32	13	8	2
Ammonium 30%	9.21	7.15	5.32	4.11	3.61
pH	6.1	6.3	6.7	7.1	7.1
COD 30%	51.31	25.35	18.69	8.51	4.74
Phosphates 30%	0.474	0.233	0.212	0.176	0.039
Turbidity 6%	29.25	12.61	6.54	4.43	2.24
Colour	290	38	12	6	3
Ammonium 6% with Al	9.31	8.93	7.25	5.91	5.16
pH	6.3	6.4	6.7	7.0	7.0
COD 6% with Al	52.71	16.31	9.69	2.51	0.47
Phosphates 6% with Al	0.474	0.263	0.118	0.046	0.059
Turbidity 6% with Al	28.35	12.27	7.29	2.41	0.72
Colour	343	24	15	7	4

3.2 Solar photocatalytic

3.2.1 Effect of initial concentration

Concentration Photo degradation of the river water was implemented at 0.2, 0.5, 1 and 2 g/L. The results showed that the rate of deterioration increased with increased concentration until it reached the plateau stage. This means the TiO_2 amount was proportional to the reaction and degradation rate until it reached the stage where the degradation rate became constant and subsequently began to decrease slightly. As the TiO_2 amount increased up to saturation point (thereby causing a high turbidity situation), the absorption of light (photons) coefficient normally decreased. Adding too much titanium dioxide lead to excess titanium dioxide particles that tend to create shade in the reactors thereby reducing the surface area of exposure to the photocatalyst.

To solve this issue, photo reactors used should be run beneath the photo catalyst's saturation point and the TiO_2 should be mixed with a sample of wastewater that must be treated to create a solution before adding it to the reactors.

3.2.2 Effects of light intensities

The rate of degradation was also directly proportional to intensity of light. Higher light intensities resulted in higher degradation rates, however, after a certain point, higher light intensities caused negligible difference in the degradation rate. This was because as light levels rose, so did the generation of electron holes. More electron holes were formed as light intensity increased. However, this became more gradual and stable with very little variation in decomposition rate as the light intensity reached its peak. Thus electron holes recombined and clashed with the separation of electron holes thereby causing a reduction in the result of light intensity and the rate of degradation [16].

4. CONCLUSION

The study of SPF and photocatalytic semiconductors proved to be effective by achieving the targets that were originally set. This improved and made more affordable the process of restoring the river water in developing countries.
By implementing this technology in countries with higher levels of incident solar radiation the outcome will be either improved, or somewhat similar but achieved in less time.
The Solar photo Fenton experiments managed to remove within 80-100% of pollutants measured from the river samples.
Using solar photocatalytic semiconductors have the advantages of being a compact process, very low doses of chemicals, fewer residues, and faster reaction time. Disadvantages include operational cost, hydroxyl radicals, reactivity is not specific, and biodegradable dissolved organic carbon (BDOC) is produced.
Photocatalysis using TiO_2 is an effective way of treating wastewater. However, technology needs intensive light in order to function as required.

COMPETING INTERESTS

There is no competing interest with respect to the work reported in this paper.

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Keyword Index

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