Sustainable Drainage Systems

Edited by

Miklas Scholz

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Miklas Scholz (Ed.)

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About the Guest Editor

Miklas Scholz, cand ing, BEng (equiv), PgC, MSc, PhD, CWEM, CEnv, CSci, CEng, FHEA, FIEMA, FCIWEM, FICE, Fellow of IWA holds the Chair in Civil Engineering at The University of Salford (since 2010). He is also the Head of the Civil Engineering Research Group. About 47% and 44% of his research activities are in water resources management systems and wastewater treatment engineering systems, respectively. The remaining 9% are in capillary processes and water treatment systems. Prof. Scholz obtained his PhD from The University of Birmingham. He has shown individual excellence evidenced by world leading publications, postgraduate supervision and research impact. Miklas has published two books and 171 journal articles. Prof. Scholz's full journal article publications are: 2009 (13), 2010 (19), 2011 (13), 2012 (21), 2013 (17) and 2014 (15). Miklas has total citations of more than 2634 (more than 1914 citations since 2010), resulting in an H-Index of 27 and an i10-Index of 61. Prof. Scholz is Editor-in-Chief of the Web of Science-listed journal *Water* (impact factors for 2012 and 2013: 0.973 and 1.291, respectively). He has membership experience in over 30 influential editorial boards. Prof. Scholz is a Member of the Institute of Environmental Management and Assessment Council.

Preface

Urban water management has changed somewhat since the publication of the UK Sustainable Drainage System (SuDS) Manual in 2007, transforming from building traditional sewers to implementing SuDS, which are part of the best management practice techniques used in the USA and seen as contributing to water-sensitive urban design in Australia. Most SuDS, such as infiltration trenches, swales, green roofs, ponds and wetlands, address water quality and quantity challenges, and enhance the local biodiversity while also being acceptable aesthetically to the public. Barriers to the implementation of SuDS include adoption problems, flood and diffuse pollution control challenges, negative public perception and a lack of decision support tools addressing, particularly, the retrofitting of these systems while enhancing ecosystem services.

This book on SuDS disseminates recent findings on current challenges faced by practicing sustainable drainage engineers and scientists. Twelve papers were selected in a rigorous peer review procedure with the aim of rapid and wide dissemination of research results and critical reviews, as well as developments and applications of relevance to both academics and practitioners. Original research papers and reviews addressing the following and related areas were initially invited: infiltration techniques, ponds and wetland systems, adoption of sustainable drainage systems, climate change adaptation measures, public perception of sustainable drainage, integration of sustainable drainage into water-sensitive urban design, and SuDS decision-support systems.

This timely book focuses on sediment transport through swales, water sensitive urban design and green infrastructure tools, hydrodynamic performance of air–water flows in gullies, fecal coliform loads in urban watersheds, infiltration performance of pavements, bioretention challenges, climate change and urbanization. Furthermore, the increased importance of ecosystem services offered by SuDS became particularly apparent.

In times of recession, this book on modern SuDS has shown that expert systems supporting drainage engineers and scientists undergo a revival. However, the retrofitting of sustainable water structures is predominantly undertaken *ad hoc* using engineering experience supported by minimal formal guidance. There is a lack of practical decision support tools that could be used in different professions for the rapid assessment of potential ecosystem services that could be created when retrofitting water structures such as SuDS.

Thus an innovative decision support tool based on the rapid estimation of novel ecosystem service variables at low cost and acceptable uncertainty has been presented in this book. This novel and timely tool proposes the retrofitting of those SuDS techniques that obtained the highest ecosystem services score for a specific urban site after assessment by a representative of one of the recognized professions.

The estimation of variables was undertaken with high confidence and manageable error at low cost. In contrast to common public opinion, statistically significant differences were found between social scientists and the general public for the estimation of land costs using the non-parametric Mann-Whitney U-test. It was also surprising to find no significant differences in the estimation of habitat for species by civil engineers and ecologists. The new methodology may lead to an improvement of the existing urban landscape by promoting ecosystem services.

Miklas Scholz
Guest Editor

Urban Sediment Transport through an Established Vegetated Swale: Long Term Treatment Efficiencies and Deposition

Deonie Allen, Valerie Olive, Scott Arthur and Heather Haynes

Abstract: Vegetated swales are an accepted and commonly implemented sustainable urban drainage system in the built urban environment. Laboratory and field research has defined the effectiveness of a vegetated swale in sediment detention during a single rainfall-runoff event. Event mean concentrations of suspended and bed load sediment have been calculated using current best analytical practice, providing single runoff event specific sediment conveyance volumes through the swale. However, mass and volume of sediment build up within a swale over time is not yet well defined. This paper presents an effective field sediment tracing methodology and analysis that determines the quantity of sediment deposited within a swale during initial and successive runoff events. The use of the first order decay rate constant, k, as an effective pollutant treatment parameter is considered in detail. Through monitoring tagged sediment deposition within the swale, the quantity of sediment that is re-suspended, conveyed, re-deposited or transported out of the swale as a result of multiple runoff events is illustrated. Sediment is found to continue moving through the vegetated swale after initial deposition, with ongoing discharge resulting from resuspension and conveyance during subsequent runoff events. The majority of sediment initially deposited within a swale is not detained long term or throughout its design life of the swale.

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1. Introduction

Sustainable urban drainage systems (SuDS) are designed to control and treat surface water flow and pollution from the increasing impervious development of urban environs [1]. SuDS form part of a blue-green drainage network, the conveyance of stormwater through the urban environment via a network of ponding (blue) and ephemeral (green) vegetated stormwater treatment elements. Urban pollution is comprised of hydrocarbons, elevated nutrient levels, heavy metals, gross pollutants and sediment. Up to 85% of nutrients and heavy metal pollutants are conveyed from urban surfaces adsorbed to fine sediment, ranging from 1 μm to 2 cm [2]. The conveyance and detention of fine sediment is therefore a key indicator of SuDS efficiency, illustrating the transport and detention process of urban pollutants through the blue-green drainage network.

The efficiency of SuDS, including vegetated swales, has been investigated by leading SuDS researchers within the laboratory and in the field under single runoff event conditions. Both simulated and naturally occurring runoff events have been monitored during research completed by Sabourin and Wilson (2008) [3], and single runoff event specific pollutant removal efficiencies have been defined through analysis of this work. Deletic (2001) [4] reported that swale total suspended solid reduction in initial event flows range between 78% and 86%. However, methodological limitations associated with long term source-pathway-sink monitoring of sediment movement through SuDS assets has resulted in limited extended case study research and analysis.

Current best practice employs an arbitrary swale design life of 25–30 years. Understanding of maintenance requirements for a swale beyond litter removal and grass cutting is limited. The long-term effects of multiple rainfall-runoff events through a swale on temporary or long term sediment deposition and removal is not clearly understood. This has led to uncertainty in defining maintenance needs, long-term design efficiencies and best practice.

To address this knowledge gap, field research was undertaken to identify the quantity of sediment from a single release that remains within a vegetated swale over an extended time period. To calculate this, it is necessary to define whether sediment deposited with a swale remains stationary or if it becomes re-suspended and transported due to subsequent runoff events. To create this sediment transport dataset, an effective sediment tracing method was identified and used to illustrate the long-term process of sediment transport in an established urban swale. The trace methodology was required to define the movement of a single sediment release within the total mass transport within the swale.

To ensure the movement of a single sediment release could be monitored over time within the swale, it was necessary to identify a trace that had long-term field resilience, was not lost from the system through sunlight exposure, plant uptake and was not transported through the vegetated environment other than by adsorption to sediment. The trace required multiple unique identifiers, supporting monitoring of multiple individual sediment releases over time.

The selected trace methodology was used on an established vegetated swale in Edinburgh, Scotland. Event and extended field sample analysis identifies the temporary and extended detention efficiency of this established urban swale. The research findings presented in this paper provide recommendations on the resulting efficiency and may be used in defining the assets maintenance needs over the life of the swale.

2. Sediment Tracing Methods

Sediment tracing has traditionally been used in agricultural research settings, investigating field and bank erosion source and processes. River banks and sand bar deposition monitoring use a range of natural sediment tracing techniques, including fingerprinting. There is an extensive range of sediment tracing methods available, from invasive chemical or physical tagging to passive photographic monitoring. The benefits and constraints of the more frequently employed techniques are listed in Table 1.

The blue-green drainage network, into which tagged sediment is released, has environmental value and importance. It is necessary that the trace used in long term monitoring not only be effective in mimicking natural sediment movement but also result in no detrimental impact on the receiving environment.

In conjunction with environmental impact considerations, the key requirement of the sediment trace method for this research was to clearly define the movement of a single sediment release within the total mass transport of a swale over an extended period of time. It was important that the trace not only stay adsorbed to the sediment for months without concentration degradation by environmental influence, but that it be available in several unique forms. These would provide unique trace signatures enabling individual sediment releases to be monitored over time within a single swale, and therefore repetition of the field experiment. Monitoring of a single sediment release over extended time periods through a SuDS is novel, and comparative datasets are not yet published. Therefore, to create this sediment transport dataset a sediment trace methodology specific for this purpose had to be created.

Of the sediment trace methods outlined in Table 1, several do not easily provide multiple unique trace signatures (total suspended solid/PSD analysis; synthetic and magnetic particles). Pollen and magnetic fluorescent material tracers are limited in availability, pollen by the natural availability and fluorescent particles by the artificial fluorescent colours available. Painted natural particles have limited field resilience, and radionuclide [3,4] tracers have been recorded to move both adsorbed and without adsorption to sediment across natural surfaces [5]. Furthermore, the use of radionuclides requires environmental agency permission in many locations, limiting the ease of method availability. Fingerprinting is an effective watershed erosion and sediment [6,7] transport tracing method. It uses the multiple naturally occurring periodic element concentrations and particle size distribution to determine a sediment source. Where the range of sediment sources have distinctly different signatures, for example forestry erosion *versus* agricultural wash-off or urban sediment, the fingerprinting method is effective. However, sediment entering an urban swale derives from road, car park and roof surfaces within the developed area. While the particle size and heavy metal concentrations differ between these sources, the source specific signatures are not easily discernable. Therefore, it is more difficult to employ the fingerprint method within the urban environment.

Rare earth oxides (REO) provide an alternative to the above sediment trace methods, providing 17 clearly identified trace signatures. REO adsorb easily to natural sediment and have shown limited field detachment in laboratory testing [6,7]. REO tracing has been used in agricultural scour and erosion research and is therefore untested in the urban SuDS environment. However, given the trace properties, it was selected for this research. The trace methodology, previously used predominantly within the laboratory, was modified to achieve single sediment release field monitoring within a swale during multiple runoff events.

Table 1. Overview of sediment tagging methods.

Trace Method	Number of Identifiers	Activity Period in Natural Environment	Recorded Use	Potential for Utilization in Urban Environment	Source
Radionuclides	numerous	30–40 years	Study of erosion and deposition in the landscape, chronometer for sediment deposition in ponds, lakes and floodplains, agricultural sediment erosion, catchment erosion and deposition in lakes.	Effective. Long activity time results in potential difficulty in replicability. High resource requirement.	[5,8–11]
Fingerprinting	numerous	Natural particle life cycle	Watershed/ catchment scale sediment budget analysis. Sediment source analysis.	Effective but requires chemical signatures to be significantly different between sediment sources. Requires technical support and laboratory equipment (AAS) and sampling for numerous chemical concentrations.	[11–15]
Painted/coated natural particles	numerous	Limited time frame due to low trace adhesion/adsorption to sediment particle. Solar degradation may shorten field activity period.	River bank erosion, sediment transport though fluvial networks, larger sediment, pebble and gravel tracing.	Highly visible. Difficulty in separating coated material from remaining sample sediment.	[16–18]
Magnetic particles	1	Extended dependent on synthetic material (coating) chosen or natural magnetism	Soil erosion within a watershed. Sediment loss and detachment from source.	Artificial material limiting natural assimilation or breakdown. Natural magnetism has limited unique signatures.	[19–21]
Magnetic fluorescent material	4	Extended dependent on the particle material. Fluorescent activity is extended due to the particle coating and design	River sediment transport. Piped network sediment transport.	Supports monitoring without loss of material from the field environment. Easily separated from total sample sediment. Highly visible.	[16,22,23]

Table 1. *Cont.*

Trace Method	Number of Identifiers	Activity Period in Natural Environment	Recorded Use	Potential for Utilization in Urban Environment	Source
REO	17 (15 readily analysed)	Extended (months–years)	Particle translocation. Surface erosion, due to rainfall-runoff, overland flow, sediment transport from multiple sources. Agricultural erosion. Solute/suspended sediment redistribution in snow, ice, urban, agricultural and rural environments.	Not visible. Limited environmental impact. Significant identifiers. Shown to be effective in alternative conditions. Untested in the urban environment but meets urban monitoring requirements.	[24–26]
Pollen	Limited to natural vegetation pollen availability	Annual time frame (not event specific) to decades	Vegetation and land use histories (chronometer). Pollen peak correlation with annual sediment erosion and deposition. Ability to trace sediment to source, when source is from natural (vegetated) surfaces.	Limited due to activity period limitations. Complexities relating to urban surface type, urban source and grassed/vegetated areas that comprise the SuDS.	[27,28]
Synthetic/ artificial particulates	limited	Extended (similar to natural particles)	Mass transport flux, TSS concentration and bed load change.	Difficult to consider source to sink movement unless limited to a single source within the network under consideration, due to limited identifiers. Replicability difficulty may not effectively mimic natural sediment characteristics.	[16,18]
Total Suspended Solid balancing and PSD analysis	limited	Extended (similar to natural particles)	Mass transport flux, suspended solid concentration change, PSD change related to influence of rainfall and source contribution (high level).	Limited to flux and balance analysis. Difficult to identify source from PSD and mass change alone.	[29–31]

3. Rare Earth Oxides

Rare earth oxides are elements naturally found within soil and bed material. They form the lanthanide group of elements within the periodic table and are classified as rare due to their very low concentration within the shallow layers of the earth's crust. The rare earth element group is comprised of lanthanides, scandium, and yttrium. As rare earths occur naturally in soil at very low concentrations, parts per billion, the analysis of natural rare earth concentrations requires strong acid digestion and assessment by inductively coupled plasma mass spectrometer (ICPMS) [32].

Rare earths have been used in agricultural scour and erosion research to monitor sediment movement [24,30]. Zhang *et al.* (2001) [6] first published rare earth tracer methodology in 2001, illustrating rare earth elements strong binding capacity to soil and low mobility after attachment due to leaching. Rare earth elements have been successfully used as unique, single signature sediment tracers to monitor soil movement through agricultural media in a laboratory setting [25,33]. The rare earth group have 15 easily analysed, unique, single element signatures that adsorb strongly to a wide variety of particle sizes (<0.01 to >4.75 mm). Adsorption of rare earth oxide (REO) occurs though preferential bonding [34]. In the natural drainage and soil environment, there is no significant leaching or movement of REO from tagged sediment to surrounding material [6]. REO are not taken up by vegetation, therefore, being appropriate for use within the blue-green drainage network, and do not naturally degrade in sunlight or de-stabalise over time [27,35]. Due to the extended field activity period (months to years), the high number of unique identifying signatures and the limited impact on the receiving natural environment, REO tracers have potential as highly effective urban sediment tracers.

Rare earth tracing, while noted to achieve effective integration with tag material, low or no solubility in water, limited plant uptake, no eco-environmental damage and to exist in very low natural concentrations [7], there are several limitations to REO tracer use. Tracer enrichment may occur due to an increase in tracer mobility with increasing soil or runoff acidity [7]. REO also preferentially bind to fine particulate material, silt and clay particles [36]. Therefore, where a large particle size distribution (including coarse sediment, sand or gravel) is used in a trace experiment, there may be an over or underestimation of REO concentration due to REO tracer transference [36]. Research in REO tracer enrichment due particle size re-distribution during erosion experiments suggests a potential error of 4% when considering a particle size range from 8 mm to below 0.9 mm [25,26,33,36].

4. Field Site and Experiment Methodology

An established, maintained, active urban swale was selected for the field trials. The swale is located within Heriot-Watt University grounds, Scotland. It is located parallel to a local road and collects runoff directly from this road network. The swale has a mild grade (less than 2%), is over 100 m in length, grassed and conveys stormwater runoff from a 500 m^2, 40% impervious, urban developed area to a piped stormwater network. Runoff from the contributing area is conveyed to the road and enters the swale via curb inlets along the road. The road has a single camber, therefore, insuring all stormwater flows to this swale.

The field experiment was designed to allow one sediment release of REO tagged material at the commencement of the monitoring period. This sediment, equating to $^1/_4$ of the annual average sediment loading, was released onto the impervious surface (road) upstream from the swale inlet. 10 kg of dry, tagged sediment was evenly spread across a 10-m long, 1-m wide strip of road upstream from the swale inlet. The tagged material was then washed off the road surface by a 30 min long, three-month return period runoff event. The runoff event was artificially created using a pressurized local water source (fire hydrant) and a level spreader was employed upstream from the sediment release location to allow runoff to sheet flow across the road towards the swale inlet.

Sediment was tagged following the detailed process described in Zhang *et al.* (2001, 2003) [6,25]. Tagged sediment was designed to be representative of the known sediment occurrence on urban roads. Road sweeping collection and particle size analysis was completed at the field site, and this, in conjunction with literature review of urban road sediment particle size distribution and loading, defined a representative sediment sample characterization (d50 = 60 µm and 50 ton/km^2/year) [37,38]. There is limited guidance on the effective concentration of REO trace to sediment ratio, and REO tracing has been limited to agricultural sediment tracing conditions to date. Literature suggests that in an agricultural scour tests in laboratory environments a concentration of 5–100 g/kg may be appropriate for effective signature analysis [26,27,35]. Deasy and Quinton (2010) [26] undertook field tests using up to 500 g/kg of REO trace to ensure a clear trace signature was created in the field environment. The nature of a trace is to provide detailed sediment transport information without significant influence to the receiving environment or sediment dynamics. Therefore, it is important to identify the minimal concentration of sediment trace necessary to effectively monitor sediment transport activity in the field without compromising the results due to weak signature strength.

To identify the effective trace concentration necessary for swale sediment transport tracing, the experiment was replicated using two unique rare earths (La and Nd) at different trace concentrations (10 g/kg and 100 g/kg respectively). The assumption that sediment in both experiments sediment should move in a similar way, providing a similar trend pattern in REO concentration) allowed trace concentration influence on signature clarity and effective (minimum) trace concentration to be defined. It should be noted that background REO concentrations (of both artificial runoff and swale soil) were low, below ppm analysis levels.

Using a local water source the first runoff event was artificially created. Tagged sediment was placed upstream from the swale inlet prior to runoff event 1. Runoff event 1 then created flow over the sediment laden road surface and entered the swale. The event ceased after 30 min, and a one-hour drying period was provided.

A second artificial runoff event, of the same duration and intensity as runoff event 1, was then artificially created. No further sediment was placed on the upstream impervious area but surface flow was allowed to follow the same path as runoff event 1. After a one hour drying period, a third artificial runoff event was created, of the same duration and intensity at the previous two runoff events.

Figure 1 provides a schematic layout of the monitoring and sampling undertaken during and after each flow event. All sediment-laden runoff entered the swale 40 m upstream from the grated downstream outlet. During runoff events 1, 2, and 3 grab samples were collected from surface flow at three locations within the main flow path of the swale. It is acknowledged in selecting this sample method that surface sampling, in the form of grab samples, may not provide detailed accurate representation of suspended sediment concentrations where sediment particle size distribution is large. Swale surface samples were collected from 1 m downstream from the swale inlet (upstream location);

20 m downstream from the inlet (central section of the swale); and one meter upstream from the outlet (downstream location). Surface samples were collected at all three locations at 5 min intervals throughout the runoff events.

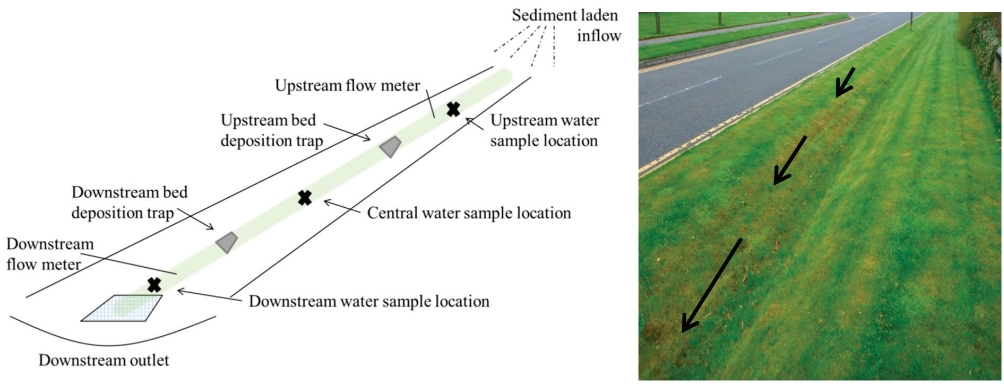

Figure 1. Schematic swale diagram.

Between runoff event 1, 2, and 3 runoff was allowed to discharge from the swale. At the cessation of swale flow, sediment deposition samples were collected from gravel bed traps placed in the swale bed at two locations (corresponding with the upstream and downstream surface sample locations). The sediment traps were square collection trays inset into the swale bed, filled with gravel and sized to collect up to 2 mm sediment particles transported by rolling, saltation or deposited on cessation of runoff flow.

Flow depth and velocity were monitored at the upstream and downstream extent of the 40-m swale reach. Stingray ultrasonic sensors were anchored on the swale bed and continuously logged flow depth and velocity from the commencement of runoff event 1 until cessation of swale flow from runoff event 3. This recorded the inflow and outflow for each runoff event supporting flow relative comparison of sediment transport results.

Once the artificial runoff events were completed, core samples to 0.02 m depth were taken at five-m intervals down the central flow path of the swale. Core samples were taken immediately post experiment completion, one week, six months, and 12 months after the release of trace tagged sediment on the upstream road surface.

The REO concentration in all samples, runoff event surface samples, bed deposition and core samples, were analysed using an ICPMS. To detach REO trace material from sediment, samples must undergo strong acid digestion [6,25]. Surface and bed deposition samples were thoroughly shaken and 50 mL of suspended sample material was processed using strong acid digestion methodology. Core samples were dried at 105 °C for 24 h. Individual dried samples were mixed thoroughly and two grams of sample material was prepared for ICPMS analysis through strong acid digestion. Filtered digestion liquid was tested by ICPMS to define sample REO concentrations. It should be noted that runoff event water and background soil samples were also tested to provide background REO concentration levels.

5. REO Trace Results

REO concentrations within runoff event flow, bed deposition and core samples taken over the six-month period were collated with swale flow depth, velocity and rainfall records. The REO trace provides a clear signature at both 10 g/kg and 100 g/kg trace concentrations throughout the 40 m monitored reach of the established swale. Figure 2 presents trace concentrations during runoff events 1, 2, and 3 and demonstrates that the presented REO trace methodology is effective in illustrating sediment transport through an urban vegetated swale under ephemeral conditions.

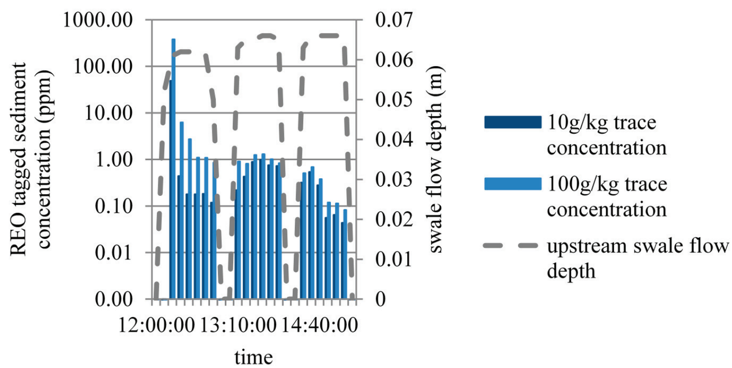

Figure 2. Tagged sediment concentrations at the upstream swale monitoring location.

REO tagged sediment of two selected tracer:sediment ratios were released. Figure 2 illustrates that both the 10 and 100 g/kg REO to sediment ratios appear to function as effective tracers within a blue-green network. The two REO tagged sediment material show concentrations that follow a similar trend when analysed at part per million concentrations by an ICPMS. The concentration of sediment entering the swale during event one follows the same curve and results in tagged suspended sediment (TSS) concentrations of similar value.

There is a magnitude shift in the TSS concentration values seen in runoff event 1. The amount of 100 g/kg tagged sediment is 8 to 10 times greater than the 10 g/kg tagged material. However, runoff events 2 and 3 show a comparable quantity of tagged sediment in the samples, as would be expected. The cause of the elevated 100 g/kg tagged sediment results during runoff event 1 is due to the absorption maxima for the tagged soil composition being reached. The increased flush of REO trace during this first runoff event is a result of excess trace being transported through the swale in suspension. Within this field research, a range of particles sizes were used, with tag media comprised of both sand and clay. Laboratory analysis undertaken by Kreider (2012) [39] suggests clay/silt material adsorption maxima to be 12,400 ppm while a range from 1900 to 43,000 mg/kg presented is in Spencer *et al.* (2007) [35]. While it is acknowledged that these adsorption maxima are not specific to the tag material used in this field research, the 100 g/kg REO concentration is noted to be significantly above these adsorption levels. Thus, while past REO trace research has used up to 100 g/kg trace to sediment tag rates, the flush of 100 g/kg REO trace in solution during the first runoff event highlights the sensitivity of tagged material composition to REO trace use.

Considering the REO signatures created by both 10 g/kg and 100 g/kg trace concentration, the lower concentration trace was selected for future sediment trace field research, minimizing the amount of material released into the environment and receiving waterway. It should be noted that concentration errors due to enrichment from the swale soil source are assumed to be insignificant, due to the low background REO concentrations.

Runoff event specific sediment detention for a swale is expected to be approximately 90% [3,4,40]. Analysis of the REO concentrations for the initial (runoff event 1) 30 min event agreed with general sediment treatment expectations. The sediment detention within the swale as a result of runoff event 1 was between 90% and 98% for all experiment repetitions.

The three monitored runoff events provided tagged sediment transport concentrations respective to the runoff event (1, 2, or 3). As would be expected, the initial event (runoff event 1) showed elevated upstream concentrations and the highest concentration relative to subsequent events (Figure 3, upstream). Within each single runoff event, the REO concentration decreased progressively down the swale (moving from the upstream to downstream sampling location); however, variance is illustrated between the extent of this decrease between each event.

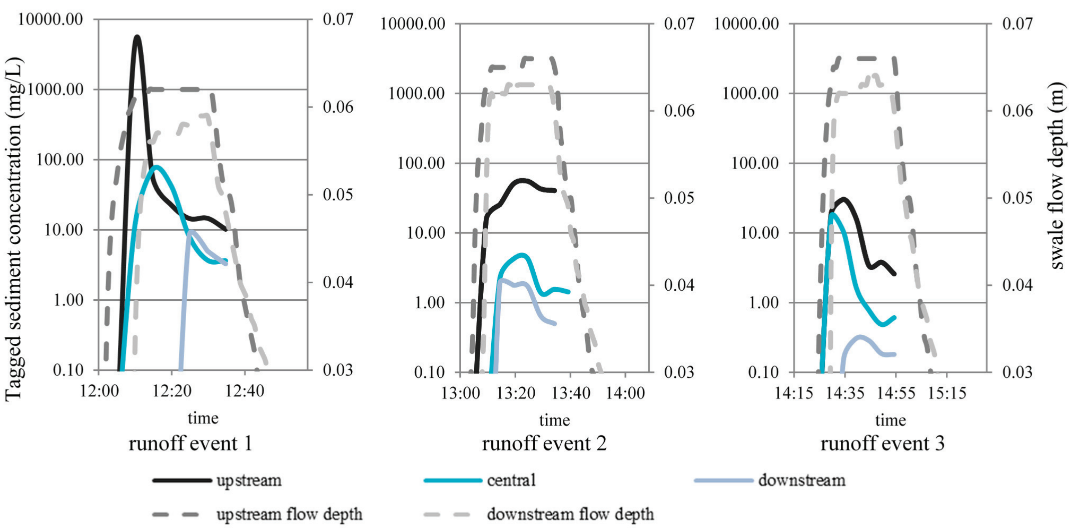

Figure 3. REO tagged sediment concentrations for artificial runoff events at the three surface runoff monitoring locations within the swale (upstream, central, and downstream locations)—Experiment 1 results.

Runoff event monitoring illustrated a rise in REO concentration occurring with the commencement of each flow event (Figure 3). During runoff event 1, this peak was approximately five times the average event concentration. Cristina and Sansalone (2003) [41] and Ellis (1996) [37] considered the high fine sediment concentration in urban stormwater movement and the occurrence of elevated sediment concentrations initiated by stormwater flow (first flush principles). The peaks illustrated within these results show a sediment concentration increase as a result of runoff flow entering the swale, but the trace concentration peak occurs concurrent or after the runoff flow peak and therefore is not considered to be a first flush occurrence. The peak in sediment concentration within the sediment pollutograph is considered to occur as a result of runoff flow movement, the initiation of transport as a direct result of the introduction of flow to a dry flow path, where rainfall is greater than the loss to infiltration.

Of interest is the change in concentration at each specific monitoring location over the three flow events. It is anticipated that the upstream concentration decreases over time, as illustrated in Figure 3. Small flow initiated concentration increases occur in events 2 and 3 at the upstream sampling location. No further sediment was applied to the upstream impervious area of the runoff flow path, thus the increase in upstream tagged sediment concentrations during runoff events 2 and 3 do not occur through continued introduction of tagged sediment from the road. Hussein *et al.* (2007) [42] undertook detailed experimental research to identify the dynamics of sediment transport from an impervious (low manning's n, 0.016) surface into a vegetated flow path (manning's n of 0.025–0.035). Their research findings illustrate a sediment deposition zone at the impervious/vegetated surface interface. This sediment deposition zone, occurring at the vegetation boundary where runoff enters the swale (within this field experiment) is found to act as a temporary sediment storage area. During runoff event 1, tagged sediment from the upstream impervious area became temporarily detained at this vegetated boundary. As successive runoff events occurred, runoff events 2 and 3, the sediment deposited at this vegetation boundary became entrained and entered the swale, therefore creating the upstream-tagged sediment concentration elevations within these runoff events (2 and 3).

The REO trace concentrations were found to generally decrease during the ongoing flow event. Concentrations decreased by 83%–99% of the inflow sediment concentration. The smaller REO concentration peaks associated with the commencement of runoff events 2 and 3 suggest resuspension or continued influx of REO tagged sediment within the monitored swale length. While no further sediment was introduced into the system during these following events, the upstream vegetation boundary was noted to have a potential influence over sediment inflow into the swale [42–44]. The receiving swale vegetation edge appears to act as a temporary detention zone, supporting ongoing sediment release into the swale with additional events. The REO concentration peaks at the commencement of event 2 and 3 are notably smaller than in event 1, however the persistent occurrence of these flow initiated peaks supports the inclusion of vegetation boundary influence in swale sediment balance analysis.

The continued decrease of tagged sediment concentration during runoff events 2 and 3 illustrate a continued transport of sediment through the swale. Sediment entering the swale during runoff event 1 is shown to travel downstream (Figure 3), while runoff events 2 and 3 illustrate a flow driven sediment pulse that is also shown to move to the downstream monitoring location. There is a general decreasing tagged sediment concentration trend for upstream and downstream monitoring locations over the three runoff events. While the overall average REO concentration over the three events decreases for the central monitoring location, there is a notable increase in peak concentration. This inconsistency in concentration flux may illustrate the influence of internal swale sediment resuspension resulting from subsequent flows.

The sediment trapping efficiency of the swale was calculated simply through comparison of the REO concentration entering and leaving the swale during each flow event. The tagged sediment concentrations shown in Table 2 illustrate the decreasing tagged sediment trapping efficiency of the swale in runoff events 1, 2, and 3 for the single sediment release. The first and second repeat of the trace and artificial runoff event results are provided in Table 2 to illustrate consistency in the tagged sediment trapping efficiency trend of the swale. REO tagged sediment continued to leave the swale during the second and third flow event, decreasing the quantity of sediment permanently detained within the swale. This supports the theory of continued sediment resuspension due to subsequent flows through a blue-green drainage system, and that the assumption that sediment detained within the initial event will remain within the swale in perpetuity is inaccurate.

Table 2. Summary of sediment trapping efficiency (tagged sediment concentration leaving the swale-the total tagged concentration entering the swale) during artificial flow events (for two replicate artificial runoff experiment sets).

Experiment	Runoff Event	Sediment Trapping Efficiency (Retention of Tagged Material)
1	1	98%
	2	97%
	3	84%
2	1	95%
	2	75%
	3	67%

Swale bed deposition was collected between each flow, using sunken sediment taps within the swale central flow path. Two sediment traps were set within the monitored swale reach. The REO concentration for each runoff event deposition is illustrated in Figure 4a. Similar to the function of a vegetated filter strip, the upstream receiving vegetated flow path detains a more significant amount of sediment than further downstream [31]. Deposition at the downstream extent is between 90% and 95% lower than upstream. Furthermore, the deposition decreases over subsequent events, supporting the theory of ongoing movement and deposition of REO tagged sediment material through the swale.

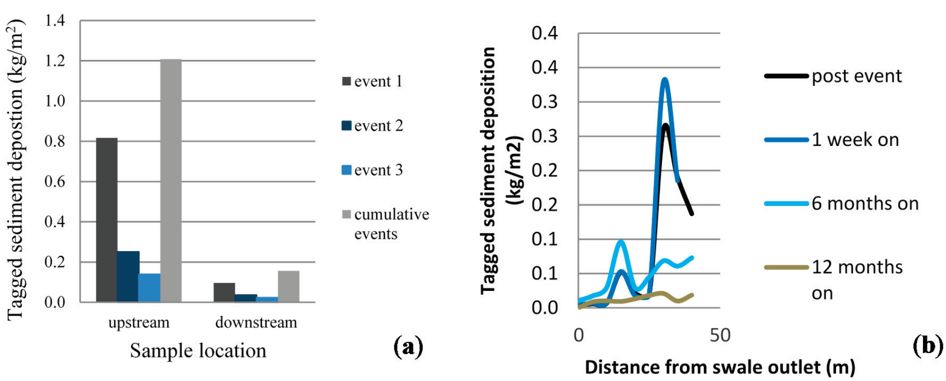

Figure 4. Deposition of tagged sediment within sediment traps placed in the base of the swale: within the swale between runoff events (**a**); and over the following 12-month period (**b**).

Core samples taken at five-m intervals across the centerline of the swale over a twelve month period indicated that re-suspension and deposition continued to occur. Over the monitoring period the quantity of REO tagged sediment within the swale flow path depletes within the upstream extent (70%–75%). The REO tagged sediment peak moves down the swale over time, from the upper 30–40 m swale point (30–40 m upstream of the outlet) to 10–20 m location over six months. Figure 4b illustrates a slow continuous sediment resuspension and deposition process that moves sediment from the initial release consistently downstream over time. After six months, the concentration at the downstream extent of the swale was noted to be greater than immediately after the initial flow events. Of the REO tagged sediment initially deposited within the swale (0.8 kg/m^2), up to 0.1 kg/m^2 remained deposited after six months. Considering the area under each time stamped deposition curve in Figure 4b, the net tagged sediment loss (REO tagged sediment mass balance loss) between post event samples and six months on is 38%. This indicates the quantity of tagged sediment that has been re-suspended and conveyed out of the swale during subsequent events during the six-month period, a continued decrease in detention efficiency due to ongoing flow events through the swale.

6. Analysis and Discussion

6.1. Cumulative Runoff Event Sediment Detention within the Swale

The rate of deposition and sediment detention over cumulative runoff events, and therefore time, is key to clarifying swale long-term efficiency in stormwater treatment for water quality improvement. Figures 3 and 4 highlight the flow driven sediment transport process and the potential for re-suspension and distribution of sediment across a swale over time.

The rate of sediment loss from the swale is directly related to runoff event occurrence, illustrated in Figure 4b. Extending this simplistic relationship across the across the field monitoring period provides a trend in detained sediment concentration within the swale. This trend shows that the quantity of sediment, from the initial tagged sediment release, detained within the swale continues to decrease as the number of runoff events flowing through the swale increases.

Field data has been collected for a period of 12 months. Using the field results, the trend in sediment deposition relative to the cumulative runoff event occurrence for one sediment release was calculated and plotted (trend line illustrated in Figure 5). However, swale design life expectancy extends 25–30 years. To provide an insight into the sediment deposition occurring within a swale from one sediment release over an extended period, multiple runoff events in excess of that which occurred during the field monitoring period need to be considered. Using the long-term site rainfall records, the expected number of runoff events over a period of 1 to 25 years equal to or greater than the three-month rainfall depth were determined. Extrapolating from the field tagged sediment deposition results, extended cumulative runoff influence on tagged sediment deposition was considered (illustrated in Figure 5 as the light blue points).

Figure 5 illustrates the estimated extended sediment deposition from the field results based sediment deposition trend (for one tagged sediment release) out past 100 rainfall-runoff events. The exponential rate of detention efficiency decrease determined from the field test values (the field test trendline) was used with historic rainfall data to estimate the potential sediment deposition within a swale, from a specific initial inflow, after multiple rainfall-runoff events. This simplistic extrapolation allowed the estimation of sediment deposition remaining within the swale after 25 years of rainfall-runoff events.

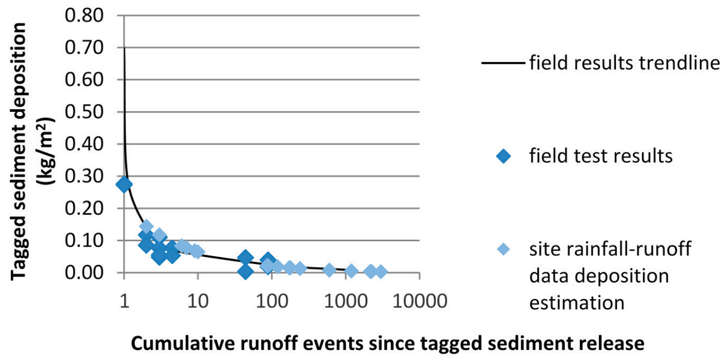

Figure 5. Field monitored and empirically estimated trace sediment deposition within the swale over multiple runoff events.

The trend suggests that there is a continued but small resuspension and release of tagged sediment over cumulative runoff events, resulting in a small long-term sediment deposition quantity (from one single sediment release) over an extended period.

It is acknowledged that this is a simplistic approach to sediment deposition estimation within this swale, however it is also one of few field based deposition extrapolations and thus provided some new evidence of ongoing sediment release from a swale as the result of cumulative runoff events. As illustrated in the field tests, greater sediment deposition occurs during initial runoff events. As the time after initial sediment entrance into the swale increases, and the number of runoff events occurring during this period also increases, the quantity of sediment remaining within the swale from the initial runoff event decreases exponentially. The relationship between tagged sediment deposition within the swale is relative to the number of events occurring over the reviewed time period. The influence of intensity and duration of the runoff event is less significant that the occurrence of the event itself, suggesting that the influence of flow entering the dry swale is a driver in sediment resuspension within this swale.

From Figure 5, the estimated tagged sediment deposition with this swale after two years (an example maintenance period for a swale) located in Edinburgh would be 0.02 kg/m^2 (8% on the initial release). This is the quantity of sediment from a single sediment release estimated to remain within the swale after 180 runoff events (greater than the threshold). Over a 25-year life cycle of a swale [45], the sediment load remaining within the swale from a single initial sediment release or entrance is estimated as 0.01 kg/m^2. To consider the sediment potentially remaining deposited within the swale 25 years after it becomes operational, a cumulative approach is needed. If it is assumed that a sediment volume equivalent to that tagged and released in the field experiment represents a three-month runoff sediment influx, and that this occurs effectively 100 times over a 25 year swale design life at relatively regular intervals, then a gross estimation of detained sediment mass (considering the ongoing runoff event sediment transportation out of the swale) for this swale would be approximately 8 kg of sediment. This residual mass is relative to the period of swale operation and number of runoff events occurring during this period, therefore incorporating the residual sediment mass from events 24 years to three months previous to the 25th swale year. The 8 kg sediment deposition is approximately 3% of the total sediment mass entering into the swale every three months over the swales lifetime. A significant proportion of urban pollutant is, therefore, conveyed downstream through a swale over time. While is it noted that significant assumptions and simplistic extrapolation has been undertaken to estimate this design life sediment deposition quantity, it does highlight that further research is required to accurately consider multiple event and extended period swale functionality. If the assumptions and extrapolation are accepted, then a single, initial runoff event stormwater mitigation analysis to calculate a swale sediment detention efficiency may not accurately represent the long term sediment detention efficiency of a swale.

6.2. First Order Decay Analysis of Swale Sediment Mitigation

The current accepted method to analyse pollutant removal efficiencies, especially for SuDS and blue-green drainage assets, is through first-order kinetic decay pollutant removal estimation. This method employs a CSTR or plug flow assumption regarding pollutant transport and treatment [45,46]. The first order decay model is well established in pollutant modeling and has been utilized within SuDS and stormwater management models such as MUSIC [46], and is described in Wong *et al.* (2006) [47] Equation (1)) as:

$$q\frac{dC}{dx} = -k(C - C^*) \tag{1}$$

where $q\ dC/dx$ = the rate pollutant concentration moves towards an equilibrium or background concentration with proportional distance along the treatment measure; C^* = the background concentration (mg/L); q = hydraulic loading rate (m/yr), the ratio of inflow and surface area of the system; x = the fraction of distance from the inlet to outlet; C = the concentration of the water quality parameter (mg/L); k = areal decay rate constant (m/yr) [47].

k is defined as a constant rate of change [4,47,48], the time taken for a pollutant concentration to change from its initial inflow concentration to the final attenuated, deposited and detained concentration [49]. This equation acts to describe the overall movement of pollution from an event based pollutant influx to an equilibrium or background pollution level. It is used to describe total suspended solid, nitrogen, phosphorus and biological oxygen demand pollutant treatment efficiencies of SuDS [47].

An alternative published description of the first order decay rate currently used in SuDS pollutant removal efficiency analysis is:

$$C_{out} = C^* + (C_{in} - C^*)\ e^{-\mathrm{k}/q} \tag{2}$$

Equation (2) is quoted from Wong *et al.* (2002) [46]. Within the published paper the equation parameters are described as the following:

C_{out} = output concentration (mg/L);
C_{in} = input concentration (mg/L);
C^* = background concentration (mg/L);
q = hydraulic loading rate (m/yr);
k = decay rate constant (m/y).

Equation (2) provides a continuous stirred tank reactor (CSTR) first order decay model [46]. This differs from Equation (1) in that it considers “lumped” pollutant removal rather than comparative distance (x) through the SuDS pollutant concentration change. Where Equation (2) considers the pollutant concentration only at the inlet and outlet, the total overall SuDS asset pollutant removal achievement, Equation (1) allows inter-event assessment and consideration of the internal SuDS asset function (as a function of the linear pathway between inlet to outlet, as a function of *x*).

The first order decay model is generally employed for steady state specific event analysis. Best practice guidance for k-C^* modelling provides expected k constant values. These range from 4000 to 15,000 m/yr [48]. Rearranging Equation (2), the change in pollutant concentration can be calculated using the representative decay rate constant (k) relative to the SuDS asset hydraulic loading rate ($e^{-k/q}$).

Multi-event sediment deposition and surface sediment samples collated from the field experiment were used to identify the k constant relevant to this swales performance. k was calculated using both Equations (1) and (2), to incorporate pollutant treatment using both CSTR and proportional distance through the SuDS system methods. Using the known C_{in}, C_{out} and C^* values for each event and the hydraulic loading rate, the field experiments concentration rate of change was calculated and compared to expected decay rate constant k.

The field experiments illustrate that over multiple rainfall-runoff events, k does not perform as a constant. The field trial concentration rates of change (the rate of sediment detention within the swale) for the first flow event is greater than k = 15,000 m/yr. k values decrease as events accumulate (a decrease over event 1 to 3), with k values falling to 6000 m/yr. Field trial sediment conveyance rates relative to specific events do not conform to the k constant rule, k values ranging from 6000 to 23,000 m/yr. The greater the k value, the less sediment is conveyed through the swale during an event, suggesting greater swale sediment detention efficiency. Figure 6 suggests the k-C^* model may effective for single initial event analysis, but requires further consideration and expansion to effectively describe subsequent flow event impact on pollutant decay rates over time.

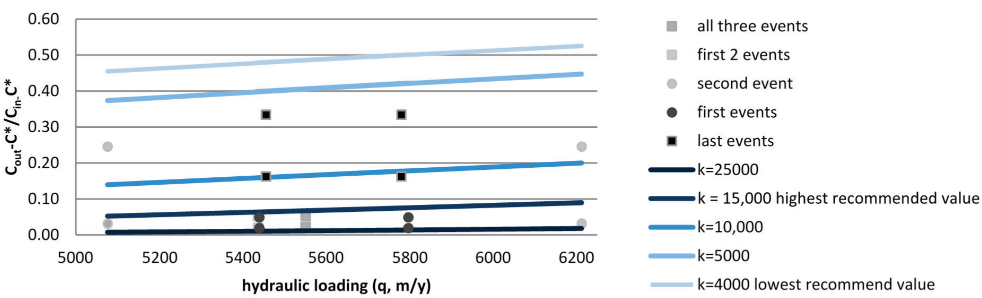

Figure 6. Pollutant concentration change relative to hydraulic loading across the swale.

k values estimated through the k-C^* model using field trial results show a higher concentration rate of change as subsequent flow events occur. As illustrated in Figures 5 and 6, sediment detention efficiency decreases with an increase in the number of flow events. The largest k value occurs as a result of the initial flow event, with subsequent events resulting in a decreased detention rates.

Deletic (2005) [50] undertook detailed grass filter strip event specific sediment transport analysis. Her research defined several key influences over the event specific sediment conveyance and deposition process, including an explanation for runoff event specific trapping efficiency due to stormwater flow over vegetation. Trapping efficiency (Trs) is a function of the amount of sediment entering the swale ($C_{s,in}$) and the sediment load at a sampling point x distance downstream from the inlet ($C_{s(x)}$).

$$\mathrm{Tr_s(x)} = \frac{\mathrm{C_{s,in}} - \mathrm{C_{s(x)}}}{\mathrm{C_{s,in}}} \tag{3}$$

where,

$\mathrm{Tr_s}$ = inflow sediment load of fraction *s*;

$\mathrm{C_{in,s}}$ = inlet sediment load of fraction *s* (mg/L);

$\mathrm{C_{out,s}}$ = outlet sediment load (at monitored point of fraction *s* (mg/L);

X = distance from the inlet of the SuDS (m) [51].

The trapping efficiency, Trs(x), was calculated using field experiment data. Monitored flow and REO concentrations during each of the replicated field trial events allowed calculation of Trs(x) as well as k. Figure 7a illustrates that the trapping efficiency is not constant across all events, but does illustrate the expected direct relationship between rate of concentration change and trapping efficiency within the asset. k is the consistent influence in the removal rate or rate of decay, and, therefore, should illustrate some relationship to the assets trapping efficiency.

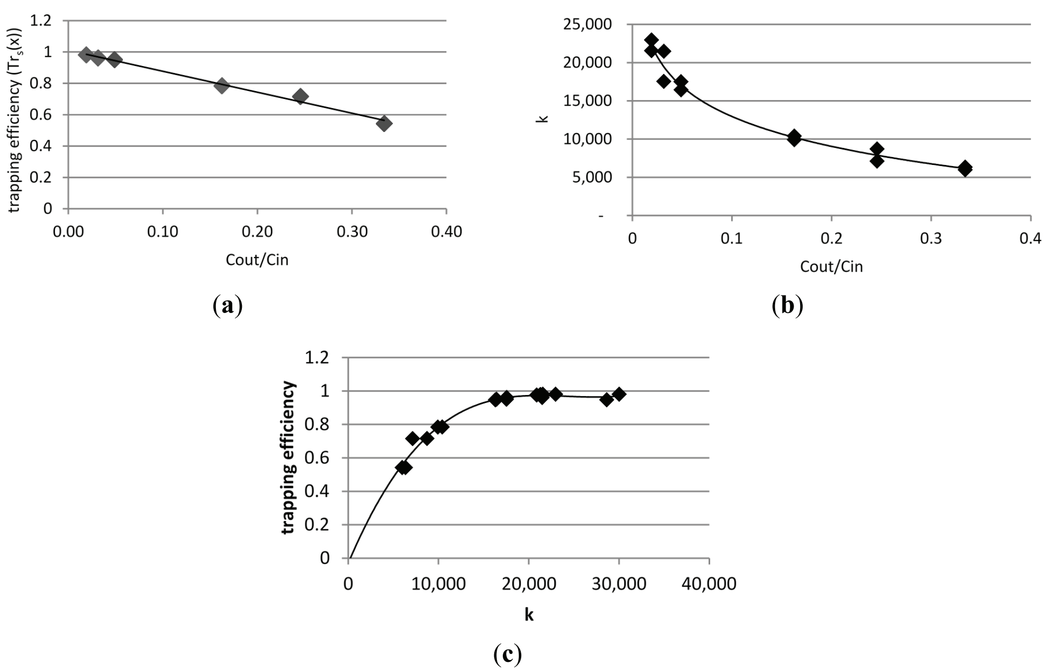

Figure 7. Correlation between sediment detention rate and (**a**) trapping efficiency; (**b**) first order decay rate constant k; and a comparison of (**c**) trapping efficiency and first order decay rate constant k.

Figure 7c compares the field trial trapping efficiencies calculated using Equation (3) and the k values calculated through Equations (1) and (2). A positive relationship is illustrated between Trs and k. k is shown to function as a coefficient rather than a constant when considered over multiple events. Figure 7c demonstrates the rapid Trs change with lower k values, and a trend towards a Trs of 1 (perfect trapping efficiency) as k values increase beyond 15,000 m/yr.

The field trial dataset created through this research provides the basis from which a matrix of k coefficients can be defined for this swale. It also provides a methodology to assess further blue-green network assets and ephemeral vegetated SuDS systems to define the long term, multiple event pollutant (sediment) decay rate and trapping efficiencies. Figures 6 and 7b emphasize the constraints of k constant proportionality assumptions in long term, multiple event analysis and the potential extension of k from constant to coefficient. k functionality as a coefficient is considered to be driven by the change in trapping efficiency resultant from multiple event influence on a single sediment release. Wong *et al.* (2002) [46] notes that k-C^* was designed for single event analysis within a conceptual modeling scenario. However, if extended and multi-event swale activity is to be considered for life cycle analysis, design improvement and provision of maintenance recommendations, modification of k from a constant to a coefficient following a positive Trs relationship curve towards Trs = 1 has been illustrated as an effective method of analysis.

7. Conclusions

REO have been effectively used to trace urban sediment pollution through an ephemeral blue-green SuDS asset (swale). Rare earth tracing methodology, previously employed in agricultural and river bank erosion monitoring, has been implemented in an urban environment. An effective trace concentration has been identified through field trails, demonstrating the use of 10 g/kg REO trace to sediment ratio to be effective in the field. REO tracing has been monitored in these field tests over 12 months, providing an extended, multiple runoff event sediment transport dataset through an established swale that defines the intra event and extended time period sediment movement. REO methodology defined within this paper is effective for ephemeral vegetated stormwater sediment tracing, providing clear unique sediment tracing signatures over an extended field period, without significant degradation or loss to the receiving environment.

Intra-event REO monitoring highlights the occurrence of a flow initiated concentration peak in the initial and subsequent flow events through a swale. Extended field monitoring has proven that pollutant (tagged sediment) residency within the swale exceeds six months, although there is a continued depletion of the quantity of sediment detained within the swale as a result of continued runoff events through the swale over time. Using a single tagged sediment release methodology, the resuspension, deposition and loss through conveyance of sediment in the swale is shown to change. Bed deposition and trapping efficiency are found to decrease progressively over multiple runoff events. Extrapolating from the field results, a tentative estimation of 25-year swale detention efficiency is calculated to be 3% of the initial inflow deposition.

This analysis considers use of the first order decay model to calculate long-term deposition. Field results show that while initial event sediment trapping or detention can be reflected through the k-C^* model, inclusion of subsequent events results illustrates the constraint in implementing k as a constant. Using the trapping efficiency equation defined by Deletic (2005) [49], the direct relationship between multiple event sediment concentration change and trapping efficiency has been proven. When multiple events are considered, k functions as a coefficient rather than a constant, supporting a positive change in trapping efficiency. The sediment trapping efficiency is influenced by event occurrence over time. This can be reflected through a decrease in k values over an extended, multiple runoff event analysis period of a single sediment release. While this field research illustrates a range of k values representative of this specific blue-green drainage assets within the local Scottish environment, the advancement of the first order decay model and definition of a novel and effective long term sediment SuDS analysis methodology have been demonstrated.

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Author Contributions

Deonie Allen, as the primary author and investigator of this research, undertook the field and laboratory analysis activities of this project; Valerie Olive completed all ICP OES analysis of prepared samples, without which this these findings would not be possible; Scott Arthur and Heather Haynes contributed valuable analysis, manuscript and internal review of his research.

Conflicts of Interest

The authors declare no conflict of interest.

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A Mapping of Tools for Informing Water Sensitive Urban Design Planning Decisions—Questions, Aspects and Context Sensitivity

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Abstract: Water Sensitive Urban Design (WSUD) poses new challenges for decision makers compared with traditional stormwater management, e.g., because WSUD offers a larger selection of measures and because many measures are multifunctional. These challenges have motivated the development of many decision support tools. This review shows that the tools differ in terms of the types of questions they can assist in answering. We identified three main groups: "How Much"-tools, "Where"-tools and "Which"-tools. The "How Much"-tools can further be grouped into tools quantifying hydraulic impacts, hydrologic impacts, water quality impacts, non-flow-related impacts and economic impacts. Additionally, the tools differ in terms of how many aspects of water they address, from those focused only on bio-physical aspects to those attempting to find the best WSUD based on multiple criteria. Finally, we suggest that variability among the tools can partly be explained by variability in local context including conditions such as type of existing stormwater systems, groundwater conditions and legislative frameworks.

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1. Introduction

The concept of Water Sensitive Urban Design has received increased interest in recent years. Some of the drivers include climate change and urbanization. These two factors, alone and combined, are causing an intensification of the adverse environmental impacts of traditional urban drainage systems, and are expected to increasingly do so in the future [1,2]. Therefore many scientists and other professionals are looking for other means of managing urban stormwater that fit into the urban environment and that lower the adverse impacts on the natural and built environment while maintaining the hygienic barriers between humans and polluted water [3,4].

A multitude of new terms for stormwater management has consequently emerged in the past decades including Sustainable Urban Drainage Systems (SUDS), Stormwater Best Management Practices (BMPs), Green Infrastructure (GI), Low Impact Development (LID), and Water Sensitive Urban Design (WSUD) [5]. We here use the term WSUD to describe any installation or intervention in the urban space that can manage stormwater (through detention, harvesting, infiltration, evaporation or transport) while contributing with some added functionality (such as recreational value, urban heat island mitigation, traffic control, *etc.*), although we acknowledge that multifunctionality is reflected to variable degrees in the different terms that are to some extent used interchangeably in the literature.

The practical experience with implementing WSUD is sparse in many regions, especially compared with the century long experience with traditional piped systems. Therefore many knowledge gaps need to be filled before large scale implementation of WSUD can be expected. Another factor that inhibits implementation of WSUD is the increased complexity compared with pipe-based systems, due to the fact that WSUD becomes an integrated part of the urban landscape rather than a distinct functionality hidden underground, a part that also takes up space (which is a valuable resource in dense cities). WSUD also has impacts on parts of the urban water cycle that are usually not considered important when assessing pipe-based systems, such as groundwater.

Not surprisingly, many tools have been developed to assist making decisions regarding the implementation of WSUD. In this context, we consider a decision support tool to be any software tool that can answer a question the decision maker asks, *i.e.*, provides information that is relevant for the decision in a manner that is clear and manageable. Hence, a decision support tool may focus on visualizing already existing information or on producing new information based on analysis of input information.

Several recent review papers have addressed the subject of WSUD and decision support. Zhou [6] offered a comparison of modelling approaches and a classification of other decision-aid tools, focusing on tools supporting the overall aim of assessing sustainability. Bach *et al.* [7] reviewed tools for modelling the broader scope of integrated urban water systems. Blumensaat *et al.* [8] compared and discussed a variety of protocols for water quality impact assessment. Jayassooriya and Ng [9] focused on tools for making cost-benefit analysis. All these reviews contribute valuable information, but none of them provide a complete overview of all the tools available to assist a decision maker considering implementing WSUD in an existing urban area.

The main aim of this paper is to provide an overview of the decision support tools available to decision makers when considering implementation of WSUD, illustrating the tools' capabilities and limitations. We provide this overview by two means:

- A categorization based on the main functionality of the tools, *i.e.*, what questions they can help answer,
- An evaluation of which aspects of the complex subject of "water" the different types of tools address.

Furthermore, we reflect on how the differences among tools correspond to different local contexts of decision making.

The paper is structured as follows. In Section 2, Methods, we describe our literature search strategy, the approach used for categorization, the theory of aspects of water and the assumption of context dependency. In Section 3, Results and Discussion, we present the functional categories identified, describe selected tools to exemplify the functionalities, show what aspects of water are addressed by the tools, offer some reflections on the context dependency of the tools, and finally discuss the limitations of our study and some perspectives for future work. In Section 4, Conclusions, we summarize our findings.

2. Methods

2.1. Literature Search

The tools reviewed were mainly found by searching for papers using the search engine and databases of Web of Science. The search phrase we used is illustrated in Figure 1. In addition to this search, some papers were found through reference lists of other papers and based on the authors' personal experience. In this paper, we generally use the term WSUD, but when citing other papers we use the term used by the original authors (such as SUDS or LID). In doing so, we assume a substantial overlap in the meanings conveyed by the different terms [5], accepting that some of the other terms may not necessarily include the multifunctionality implied by the term WSUD.

Urban	Stormwater Drainage Runoff	Model* Tool* Approach* Method* Framework*	Asses* Evaluat* Predict* Effect* Impact* Benefit* Multicriteria "Decision Support"	"Water Sensitive Urban Design" (WSUD) "Best Management Practice" (BMP) "Sustainable Urban Drainage Systems" (SUDS) "Low Impact Development" (LID) "Alternative techniques" "Source control" "Stormwater Control Measures" (SCM) "Green Infrastructure" (GI)

Figure 1. Illustration of the search phrase used in this study. The boxes are connected with "AND" while words within a box are connected with "OR". An asterisk (*) represents a wildcard.

2.2. Categorization Based on Questions Addressed by the Tools

We found that the tools are different from each other in many ways yet overlapping in other ways, and no set of categories could place them in mutually exclusive boxes. We reasoned that the primary concern of a decision maker when choosing a decision support tool would be whether this tool could assist in answering a set of questions that were identified as important to address for making a well-informed decision. We hence identified the most common questions that the tools we found may assist in answering, and designed a logical structure that sorts the different questions into groupings and sub-groupings.

2.3. Characterisation Based on Aspects of Water Valued by Stakeholders

Aspects of water is a methodology for mapping perceptions and values in urban stormwater management [10]. We used these aspects to characterize a selection of tools as another way of revealing their different focus areas. The aspects of water are a further development of the aspects theory developed by the Dutch philosopher Dooyewerd [10]. Dooyewerd used 15 aspects, ranked in order of importance, to describe the richness and multifacetedness of reality. The lower aspects obey the laws of nature, and may also be described as bio-physical aspects. The upper aspects affect how people deal with nature, and may also be described as human aspects. Valkman *et al.* [10] reduced the number of aspects to 12, including only three aspects in the bio-physical domain and omitting the highest aspect (pistic), see Table 1. They applied these aspects to water and suggested using them as a framework for drawing a complete picture of stormwater related issues, uncovering the different perspectives among stakeholders which are not water professionals. A slightly modified version of the aspects of water was later used by Fratini *et al.* [11] to analyse which issues were prioritized by different groups of stakeholders when interviewed about the same projects Their results indicate that water professionals need to learn how to extend their scope of aspects in order to create projects valued by a wider range of stakeholders.

Table 1. The 12 aspects of water used in our analysis, adapted from Valkman *et al.* [10].

Aspect	Essence	In Relation to Urban Water, with Specific Examples
Human Aspects		
12. Moral	Views concerning good treatment	Views concerning good water management • Safety, or the prevention of damage • Sustainability
11. Legal	Law	Regulations for water • Issue of permits for sewer overflow
10. Aesthetic	Beauty	The beauty of water • Reflecting water • Sunset by the sea
9. Economic	Way of saving	Economic water management • Do the costs of water projects weigh up against the benefits/values? • No wastage of groundwater
8. Social	Dealing with people	Meeting by the water • Discussion by the drinking water well in Africa • Resident evening concerning disconnection project
7. Linguistic	Symbolic significance	Writing about water • Poems • Water leaflet
6. Historical	Management by free forming	Intervention in the water system • Land reclamation • Delta works
5. Logical	Analytical distinction	Thinking about water • Thales: “Everything is water” • Organizing the water chain
4. Psychological	Perception	Water stimulates the senses • Water is wet • Delicious drinking water
Bio-Physical Aspects		
3. Biotic	Life processes	Water as the first condition for life • A person can survive for a maximum of 3 days without water • Fish live in water
2. Chemical	Matter	Water carries other substances • Water quality parameters
1. Physical	Uninterrupted extendedness, uniform movement	Water occupies space and water flows • a pond contains a quantity of water • water flows with gravity in unpressured pipes

2.4. Context Dependency

The large variation we found among the tools encouraged us to consider how the local context has shaped each tool by helping to answer the questions that were deemed urgent by the tool developers at a given time and place. We based our analysis on the findings of the literature search coupled with our research experience and practical experience with WSUD projects.

3. Results and Discussion

3.1. Categorization Based on Questions Addressed by the Tools

The structure that emerged from analysing what types of questions the different tools can help answer is shown in Table 2. On the highest level there are three types of questions: "How Much", *i.e.*, tools that provide quantitative answers, "Where", *i.e.*, tools that provide spatial answers, and "Which", *i.e.*, tools that help choose among options. The "How Much" category is further divided into tools that quantify different types of impacts: impacts related to hydraulics, *i.e.*, the flow of water through pipes and across surfaces, impacts related to hydrology, *i.e.*, the flow of water through the entire urban water cycle including groundwater and the atmosphere, impacts related to water quality, *i.e.*, the pollution carried with water, impacts that are not directly linked to the flow of water (such as aesthetics and recreation), and economic impacts.

When going through our search results we focused more on water quantity issues than water quality, and hence tools that focus on water quality were omitted. For examples of tools with specific focus on water quality issues, see e.g., [12–14]. We also omitted tools that focus on the broader issue of integrated urban water management, although some of these tools include functionality that is similar to the categories defined here; for examples of such tools see e.g., [15–18]. Finally, we also omitted process support tools, *i.e.*, tools that provide a framework for a decision making process rather than providing concrete information to be used in such a process; for examples of such tools, see e.g., [19–23].

Note that some tools that provide the same functionality (*i.e.*, answer the same questions) may do so with different methods, which may vary greatly in terms of input requirements, software requirements, expertise required of intended users and overall complexity. We have included a few different examples of tools in each category (listed in the rightmost column of Table 2) in order to describe some of this variability, but in order to preserve clarity, we have not attempted to cover all the variability in this review.

The following sections offer descriptions of examples of tools within each of the functional categories as well as some examples of tools that combine several types of functionality.

Table 2. The headings of this table present a structure for categorizing types of questions answered by WSUD decision support tools. The right column contains examples of tools that are further described in the following sections. The tools are grouped (indicated by the horizontal lines) according to which types of questions they may help in answering (indicated by the Xs).

How Much					Where	Which	Examples Covered in This Review
Water Quantity		Water Quality	Non-Flow related Impacts	Economic Impacts	Could WSUD Be Placed	WSUD Is Best	
Hydraulic	Hydrologic						
X	X						SWMM [24] MIKE URBAN [25]
X	X	X		X			MUSIC [26]
	X						Modflow IDD [27]
			X				LCA [28] Carbon footprint [29] Stakeholder preferences [30] Thorough ecosystem [31] Rapid ecosystem [32]
					X		Flext (DayWater) [33] SWMPT [34]
						X	BMP MCA [35] BMP DSM [36] Project choice [37] MCA/cost [38]
					X	X	SWITCH BMP DSS [39] SUDS potential [40]
X		X		X	X	X	SUSTAIN [41] UHRU [42]
X				X		X	LIDRA [43] STEPL [44]
X		X	X	X		X	MCA&CBA [45]
X			X	X		X	Flood Risk CBA [46]
X					X	X	SUDSLOC [47]

3.1.1. "How Much Water"-Tools

Hydraulic and hydrologic models generally answer interrelated questions such as "How Much Water, Where and When", by transforming rainfall data into surface and subsurface flows and storages, and routing these flows through representations of natural and technical systems such as pipes, basins, rivers and groundwater reservoirs. For a thorough review on different types of hydraulic and hydrologic models, please refer to Zoppou [48]. Elliot and Trowsdale [49] provided a thorough review of how well 10 of the more popular modelling tools enable representations of LID technologies such as swales and rainwater tanks. They documented that the models differ in terms of temporal and spatial resolution, whether they include a groundwater component, how many contaminants can be modelled, which LID devices are included explicitly, and whether they incorporate GIS (Geographical Information System) and other graphical interface features. They conclude that none of the models are intended for the full spectrum of uses that could be demanded in relation to LID, and that there is considerable scope for improving their capabilities. Seven years later, Fletcher *et al.* [50] noted that an important gap remains between models which allow assessment of hydraulic impacts at the network and catchment level, and models that represent source control measures well but are unable to predict their impact on catchment level, and that the integration of these scales remains a question for further research.

A recent example of applying a traditional stormwater model to a BMP implementation case is given by Petrucci *et al.* [24]. Their study included modelling the hydraulic impacts of implementing rainwater tanks in a Parisian suburb using SWMM5. As noted by Elliot and Trowsdale [49], rainwater tanks are not explicitly included in SWMM but can be modelled indirectly; in this case the rainwater tanks were represented in the model using the initial loss parameter, which was set to vary so that it represents the expected available space for storage as a function of filling by rainfall and emptying by evapotranspiration (representing usage of the stored water for garden watering).

An example of improving a traditional stormwater model to better represent WSUD is given by Roldin *et al.* [25]. They presented a methodology to estimate the impacts of extensive stormwater infiltration including a new module for dynamical modelling of soakaways in MIKE URBAN CS (formerly MOUSE). They applied the methodology to an urban catchment in Greater Copenhagen, studying three scenarios: baseline, full spatial potential implementation of soakaways and realistic implementation of soakaways limited by rising groundwater tables. The two latter scenarios were each modelled both using the dynamic soakaway module and a simplification where the impervious area routed to soakaways was completely disconnected from the stormwater model. Their results showed that simplifying the soakaways by removing the impervious areas from the model produced similar results to using the dynamic module; however, this was attributed to the relatively large volumes of the soakaways, resulting in few overflows to the sewer system.

By contrast to the stormwater models mentioned above (SWMM5 and MIKE URBAN), MUSIC was developed explicitly to represent WSUD elements and assess their impact on stormwater quality and hydrology [50]. An example application of MUSIC to compare the hydrological impacts of conventional stormwater management *versus* flow-regime management is given by Burns *et al.* [26]. They showed that catchments managed with focus on drainage efficiency or load reduction result in streamflows very different from an undeveloped catchment. In contrast, a management strategy focused on flow regime, using a combination of rainwater tanks and rain gardens, successfully reduced the frequency, magnitude and volume of stormwater runoff and likely contributed to restoration of baseflow to streams.

A few modelling applications focus explicitly on the hydrological impacts of WSUD on groundwater. For example, Jeppesen [27] developed a new package for simulating the two-way interaction between groundwater and infiltration-drainage devices in the groundwater modelling tool Modflow. His results showed that this interaction may have significant impact both on the groundwater table and on the functioning of the infiltration devices in areas with slow infiltrating soils. Efforts towards modelling WSUD interaction with groundwater in hydraulic urban drainage models are also underway [51].

3.1.2. "How Much of Non-Flow Related Impacts"-Tools

These tools answer less commonly asked questions regarding impacts of WSUD implementation, which may collectively be described as non-flow-related impacts. De Sousa *et al.* [28] applied a life cycle perspective to answer the question "which stormwater management strategy has the lowest greenhouse gas emissions". Strategy one used decentralized green infrastructure technologies, strategy two used a concrete detention tank from which water is subsequently pumped to a wastewater treatment plant, and strategy three used a concrete detention tank where the water is treated locally and then discharged to the river. A model set up using SWMM5 was used to show that all three strategies achieve the same reduction in combined sewer overflow from the sewer catchment to the Bronx River (NY, USA). The net greenhouse gas emissions of the green strategy over a period of 50 years were significantly lower than for the two grey strategies. Moore and Hunt [29] presented a complementary framework for predicting and comparing the carbon footprint of stormwater control measures and traditional conveyance-based system components.

Kaplowitz and Lupi [30] used choice experiment surveys to answer the question "what is the best BMP in terms of amenity value, as seen by the target group of such value". Their findings show that homeowners cared about the types and combinations of BMPs suggested for improving river water quality in their watershed, and unambiguously preferred management plans with high levels of stream bank naturalization and some wetlands.

Moore and Hunt [31] presented an assessment framework to help answer the question "which stormwater control measure provides most ecosystem services?". The framework suggested means of assessing some benefits that are often acknowledged but rarely quantified, including carbon sequestration, biodiversity and cultural services. Their results indicated that constructed wetlands demonstrated greater potential in all three categories than constructed ponds. Uzomah *et al.* [32] presented an expert tool designed to answer a similar question more rapidly, to be used in specific cases of retrofitting in urban areas.

3.1.3. "Where"-Tools

These tools generally answer the question "where can WSUDs be implemented" within a given area. One of the earlier tools of this type was FLEXT, developed within the framework of the European project DayWater [33]. The tool includes a knowledge base which stores information on the factors that affect a site's suitability for stormwater infiltration, such as soil permeability and distance to vulnerable structures such as building foundations. The knowledge base is open to the user and can be modified to reflect e.g., project specific needs or data availability. The knowledge base and associated rule operating system are integrated into the GIS software package GeoMedia, including a graphical user interface.

Lathrop *et al.* [34] provided an example of a GIS tool which is much simpler. It is an interactive web-based map query tool which allows for municipalities and counties to see location and basic details about existing stormwater basins. This information was in high demand by the practitioners surveyed, and was earlier only available in hardly accessible analogue archives.

3.1.4. "Which"-Tools

These tools answer the question "which is the best WSUD technology". Tools that provide this functionality alone are generally multicriteria tools, *i.e.*, tools that define multiple criteria to base the choice on and a method for weighting of these criteria. Some of these tools use global scores for the criteria, while other tools allow considering site specific parameters that affect the criteria scores.

An example of a tool from the first group (using global scores) is the multicriteria decision aid approach for WSUDs developed by Martin *et al.* [35], based on results from a national survey on performance of WSUDs in France. The tool allows the user to rank eight selected WSUDs using eight selected criteria with predefined scores by applying different sets of weights, reflecting the values of different stakeholder groups.

An example of a tool from the second group (considering site specific parameters) was reported by Scholz [36]. The tool is based on a matrix and an associated weighting system. On one axis the matrix includes 16 different BMPs such as wetlands, ponds and infiltration basins, and also allows assessing combinations of two BMPs. On the other axis the matrix includes 15 different criteria, some quantitative, such as catchment size (m^2) and area available for BMP (m^2), and some qualitative, such as runoff quality (must be either "good" or "average" depending on BMP intended) and land value (assessed by an expert on a scale from 1–5). Depending on the combination of BMP and criteria, a criterion becomes either "dominant", which means it is critical for whether this BMP is feasible, or "supplementary", which means it can be used to decide on the most appropriate BMP among those feasible for a site. The supplementary criteria were weighted by the author according to their relative importance for each BMP technique on a scale from 0–3. Thus, for each feasible BMP a cumulative sum can be calculated and compared to the highest possible sum for the given BMP. The ratio between the actual sum and the maximum possible sum can be used as a suitability index of the BMP for the given site.

Multicriteria tools in the context of WSUD can furthermore answer other questions than "which is the best WSUD". For example, the utility company Melbourne Water developed a multicriteria tool to answer the question "which is the best project proposal for the Living Rivers Stormwater Program" [37], while Moura *et al.* [52] developed a tool to answer the question "how well does an infiltration measure perform over time".

3.1.5. Combined Tools: "Where" and "Which"

A few tools answer both the question "where can WSUDs be implemented" and the question "which is the best WSUD at a given site". One example is the BMP-DSS tool developed within the European framework project SWITCH [39]. This tool extends the ability of identifying potential sites for implementation of BMPs (as seen in Flext [33]) by also integrating a multicriteria comparator approach that supports wider (and non-spatial) considerations. The multicriteria approach is implemented using a table that benchmarks the performance of BMPs against a list of criteria, subdivided into indicators and populated with default scores. The scores can be altered by the user, who can also assign weights to each indicator. The combined result is a ranking of the BMPs that are feasible at any identified BMP-suitable site.

A similar more recent GIS-based decision support tool for selecting stormwater disconnection opportunities was described by Moore *et al.* [40]. The tool was developed in the GIS package ArcView, using SQL rules to search for potential lots. However, not all steps were automated; e.g., retrofitting roofs with green roofs was based on firstly manual digitization of flat roofs using aerial photography, secondly GIS was used to select roofs larger than a predefined threshold, and finally engineering judgment was used to select buildings with likely suitable load bearing capacity. The output is in the form of multiple map layers indicating locations where each specific SUDS measure may be feasible, and in many cases more than one option may be feasible in any given location. In this case, the tool uses a general hierarchy to choose the most suitable option. The tool cannot quantify the expected impacts of the disconnections, but the authors present a methodology for transforming the results into inputs to a sewer model (InfoWorks CS) and modelling the SUDS measures indirectly, in line with the work of [24] and [25] referred to in the ”how much water”-tools section.

3.1.6. Combined Tools: “Which”, “How Much Water”, “How Much Money” and More

A few tools, or rather sets of tools, can assist in answering three or more of the types of questions we mapped, usually centred around the question of which WSUD strategy to choose. The difference between these tools and the more simple “which” tools is that these tools include functionality to assess the impacts of WSUD based on site specific input data so that (some of) the different criteria become case sensitive rather than relying on generic and fixed performance data. These tools often also include the economic costs of WSUD, and a few also consider the economic benefits of WSUD.

A notable example is the System for Urban Stormwater Treatment and Analysis Integration, SUSTAIN [41]. This is a public domain tool developed by the USEPA to assist in evaluating the optimal location, type and cost of BMPs. It includes: a framework manager developed in ESRIs ArcGIS; a tool to find suitable sites for BMPs (using ESRIs Spatial Analyst); the runoff and pollutant generation module and conveyance module of SWMM5; a module to compute flow and pollutant transport in BMPs; a module to compute the costs of implementing BMPs; and finally an optimization module to find the most cost-effective BMP strategies based on the user’s choice of evaluation criteria. The available evaluation criteria are hydraulic impacts (e.g., peak discharge) and water quality impacts (e.g., annual average pollutant load). Another tool that assists in finding cost-effective BMP strategies but based on a more simplified hydrological modelling approach was presented by Eric *et al.* [42]. A few other tools for supporting cost-effective decisions, e.g., LIDRA 2.0 [43] and STEPL [44], have simplified the calculation approach to a degree where they can be implemented online. Further examples of tools for assessing cost-efficiency, together with a more thorough review of the differences among them, can be found in a recent review by Jayasooriya and Ng [9].

Chow *et al.* [45] developed a tool that combines an economic assessment in the form of a cost-benefit analysis with a multicriteria approach. The cost-benefit analysis includes expected costs of WSUD implementation as well as expected monetary benefits. The monetary benefits are calculated based on quantitative indicators of performance, e.g., the potential increase in property value is a function of the expected change in the 100-year floodplain. The performance indicators are in turn calculated based on site specific input values combined with parameter values derived from guidelines and previous studies, e.g., the reduction in runoff volume resulting from permeable pavements is a function of the permeable surface (input), the annual precipitation (input) and the percentage of runoff retained (parameter value). The performance indicators are also summarized into four overarching criteria. The criteria scores and the monetary cost-benefit values are presented visually side by side to the decision maker, providing an overview of the multiple factors assessed in the framework.

Another example of a tool that includes monetary benefits of WSUD implementation was developed by Zhou *et al.* [46]. Their methodology focusses on flood risk mitigation and allows evaluation of both traditional stormwater management solutions and WSUD solutions in terms of hydraulic performance under extreme precipitation by using 1D-2D models, and quantification of both the economic costs and benefits of the solutions. Another example of a tool that enables evaluating the flood mitigation impact of SUDS under rare rainfall events is SUDSLOC [47]; here, the hydraulic 1D–2D functionality is combined with a multicriteria tool.

3.2. Characterization Based on Aspects of Water Valued by Stakeholders

Table 3 shows our evaluation of what aspects of water are addressed by the tools that were included in Table 2. Note that tools within the same group (as indicated by the horizontal separation lines), *i.e.*, tools that according to the logic of Table 2 could help answer the same type of questions, do not necessarily address the same aspects. In other words, the aspects of water method reveals some nuances that were not clear from the functional categorization.

All tools are considered to address the logical aspect, in the sense that they have a logical structure, a logical step-wise application and are based on logical cause-and-effect-relations; the logical aspect is in fact inherent to our definition of a decision support tool and thus a precondition for being included in this study.

All but two of the tools are considered to address the physical aspect in the sense that they address the impacts of WSUD on the flow of stormwater. The exceptions are the tool that simply displays GIS-data [34] and the tool that reveals stakeholders' preferences [30] (assumed that these preferences are not affected by the options' hydraulic performance since the stakeholders were not informed of these).

Table 3. Aspects of water addressed by the tools reviewed in this paper. The tools are kept in the same horizontal groups as in Table 2 (reflecting which types of questions they aim to help answer).

Tool	Bio-Physical Aspects			Human Aspects								
	Physical	**Chemical**	**Biotic**	**Psychological**	**Logical**	**Historical**	**Linguistic**	**Social**	**Economic**	**Aesthetic**	**Legal**	**Moral**
SWMM [24]	+				+							
MIKE URBAN [25]	+				+							
MUSIC [26]	+	+			+				+			
Modflow IDD [27]	+				+							
LCA [28]	+	+	+		+							+
Carbon footprint [29]	+				+							
Stakeholder preferences [30]				+	+			+		+		
Thorough ecosystem [31]	+	+	+		+	+		+			+	
Rapid ecosystem [32]	+	+	+	+	+			+	+	+		
Flext (DayWater) [33]	+				+							
SWMPT [34]					+							
BMP MCA [35]	+	+			+			+	+			+
BMP DSM [36]	+	+			+				+		+	
Project choice [37]	+	+			+			+	+		+	
MCA/cost [38]	+	+	+		+			+	+	+		
SWITCH BMP DSS [39]	+	+			+			+	+	+	+	
SUDS potential [40]	+				+						+	
SUSTAIN [41]	+	+			+				+			
UHRU [42]	+				+				+			
LIDRA [43]	+				+				+			
STEPL [44]	+				+				+			
MCA&CBA [45]	+	+	+		+				+			
Flood Risk CBA [46]	+				+				+	+		+
SUDSLOC [47]	+	+			+			+	+	+	+	+

Hydraulic models, exemplified by a SWMM application [24] and a MIKE URBAN application [25], as well as hydrologic models, exemplified by [27], address only one aspect besides the logical: the physical. This reflects the traditional focus of civil engineers on predicting the hydraulic performance of piped stormwater systems, and indicates the limitations of this approach when addressing WSUD performance, considering that WSUD per definition aims at providing multiple functions extending beyond drainage. By contrast, MUSIC [26], which was developed specifically for WSUD applications, addresses also the chemical and economic aspects, yet still lacks other essential aspects such as biotic and social.

The tools that focus on non-flow related aspects [28–32] and the multicriteria tools [35–38] generally address more aspects than any other group of tools. Another tool that addresses many aspects is the cost-benefit flood risk framework [46], which incorporates a multicriteria tool. The aspects included by many of these tools and few of the other tools are the biotic, the social and the legal. The spectrum of aspects addressed by each tool generally reflects the emphasis of the approach used, *i.e.*, the life-cycle cost tool addresses aspects relevant for the environment and the cost-benefit tool addresses aspects relevant for the economy.

None of the tools address all aspects, indicating that none of the tools can be used as the sole input to a decision process that aims to be complete. The linguistic aspect is not addressed by any of the tools, while the historical aspect is addressed by only one tool and the psychological by only two tools. Other aspects that are rarely considered are the biotic, aesthetic, legal and moral.

3.3. The Significance of Context

The variation among the tools available for decision making suggests that some parameters affect decision making in some regions while other parameters are more important in other regions. In the following, we describe how some parameters that vary among regions seem to have affected the design of the functionality of the investigated tools.

3.3.1. Combined or Separate Sewer Systems

In combined sewer systems, which are generally predominant in old city centres in Europe, the pollution issues associated with stormwater runoff are generally considered under control since it is largely treated at the wastewater treatments plants. Thus, reducing hydraulic load on the system is a main driver for implementing WSUD, and attention is focused on studying the hydraulic impacts of WSUD on the existing sewer system, using hydraulic modelling tools (see e.g., [24,25]). By contrast, in separate systems, which are generally dominant in e.g., the US and Australia, stormwater runoff is traditionally discharged into surface waters without any treatment. Thus, reducing the pollution carried by stormwater is a main driver for implementing WSUD and attention is focused on investigating and documenting the pollution control impact of WSUD by use of tools that explicitly incorporate water quality impacts (see e.g., [13,41]).

3.3.2. Groundwater Conditions

In e.g., Denmark, the groundwater level is generally close to the surface and represents a threat to building foundations as well as a nuisance in the form of infiltration into drains and sewer pipes. Therefore, groundwater presents limitations to the desired extent of infiltration based WSUD. In regions where groundwater levels are generally at a safe distance to the surface and rising groundwater levels are less of a worry, increased groundwater recharge is seen as a positive impact, contributing to improved baseflow in streams and enhanced resource for abstraction (see e.g., [26]). This could partly explain why dedicated tools for modelling the two-way interactions between infiltration based WSUD elements and groundwater are being developed in Denmark (see [27,51]).

3.3.3. Legislative and Economical Framing

Many tools which attempt to calculate cost-efficiency of management strategies emerged in the US (see e.g., [41–44]). These tools focus on a limited set of impacts reflecting WSUD's ability to meet regulatory demands for reduction of pollution and hydraulic loads. Other tools, mainly originating in Europe, show that other benefits of WSUD, such as recreation and aesthetics, can be translated into monetary values and tip the comparison between stormwater management scenarios in favor of WSUD (see e.g., [45,46]). Thus, an economic assessment depends on the framing of the economic system, whether it is the larger socio-economic system or the budget of a single institution made responsible for improving stormwater system performance.

3.3.4. Drinking Water Supply

In some areas, such as southern Europe and Australia, there are severe threats to drinking water resources. Saving water is therefore a main driver for rainwater harvesting, and assessing the volume of water that can be harvested and used is of great interest (see e.g., [53,54]). By contrast, in regions where drinking water resources are abundant, such as northern Europe, the option of substituting drinking water with harvested rainwater is considered more of a "luxury", with many active opponents (warning against risks of contamination and unnecessarily high costs) (see e.g., [24,55]). Thus, the potential of replacing potable water with harvested water is not as often considered in WSUD assessments in water-abundant regions as in water-scarce regions.

3.4. Limitations of the Study

While the Web of Science search engine and database is a credible source for scientific literature, this database also reflects the varying levels of attention that the scientific literature and science *per se* devote to different aspects of reality. Besides the limitations of the Web of Science database, we further limited the search results by our choice of search phrase. The search phrase is comprised of terms used in the field of urban drainage management and thus implicitly limits the results to papers published mainly in technical journals. The tools included in this review have a high representation of the physical, chemical, logical and economic aspects and a low representation of other aspects such as historical, linguistic and moral. We argue that this may reflect a general tendency in the scientific literature, or at least in the technical literature devoted to urban water management.

Our results may not correctly reflect the representation of aspects in tools used in reality, since not all tools used by practitioners are reported in the scientific literature. Given the history of development of urban drainage management (dominated by technocrats), we feel it is unlikely that the situation in real life shows significantly different trends from the one we found in the literature. However, other professionals are gaining momentum in relation to urban water management and this is likely to influence decision making in the future.

The issue of representation of aspects is further complicated by the nature of what we have termed "process tools": guidelines, frameworks *etc.* that aim to support the process of decision making regarding WSUD. One example is the Three Points Approach [11], originally developed to facilitate decision making processes in urban flood risk management. It defines three decision domains for urban stormwater management, which correspond to three domains in the probability distribution of rainfall. In this sense, the tool directly addresses only the physical aspect of water. However, when the concept is used in a decision making process involving multiple stakeholders, it provides a holistic thinking system and improves communication among stakeholders from different backgrounds, and in this process it ensures that multiple aspects of water are addressed. Thus, if we had included "process tools" in our study, we may have found a broader distribution of aspects addressed by tools.

Our categorization based on questions addressed by the tools provides a useful overview of the tools available, using a structure that is simple and clear. The assessment of which aspects of water are addressed by the tools sheds new light on how holistic an answer any tool can provide. Yet, these two methods ignore other important qualities of the different tools that would be important to take into account when choosing which tool to use, such as input data requirements, necessary user expertise *etc.* For more information on this, the reader is referred to other more technical reviews such as [9,49].

3.5. Perspectives and Recommendations

The discussion presented in Section 3.3. on the significance of context may be just the tip of the iceberg, *i.e.*, there are probably many more local factors that have an even greater and more profound impact on shaping tools than what we have pointed at. This may be inevitable and is not necessarily undesirable. However, we believe that it is important for tool developers, tool users and decision makers to be aware of these relations between context and tool. When using a tool within the context it was developed for, users will be operating based on implicit assumptions and traditions that may not be considered valid by all stakeholders. When using a tool outside of its development context, tool users may experience difficulties with applying the tool, and decision makers may experience difficulties in interpreting the results, sometimes without being able to pin-point what causes these difficulties. Future socio-technical research may help identifying the types of assumptions and dogmas that are typically embedded in tools, and how they can be articulated and addressed.

The lack of a single tool that addresses all aspects of water raises many questions, e.g., is it possible to include all aspects of water in a "hard" (software-based) tool? Would that be a useful tool or would it become too complex or too simplified? Could a process tool be better suited to ensure more holistic decision making? Is there a single process tool that fits all decision processes or are the processes too diverse? How can process tools and quantitative tools support each other? Again, more socio-technical research would be required to properly address these questions; we believe the answers would be valuable to practitioners seeking to improve decisions regarding planning of WSUD.

4. Conclusions

A categorization of tools for supporting decisions regarding WSUD based on questions addressed by the tools showed that the tools can be divided into three main groups: those that can assist in answering the question "How Much", those that can assist in answering the question "Where can/should WSUD be placed", and those that can assist in answering the question "Which WSUD is the best". The "How Much" tools can further be subdivided depending on what type impacts they quantify: water quantity impacts (hydraulic or hydrological), water quality impacts, non-flow related impacts, or economic impacts. Some tools address various combinations of these questions, while none of them address all the questions.

A characterization based on aspects of water addressed by the tools revealed that none of the tools address all aspects that can be relevant for informing WSUD planning decisions, and many commonly used tools such as hydraulic models address only very few aspects.

The two methods we applied were complementary in describing variations among tools, yet they were not exhaustive in the sense that there are additional variations that are not captured in this analysis. Also, the framing of the literature search entails some limitations on the completeness of this review.

We noted that there are some clear influences of local context on the development of tools, and that this has implications for the transparency of tools and the potential for using them outside their original context. There seems to be room for a more thorough socio-technical analysis of this question, and a need for more awareness among tool developers and users on the significance of context to WSUD planning decisions.

The fact that none of the reviewed tools addresses the full spectrum of aspects of water indicates a challenge for decision makers who rely on decision support tools. We propose to further investigate how the use of both "soft" and "hard" tools can assist in making more inclusive decisions.

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Conflicts of Interest

The authors declare no conflict of interest.

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Green Infrastructure Design for Stormwater Runoff and Water Quality: Empirical Evidence from Large Watershed-Scale Community Developments

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Abstract: Green infrastructure (GI) design is advocated as a new paradigm for stormwater management, whereas current knowledge of GI design is mostly based on isolated design strategies used at small-scale sites. This study presents empirical findings from two watershed-scale community projects (89.4 km^2 and 55.7 km^2) in suburban Houston, Texas. The GI development integrates a suite of on-site, infiltration-based stormwater management designs, and an adjacent community development follows conventional drainage design. Parcel data were used to estimate the site impervious cover area. Observed streamflow and water quality data (*i.e.*, NO_3-N, NH_3-N, and TP) were correlated with the site imperviousness. Results show that, as of 2009, the impervious cover percentage in the GI site (32.3%) is more than twice that of the conventional site (13.7%). However, the GI site's precipitation-streamflow ratio maintains a steady, low range, whereas this ratio fluctuates substantially in the conventional site, suggesting a "flashy" stream condition. Furthermore, in the conventional site, annual nutrient loadings are significantly correlated with its impervious cover percentage ($p < 0.01$), whereas in the GI site there is little correlation. The study concludes that integrated GI design can be effective in stormwater runoff reduction and water quality enhancement at watershed-scale community development.

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1. Introduction

Mitigating the development impact on stream hydrology and stormwater quality has been extensively discussed [1–6]. Excessive runoff leads to costly flooding events and the export of nitrogen and phosphorous to streams, which become concerns of environmental protection agencies and health agencies [7,8]. The main culprit behind these events is the elevated level of impervious cover as a result of community development. Impervious covers such as rooftops, streets, and parking lots alter the natural hydrological cycle and increase the pollutant loadings [9–11]. During the past three decades, imperviousness continues to be the single most important variable to define the amount of urban development, and a considerable number of studies have suggested a definite relationship between impervious areas and watershed stream health [12–14]. Watershed degradation generally is a result of a gradual process when impervious cover increases. However, many studies suggest that when impervious cover exceeds a range from 10% to 20%–25%, negative impacts are more prominent, including severe erosion, high nutrient flux, diminished stream species diversity, alteration of stream morphology, and an overall degradation of the aquatic system health [9,15–17].

In community development, the conventional drainage solution (curb-and-gutter, drop inlet, and underground piping) transfers stormwater faster than the natural hydrological cycle and may contribute to downstream flooding [18–20]. Common mitigation measures such as detention basins focus on peak discharge reduction and present limited success in runoff volume reduction or water quality improvement [2,21]. Moreover, if the basin is located inappropriately, it may aggravate flooding [22–24]. The U.S. Environmental Protection Agency (USEPA) advocates retrofitting the conventional stormwater system toward using low-impact development (LID) and green infrastructure (GI) design [7,25,26]. GI design typically encompasses open space, parks, green roofs, bioretention and constructed wetlands, decentralized water management (e.g., rainwater harvesting), protection of riparian areas, and various hybrids of pervious surfacing options [26,27]. GI design is based on ecological engineering principles [28,29], and a number of studies have suggested that integrated GI designs can be more effective than single-design strategies [30–32]. While the main focus of conventional drainage solution is peak discharge reduction, GI design aims at restoring the predevelopment flow regimes, such as reduction of runoff volume, enhancement of stormwater quality, and maintenance of base flows [33–35].

To achieve the above performance benefits, GI design treats runoff close to where it is generated. For instance, runoff is detained or infiltrated onto permeable surfaces on-site. As a result, the amount of effective impervious area (EIA) that directly contributes to runoff is reduced. EIA is a subset of the commonly used term "total impervious area" (TIA), which is often used to define the extent of community development. TIA is the sum of all noninfiltrating surfaces. EIA, or directly connected impervious area, includes only those impervious areas that drain into a piped storm sewer and further discharge into a surface-water body (e.g., parking lot runoff goes directly to a stormwater drain) [36,37]. Recent studies suggest that EIA increases when the connectivity of TIA increases and that development patterns can be better indicators than TIA alone in estimating stormwater runoff and pollutant exports [38,39].

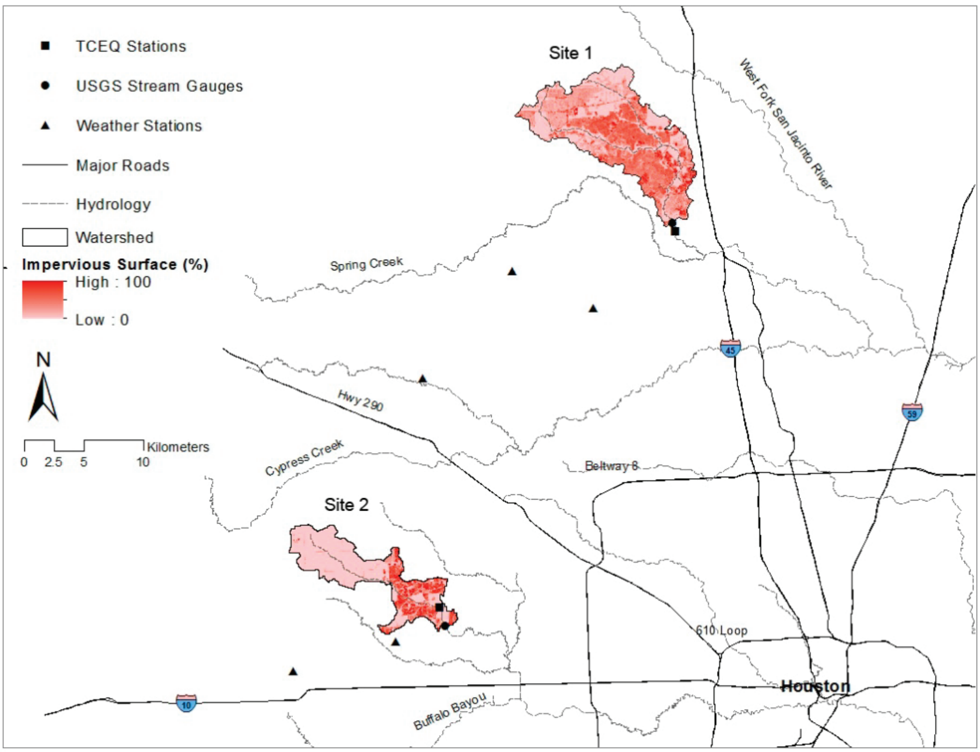
Although community development inevitably increases the TIA, GI design can be effective in reducing the EIA and runoff volume [35,40–42]. However, most current knowledge of GI design is based on isolated design strategies used at small-scale sites. Few studies have fully measured the effectiveness of integrated GI design that encompasses entire watersheds [43,44]. This current study presents empirical findings from two watershed-scale community projects in suburban Houston, Texas. One of the projects, The Woodlands, is a renowned large-scale community development. It is a precursor of the USEPA's GI design initiative: a series of integrated GI design strategies were used to meet flood control and stormwater quality goals [44–47]. Several recent studies have demonstrated the effectiveness of The Woodlands GI design on flood mitigation [31,32,46]. In addition to stormwater quantity, this current study further assessed water quality performance of The Woodlands' GI design with built communities that use the conventional drainage design.

2. Materials and Methods

2.1. Study Site

The test-bed watersheds are Panther Creek watershed (The Woodlands, GI design) and Bear Creek watershed (west Houston, conventional drainage design). These two watersheds have different stormwater infrastructures, development densities and patterns, and levels of impervious surface cover (Figure 1, Table 1). Both watersheds belong to the northern humid gulf coastal prairies of Texas and present similar land use land cover conditions before development.

Figure 1. Study sites Panther Creek watershed (Site 1 The Woodlands green infrastructure development) and Bear Creek watershed (Site 2 conventional development) in Texas. USGS, US Geological Survey. TCEQ, Texas Commission on Environmental Quality.



Notes: (1) At both sites the USGS and TCEQ gauge stations are on the main stream channel; (2) At Site 1, the TCEQ gauge station (No. 16628) is 55 m downstream of the USGS gauge station (No. 08068450). For graphic presentation purposes, the distance between these two stations is shown as larger than 55 m. There is a 1.01 km^2 (250-acre) recreation lake (built in 1985) 2332 m upstream of the TCEQ gauge station; (3) At Site 2, the TCEQ gauge station (No. 17484) is 1,656 m upstream of the USGS gauge station (No. 08072730).

Table 1. Study sites and respective watersheds.

Watershed	Drainage area (km^2)	Development start date	Population	Household number	% Impervious cover (2009)
1. Panther Creek (Woodlands, GI)	89.4	1974	66,143	24,655	32.3
2. Bear Creek (comparative)	55.7	1976	33,763	9,559	13.7

Notes: Watersheds are defined by the U.S. Geological Survey gauging stations: No. 08068450 (Panther) and No. 08072730 (Bear). Slopes in these two watersheds are less than 1%. Population and household information is based on 2010 U.S. Census Block data.

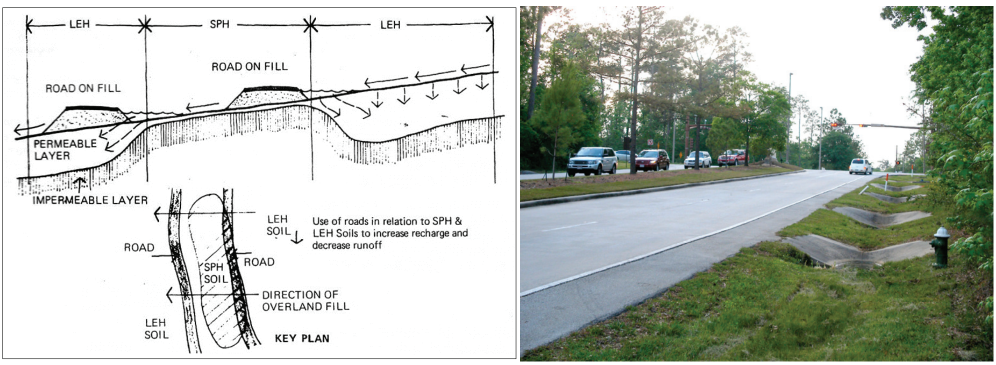
Development started around the same period in these two watersheds in the 1970s, whereas stormwater management methods differ. The Woodlands has been well-managed as a planned community from its inception [45,47]. Ecological planner Ian McHarg laid out a suite of decentralized, infiltration-based drainage designs to reduce runoff volume and improve water quality [48–51]. McHarg's GI design was ahead of his time in that most Houston subdivision communities have been adopting the conventional drainage practices. One of McHarg's important land planning strategies was to determine building densities and land use based on soil permeability. This is achieved by preserving land with high soil permeability as open space and land with low soil permeability for development. Hence, runoff is infiltrated in close proximity to where it is generated [48]. In addition to the extensive infiltration-based drainage designs, other development strategies also help minimize the TIA and the EIA. Typical streets in The Woodlands are 5 to 8 ft (on average 10%) narrower than Houston subdivision standards for road width [49]. Open surface drainage channels were used to detain runoff, and curb-and-gutter drainage was avoided (Figure 2) ([48], p. 10).

Figure 2. (**a**) A typical view in The Woodlands: preserving the original vegetation, minimizing turfgrass areas, and using open surface drainage; (**b**) A typical collector street in The Woodlands: stormwater drains to the vegetated medium for treatment.

(**a**) (**b**)

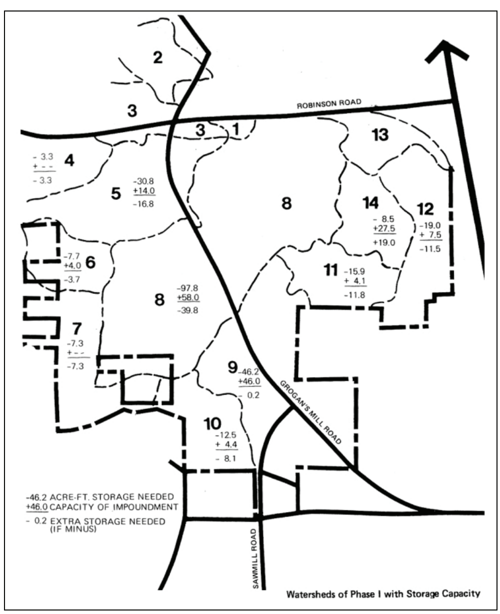
In collector streets, runoff is detained and treated in the vegetated street medium for better water quality (see Figure 2). Check dams were used to retard runoff and further soak it (Figure 3). Porous pavements were used in the commercial district of the first subdivision village and other locales [52]. Wetlands are protected for water quality treatment and to facilitate ecosystem services [53–55]. Modeling analyses projected the after-built runoff scenarios and ascertained that runoff is detained as close as possible to where it is generated (Figure 4).

Figure 3. (**a**) Open drainage design guideline which promotes impoundment on permeable soils. Check dams retard runoff and increase infiltration. LEH: medium to well-drained soil; SPH (Splendora): poorly drained soil ([50], p. 31) (Image courtesy: WRT); (**b**) Open surface drainage along collector streets in The Woodlands.



(**a**) (**b**)

Figure 4. Modeling analysis on runoff storage in Phase I development (8 km^2), with no excessive runoff allowed ([50], p. 9) (Image courtesy: WRT).



Historical extreme storm events attested to the success of McHarg's integrated GI design. The Woodlands survived 100-year storms in 1979 and 1994 with little property damage, while several adjacent communities and Houston (50 km to the south) were severely flooded [56,57]. In addition, the first phase development alone would save $14 million in construction costs compared with the conventional drainage method [45]. However, residents did not appreciate the aesthetics of the GI design. Market studies showed that most residents preferred visually appealing conventional drainage design (e.g., curb-and-gutter street). The rustic appearance of natural vegetation and unmaintained understory are contrary to average American's preference for a manicured lawn (see Figure 2a) [54,58]. As a result, starting around 1985, a hybrid approach was used in the later phases of development—conventional underground pipe drainage was introduced in subdivisions [59]. After 1997, McHarg's approach was largely abandoned when The Woodlands was sold to a different developer [46,58].

Bear Creek watershed is located in the fast growing west Houston region. Population in this region has surpassed 1 million since 1999 [60]. Over 34% of the residential community development in Greater Houston is projected to occur here, given the fact that Houston is currently one of the most rapidly expanding regions in the nation [61]. Bear Creek watershed presents typical subdivision developments: cookie-cutter lot layout, turfgrass-dominated landscaping, and curb-and-gutter and underground pipe drainage (Figure 5). Bear Creek watershed is part of residential development areas, designated by the West Houston Association (WHA) [60] and City of Houston General Plan [62]. There are significant flooding and water quality concerns in this region [63].

Figure 5. Typical neighborhood views in comparative Houston communities in the Bear Creek watershed (less consideration of preserving vegetation and curb-and-gutter conventional drainage).

2.2. Data

2.2.1. Development Data

In land use planning, three data sources and methods are generally used to capture the impervious cover area: (1) use parcel data to quantify the impervious area [36,64]; (2) classify Landsat remote sensing imagery to extract the impervious area [3,65,66]; and (3) digitize high-resolution aerial photographs to delineate the impervious area [66,67]. This study used the first data source to quantify impervious cover area. Parcel data provide the parcel boundary and location, parcel area, building type, year built, and building square footage. Road information was obtained from the Texas Transportation Institute [68].

2.2.2. Soil Data

Soil infiltration capacity can be assessed through examining the area of hydrologic soil groups in the two watersheds. The soil dataset used was the 1:24,000 scale Soil Survey Geographic (SSURGO) database developed by the Natural Resources Conservation Service (NRCS) [69]. The U.S. Department of Agriculture (USDA) [70] defines four hydrological oil groups (A, B, C, and D) based on soil infiltration rates. A soils are sandy and loamy sand soils; B soils are sandy loam and loam soils; C soils are silt loam and sandy clay loam soils; and D soils are clay loam, silty clay loam, and clay soils. A soils have the highest infiltration rate, B and C soils have moderate infiltration rates, and D soils have the lowest infiltration rate.

2.2.3. Precipitation Data

Historical precipitation data were obtained from the National Climatic Data Center (NCDC) [71]. The Thiessen polygon method [72] was used to estimate precipitation for each watershed. Three weather stations (COOPID No. 411956, No. 419076, and No. 414300) were identified for Panther Creek watershed, and three other stations (COOPID No. 412206, No. 414704, and No. 414313) are used for Bear Creek watershed (see Figure 1). The area weighted percentage of each station was used to calculate the composite precipitation value.

2.2.4. Streamflow and Water Quality Data

Streamflow and water quality data of 2002–2009 were used for comparison. Streamflow data were collected from the U.S. Geological Survey (USGS) gauge stations No. 08068450 and No. 08072730 [73], at the watershed outlets (see Figure 1). Water quality data were obtained from the Texas Commission on Environmental Quality (TCEQ) [74] stations No. 16628 and No. 17484. The TCEQ also collects streamflow data when water quality data are collected but with some data gaps. Because the TCEQ monitoring stations are placed close to the USGS gauge stations (see Figure 1), the USGS streamflow data were used for consistency. Since 2000, the TCEQ has been collecting 5 to 12 water quality samples each year for each station. Water-quality samples were consistently obtained on the same day at these two stations. The date of sampling during a particular month was irregular, and the samples may not necessarily have been taken after a rainfall event. Nutrient-related parameters that show consistent records from these two stations were analyzed, including nitrate-nitrogen (NO_3-N), ammonia nitrogen (NH_3-N), and total phosphorous (TP). If in either site there were fewer than six samples for a year, that year was excluded from the analysis.

2.3. Analysis

Three sets of analyses were conducted to compare the impacts of different drainage methods on flow regime and water quality. The first set of analyses assessed development extent and soil conditions to provide background conditions of stormwater quantity and quality comparisons. Geographic Information System (GIS) was used to analyze the parcel data. Building footprint and other impervious cover areas were calculated and sorted by year built, which provides the state of development in the watershed each year. Road surface area was estimated by multiplying the road length by the average width of the roads in the watershed [64]. A majority of the developments in this study have sidewalks on both sides of the road. Hence, the road length was doubled for the sidewalk length. Estimation was also made of the driveway impervious area. Previous studies have used the number of garage stalls multiplied by the average width (3 m) of the driveway [75,76]. However, parcel data do not provide driveway information. The Woodlands Residential Development Standards specified the front yard setback distance: "a garage or garage addition must be set back at least 16 feet (4.88 m) from the side property line" ([77], Section 2.1, p. 14). This setback distance was multiplied by the width of a two-stall garage (6 m) to approximate the driveway impervious area in The Woodlands, calculated by Equation (1):

$$\text{Driveway area (m}^2\text{)} = \text{Front yard setback (m)} \times 3\text{ m} \times \text{Number of garage stalls} \tag{1}$$

Then, this driveway area was multiplied by the total number of parcels in the watershed to estimate the total driveway areas. Likewise, estimation of driveway area was made for Site 2 (Bear Creek watershed), based on the 20-ft (6.1-m) garage setback distance for local streets [78]. GIS was also used to analyze the percentages of different hydrologic soil groups, which will provide insights into the overall stormwater infiltration capacities of the study sites. Soil condition is of particular importance to The Woodlands because McHarg's unique development concept is to preserve high-infiltration soils for stormwater management.

The second set of analyses examined the relationships of watershed streamflow volume and streamflow-precipitation ratio with impervious cover percentage. Streamflow depths and streamflow-precipitation ratios were examined for water years 2002–2009 for each watershed. A water year, according to the USGS definition, is from October of the preceding year to September of the current year (*i.e.*, water year 2002 = 1 October 2001 to 30 September 2002).

Streamflow-precipitation ratio (as %) for each year were calculated by dividing annual streamflow (m) by annual precipitation (m), and multiplying by 100. Annual streamflow depth (m) is calculated by dividing the total streamflow volume (m^3) by the watershed area (m^2), using Equation (2) below:

$$H = \frac{Q_i \times t}{A} \tag{2}$$

where H is the watershed annual streamflow depth (m); Q_i is the annual mean flow at year i ($m^3 \cdot s^{-1}$); t is a constant, 31,536,000 s, the total number of seconds in a year; and A (m^2) is the watershed area. This method assumes a uniform depth of precipitation falling onto the watershed; therefore, flow volume is standardized and becomes comparable.

The third set of analyses examined annual nutrient export. This study used the annual flow-weighted method developed by Littlewood [79,80] to calculate nutrient loadings for NO_3-N, NH_3-N, and TP, according to Equation (3):

$$Flux = KV \frac{\sum_{i=1}^{n} C_i Q_i}{\sum_{i=1}^{n} Q_i} \tag{3}$$

where K is the conversion factor to adjust for units and intervals of sampling; V is the annual accumulative flow (calculated from continuous data) ($m^3 \cdot s^{-1}$); C_i is the concentration measured at the day and time of the ith sample ($mg \cdot L^{-1}$); and Q_i is the flow rate measured at the day and time of the ith sample ($mg \cdot L^{-1}$).

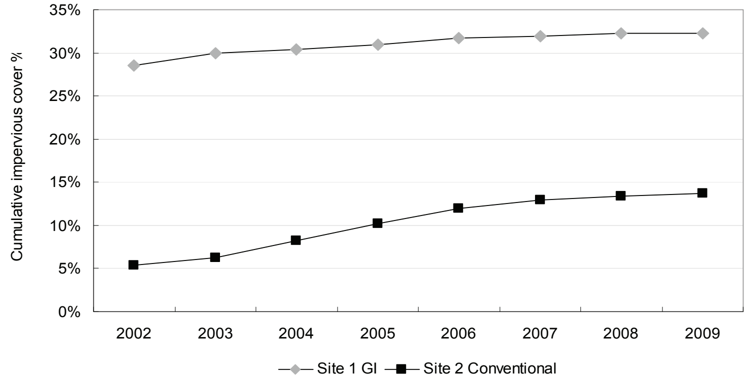
Regression analysis was conducted for each watershed, with the independent variable being watershed impervious coverage (%), and pollutant loading being the dependent variables. Each point on the graphs therefore represents a year. Regression significance testing, R^2 calculations, and parameter estimates were performed with the SPSS statistical package.

3. Results

3.1. Impervious Cover

Figure 6 shows the accumulative impervious cover percentage of the two sites with development from 2002 to 2009. It is evident that Site 1 (GI) shows a higher impervious cover percentage than Site 2 (conventional) across the study period. As of 2009, the percentage of impervious cover in Site 1 (GI, 32.3%) is more than twice that of the Site 2 (conventional, 13.7%).

Figure 6. Accumulative percentages of impervious cover area of Site 1 (Panther Creek watershed, The Woodlands green infrastructure development) and Site 2 (Bear Creek watershed, conventional development), 2002–2009.



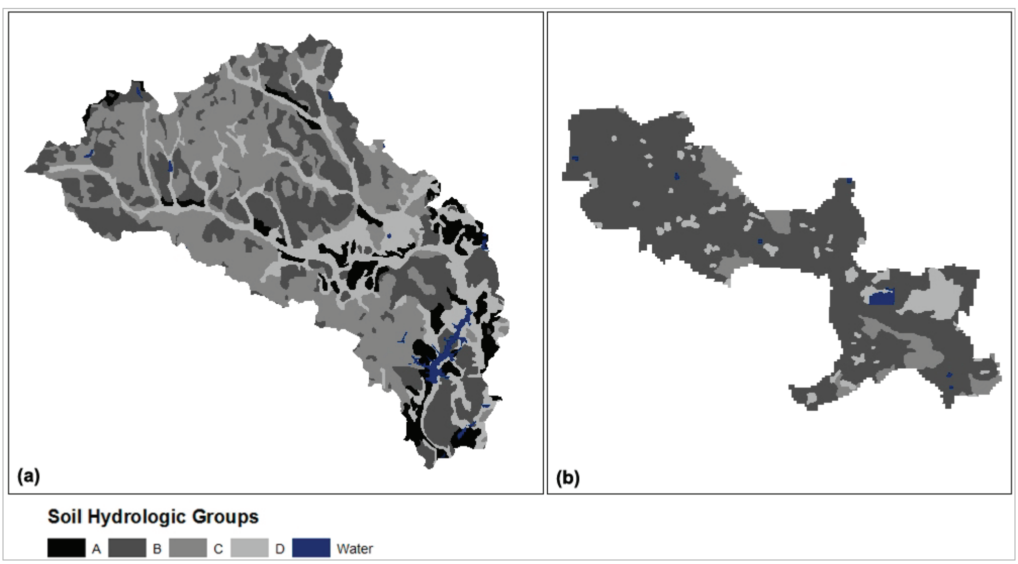
3.2. Hydrologic Soil Group Distribution

Table 2 and Figure 7 show the area distribution of four hydrologic soil groups in Sites 1 and 2. These four soil groups were further divided into two groups: A & B (sandy and loam), and C & D (silt and clay), in order to show the overall stormwater infiltration capacity (e.g., good *versus* poor). It is evident that stormwater infiltration capacity of Site 1 (GI) is lower than that of Site 2 (conventional), because Site 1 has a much lower percentage of A & B soils (38.7% *versus* 80.2% in Site 2).

Table 2. Hydrologic soil groups in Site 1 (Panther Creek watershed, The Woodlands green infrastructure development) and Site 2 (Bear Creek watershed, conventional development).

Hydrologic soil groups	Site 1 (GI)	Site 2 (conventional)
A	8.3%	0
B	30.4%	80.2%
C	40.1%	9.8%
D	19.9%	9.0%
Water	1.2%	0.9%

Figure 7. Area distribution of four hydrologic soil groups and water surface in Site 1 (Panther Creek watershed, The Woodlands green infrastructure development) and Site 2 (Bear Creek watershed, conventional development).



3.3. Precipitation and Streamflow

Figure 8 shows the annual precipitation in Sites 1 and 2 (approximately 45 km from each other). Figures 9 and 10 show the annual precipitation depths (m) and the annual streamflow-precipitation ratios (%) at Sites 1 and 2, respectively. The average precipitation of 2002–2009 at Site 1 (GI, 1.48 m) is 15.3% higher than that at Site 2 (conventional, 1.28 m). Despite this, Site 1's streamflow volume is 6% lower than that of Site 2. More importantly, Site 1 (GI)'s precipitation-streamflow ratio is kept within a steady, lower range (32%–49%) than that of Site 2 (conventional, 30%–66%). Site 2's more fluctuating ratio suggests a "flashy" stream condition.

Figure 8. Annual precipitation in Site 1 (Panther Creek watershed, The Woodlands green infrastructure development) and Site 2 (Bear Creek watershed, conventional development), 2002–2009.

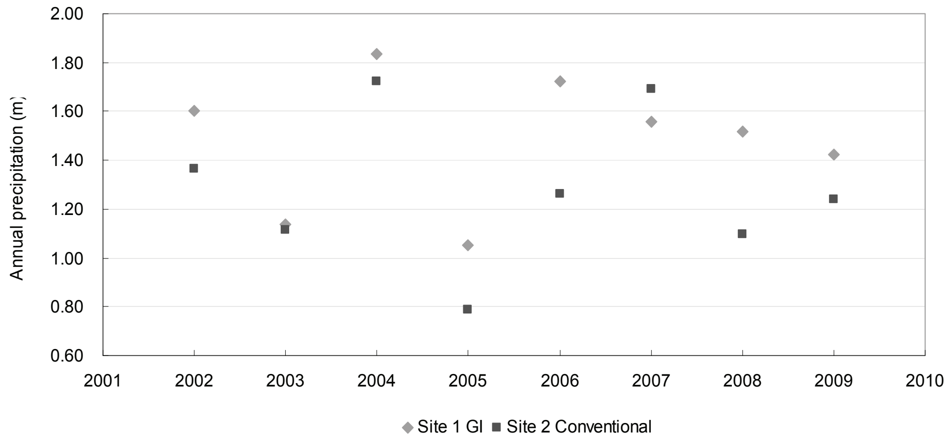


Figure 9. Annual streamflow-precipitation ratio and precipitation depth of Site 1 (Panther Creek watershed, The Woodlands green infrastructure development), 2002–2009.

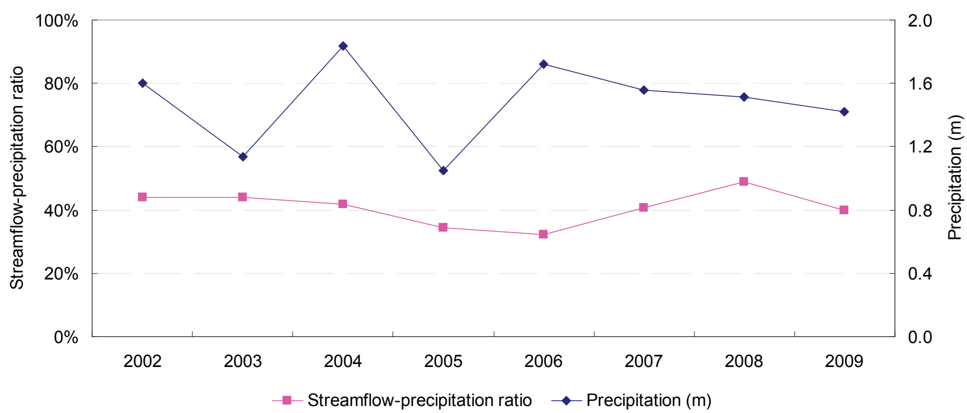
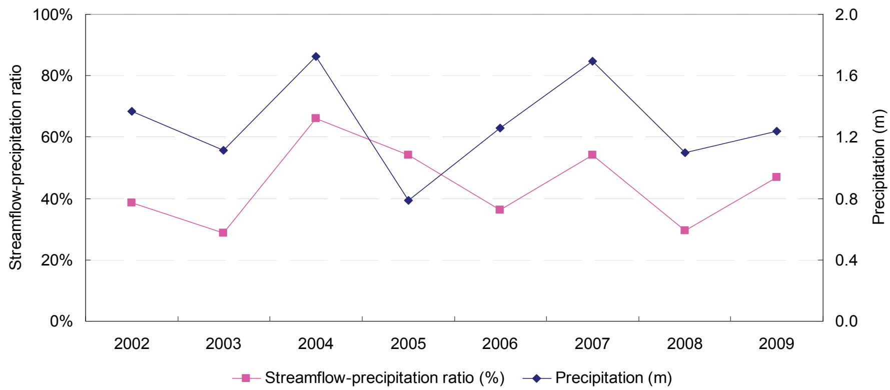


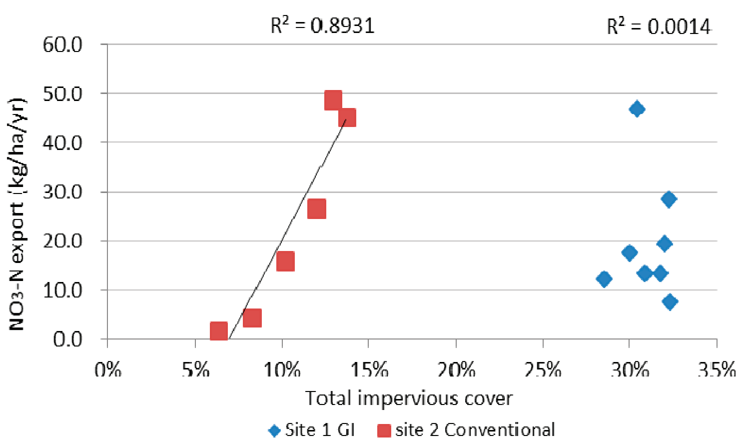
Figure 10. Annual streamflow-precipitation ratio and precipitation depth of Site 2 (Bear Creek watershed, conventional development), 2002–2009.



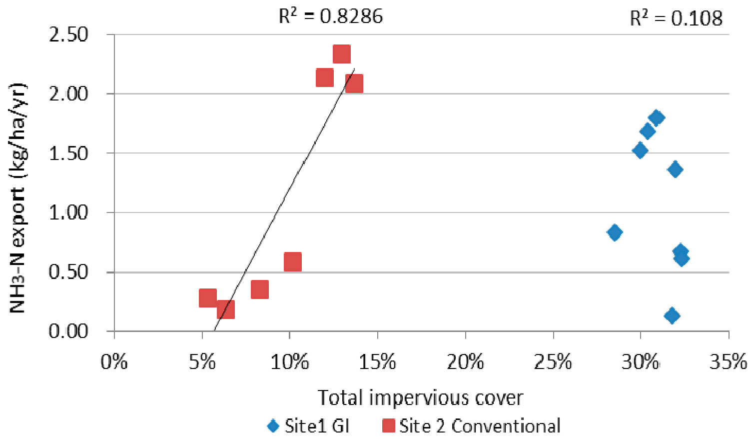
3.4. Nutrient Export Loading

Figure 11, Tables 3 and 4 show the regression analysis between nutrient loading and impervious cover percentage. The results reveal that nutrient loadings are tightly correlated with impervious cover in Site 2 (conventional). In contrast, in Site 1 (GI), there is little correlation between nutrient loadings and the extent of impervious ground cover. These analyses further suggest that GI design can create a robust system that is tolerant to development impacts. Thus, nutrient loadings show a similar response to streamflow volume analyses. NO_3-N export increased in Site 2 (conventional) after development; however, little change was found in Site 2 (GI). NH_3-N export showed a similar trend as NO_3-N export from Site 2 (conventional). Likewise, TP export presented a significant ($p < 0.01$) trend in Site 2 (conventional), whereas no trend was found for Site 1 (GI).

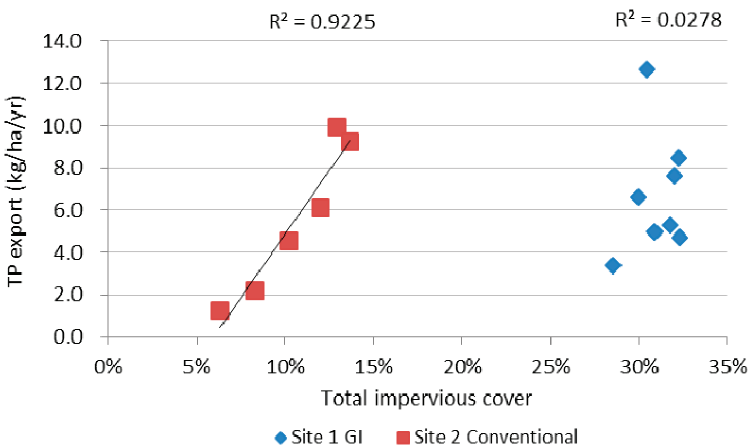
Figure 11. Annual loadings of nutrient exports from Site 1 (Panther Creek watershed, The Woodlands green infrastructure development) and Site 2 (Bear Creek watershed, conventional development), 2002–2009: (**a**) NO_3-N, (**b**) NH_3-N, and (**c**) TP.



(**a**)



(b)



(c)

Table 3. Relationship between watershed impervious cover percentage and nutrient loading in Site 1 (Panther Creek watershed, The Woodlands green infrastructure development).

Nutrient	R^2	Equation	*P*-value	Sample size (2002–2009)
NH_3-N	0.108	NA	0.427	58
NO_3-N	0.001	NA	0.930	33
TP	0.028	NA	0.693	33

Table 4. Relationship between watershed impervious cover percentage and nutrient loading in Site 2 (Bear Creek watershed, conventional development).

Nutrient	R^2	Equation	*P*-value	Sample size (2002–2009)
NH_3-N	0.829	$y = 0.028x - 0.002$	0.004	78
NO_3-N	0.894	$y = 0.666x - 0.046$	0.004	57
TP	0.923	$y = 0.12x - 0.007$	0.002	56

Note: x is watershed impervious cover percentage; and y is nutrient loading.

4. Discussion

The eight years of empirical data yield consistent results showing that GI design produces less development impact on the flow regime and better stormwater quality than the conventional drainage design. As of 2009, the percentage of impervious cover in Site 1 (GI, 32.3%) is more than twice that of the Site 2 (conventional, 13.7%). In addition, Site 1 (GI)'s total precipitation is 15.3% higher than that of Site 2 (conventional). Further, Site 1 (GI) has a much lower runoff infiltration capacity than Site 2 (conventional) (e.g., 38.7% *versus* 80.2% of A & B soils). The opposite is true, however, when comparing watershed outputs—the Site 1 (GI) streamflow volume and streamflow-precipitation ratio are lower than those of Site 2 (conventional). Therefore, the differences in streamflow response can be largely attributed to the different drainage designs.

Figures 9 and 10 show that Site 2 (conventional) streamflow is more sensitive to precipitation and that it has a lower runoff storage capacity than Site 1 (GI). Large variability of streamflow-precipitation ratio suggests "flashy" stream conditions. In wet years such as 2004 and 2007, streamflow-precipitation ratios in Site 2 (conventional) increased dramatically. In contrast, Site 1 (GI) maintained a stable flow regime, which can be interpreted as low disturbance on the riparian habitat and riverine ecology. Likewise, water quality analyses showed consistency with the findings in streamflow. Nutrient exports from Site 1 (GI) are in general lower than that of Site 2 (conventional) (see Figure 11).

This study also demonstrates that GI design can be applied across different scales. The study shows that large-scale GI performance (e.g., a few thousand acres) can be as effective as in site-level scales (e.g., 10–50 acres). GI design was implemented across various scales in The Woodlands. At the regional scale, lands with large patches of sandy soils were preserved as open space to infiltrate runoff [45]. Road alignment considered sandy soil locations where check dams were built to slow runoff velocity (see Figure 3) [31,50]. At the site level, a Landscape Clearance Index was developed to ensure the minimum clearance of vegetation. This index specified guidelines to preserve vegetation under different soil conditions in order to achieve the objective of zero runoff from individual parcels ([51], p. 46).

Moreover, this study confirms with previous studies that *integrated* GI design strategies are better than a single strategy [30,31,44]. This is because Site 1's GI design mimics the natural hydrological cycle by keeping the portion of runoff that originally infiltrates underground. Soil and vegetation medium further improve water quality. In other words, the decentralized, on-site runoff treatment reduced the EIA after The Woodlands development.

The effectiveness of single GI designs is often reported in the literature, such as pollutant removals of rain gardens, green roofs, and porous pavements [81–85], and the USEPA's current guidelines are also focusing on performance measures of individual GI designs [7]. This study contributes to the USEPA's guidelines by demonstrating that integrated GI design strategies are effective in reducing the EIA and improving water quality. Site 1 (GI) presents a much higher TIA than Site 2 (conventional) (*ca*. 2.4–5.4 times). However, its streamflow volume is 6% less than that of Site 2 (conventional). This means that the EIA of Site 1 (GI)—the direct contributor to runoff volume and quality impairment—is considerably lower than Site 1's TIA (32.3%). Site 1's EIA can be even lower than Site 2 (conventional)'s TIA (13.7%), because the impervious surface areas in Site 2 (conventional) are considered to be well connected for efficient drainage design.

However, the efficacy of McHarg's GI design may not be fully revealed because of the research design of this study. McHarg's GI design innovations were primarily used in the early phases of community development, during which McHarg presided the design [45]. Unfortunately, residents do not appreciate some of the GI designs (e.g., bioswale) [52]. As a result, a hybrid approach which combined McHarg's design and conventional drainage design were used in the later phases of development [58,59]. About one third of The Woodlands' early phases (*i.e.*, followed McHarg's design) do not lie in the Panther Creek watershed. Hence, performance of GI design may be underestimated due to these study limitations. Also, the study cannot completely tease out the performance of The Woodlands early and later development phases in respect to their stormwater performance, because they were treated together as one study site (*i.e.*, Site 1).

Another study limitation is that the small water-quality sample size may decrease the precision of nutrient loading estimations. The TCEQ uses a sampling frequency of one month to meet the monitoring objectives in western Houston areas. Littlewood's method used in this current study is based on these discrete water-quality data to estimate annual mass loads [79,80]. The precision range and confidence level of estimation decrease when the sampling frequency (e.g., monthly) and the length of the estimation period (e.g., five years) decrease. A sampling frequency that is too low (e.g., less than six samples per year) is not recommended—a principle followed in this study. In addition, estimates for dry years exhibit higher precision than those for wet years. This study used the best available data and the study period contains normal variations of dry *versus* wet years.

Finally, the 1.01 km^2 (250-acre) lake in Site 1 (GI, The Woodlands) upstream of the TCEQ gauge station is likely to dilute the concentration of pollutants contributed by the upstream areas of the lake. This study cannot tease out this lake dilution effect and the effect presents some limitations. However, according to the original design [86], the lake is intended to serve as a recreation amenity and as a flood control device in The Woodlands comprehensive stormwater management plan. Therefore, this integrated design strategy showed success in flood control and water quality improvement. Nonetheless, future studies are called for to evaluate the performance of each individual strategy and to compare it with the overall efficacy of the integrated strategy, as shown in this current study.

5. Conclusions

This study compares the stormwater management performance of GI design and conventional design at large-scale community developments (89.4 km^2 and 55.7 km^2). Empirical evidence strongly suggests that integrated GI application can be effective in stormwater runoff reduction and water quality improvement. Despite a much higher TIA, GI design results in a much lower runoff volume, compared with conventional design. Further, nutrient outputs are significantly correlated with the extent of development in the conventional site, whereas there is little correlation in the GI site.

In the United States and many parts of the world, community development continues to be a major development project and covers a large territory of land. The status quo of stormwater management practices increasingly draws criticism for causing environmental quality problems. This study shows that integrated GI design can help achieve multiple stormwater management goals and may provide a cost-effective solution. In addition to the enormous construction cost savings, the potential flooding costs and potential pollution treatment costs can be avoided. Future studies need to directly assess the EIA and GI performance during peak events. Further studies are also needed to test GI performance in different climatic conditions and to improve the aesthetics of GI design based on public input.

Acknowledgments

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Conflicts of Interest

The authors declare no conflict of interest.

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Addressing Flooding and SuDS when Improving Drainage and Sewerage Systems—A Comparative Study of Selected Scandinavian Cities

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Abstract: Pluvial flooding already challenges the capacity of drainage and sewerage system in urban areas in Scandinavia. For system owners this requires a stricter prioritization when improving the systems. Experts seem to agree that a regime shift from improving old combined sewers by piped solutions to more sustainable drainage systems (SuDS), must take place. In this paper results from an investigation amongst the largest cities in Norway, Denmark and Sweden concerning drivers and preferred methods for improving the old system are presented. The results indicate that Norway ranks flood prevention lower than the other Scandinavian countries. During the last decades, Norwegian authorities have had a strong focus on pollution from wastewater treatment plants (WWTP). The attention to drainage and sewerage system regarding flooding, water leaks, infiltration or pollution has been neglected. Renewal or rate of investment in relation to existing drainage and sewerage system is easy to register, and provides a measure of the activity. In order to optimize flood prevention, and may be promoting the use of SuDS, the cities should be required to measure the efficiency, either by monitoring or modeling the impact of stormwater to the system. Lack of such requirements from Norwegian authorities seem to be a plausible explanation to why Norwegian cities are less focused on flood prevention compared to Swedish and Danish cities.

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1. Introduction

In a period with changing climate, impacts on both precipitation patterns and urban drainage will occur [1]. Increasing total rainfall and rainfall intensity will result in a greater load on the drainage and sewerage systems. These important infrastructure systems were designed and built years ago, and increased precipitation was not part of the design criteria. In addition, improper maintenance, aging *etc.* causes many problems. In Norway more than half of the systems are built before 1980 [2], and in central parts of the cities you will find the oldest systems.

Conventional piped drainage systems are designed for specific maximum flow rates and will be unable to meet the increase in the water volume [3]. Sustainable Urban Drainage Systems (SuDS) like ponds, open ditches, green roofs, *etc.* are in many countries made for stormwater treatment. In urban areas in Scandinavia the authorities only to a small extent have required stormwater treatment, and SuDS have then largely been considered as a flood prevention measure e.g., in Malmö, Sweden [4]. It has been shown e.g., in Denmark and Germany that decentralized solutions for stormwater handling are more flexible than conventional drainage systems. This flexibility is important when dealing with the uncertainties regarding future consequences [5–8]. An Irish study [9] concluded that although the benefits of SuDS are obvious, they are not sufficiently appreciated. The water and wastewater sector is considered to be very conservative [10,11], and the engineering culture is often referred to as a key barrier to implementing sustainable approaches in practice [11,12].

The Norwegian governmental report “Adaptation to a changing climate” released in December 2010, points to the many challenges that Norway is facing in relation to global climate change [13]. The future pace and scale of expected climate change are unknown, and implementing good and adaptable systems today is therefore a prerequisite for a less vulnerable Norway in the future. Urban areas are expected to be areas where the climate changes will be most apparent in everyday life [14]. Population growth and more impermeable surfaces due to more buildings, roads, parking lots, *etc.* are causing increasing strain on the drainage systems in the cities. A change to more sustainable stormwater systems in cities can reduce possible flooding in the urban environment [15].

Norwegian cities, like cities in many other countries, already experience challenges related to urban flooding. There are mainly three reasons for this: Climate changes, rapid urbanization and under-designed sewers [16]. The current pipes in the drainage systems in Norway cannot easily be replaced by larger pipes [17]. Heavy rain storms can lead to a runoff situation where the pipe capacity is exceeded, resulting in flooding events and backflow of wastewater into buildings and basements. This is already a major problem in several Norwegian cities [13]. So far, there has been limited development of *lokal overvannsdisponering-LOD (Local Stormwater Handling)*, which cover both infiltration and detention and is the Norwegian term that best corresponds with SuDS [18].

The organization of the wastewater sectors in the Scandinavian countries is comparable. Water distribution- and wastewater services in Scandinavian cities are all public services. The main systems are directly or indirectly owned by the municipalities and are managed either by their own employees or contracted professionals. The municipalities in all Scandinavian countries have for decades been encouraged by the national authorities to increase the use of SuDS [19–21]. The similarities in organization of the wastewater sector make it possible to investigate differences in how future challenges are met, and if this is reflected in the prioritization of the measures. There are some historical differences, while Denmark traditionally dimensioned their combined sewer for a 2 years flood recurrence interval before 1990 [21], Norwegian authorities recommended 5-years [22]. Regarding the responsibility for basement flooding from sewers, Norwegian municipalities have stricter obligations than in Sweden and Denmark [18].

Flood prevention measures involve many stakeholders with different perspectives although they are often seen in multidisciplinary cooperation. It is generally believed that climate changes are expected to cause more flooding in urban areas in the future [1,6,15,17], but how these changes will develop are not further discussed in this paper. Much of the impact of heavy rainfall in urban areas, are related to the drainage and sewerage system. The aim of this paper is then to investigate how the system owners’ in practice are focusing on measures to reduce or prevent problems with pluvial flooding in urban areas e.g., backflow and flooding of basements. This includes measures either to avoid, delay or convey stormwater in the system. This is believed to be a challenge in urban areas worldwide, but as a basis for this study, a survey among the largest Scandinavian cities was carried out. Since this study deals with urban flooding, it was assumed that the largest cities were the most relevant selection for the study. The hypothesis was that the system owners in Norway, when improving old drainage and sewerage system, have little focus on flood-prevention, while other Scandinavian countries dealing with the same challenges rank flood prevention higher. In this paper, the term *improvement* is used independent of whether the methods are convential (renovating or renewing the piped sewers) or using SuDS. Summarized, the aims of this study are:

- How prioritized is flood prevention when Norwegian cities are improving their drainage and sewerage system? To what extent are SuDS the preferred method when improving the system?
- Are there any differences amongst the Scandinavian countries in how the cities or the national authorities meet this issue?

Key factors, such as technical conditions, incidents, economy and competence are believed to affect the priorities which are chosen. These factors are compared to identify possible causes for why flood prevention in urban areas is prioritized differently in the Scandinavian countries.

2. Background

The annual precipitation in Norway has increased by 20% during the 1900s, and some places it has increased with almost 2% per. 10 years some places since 1980 [13]. Extreme rainfall events in Norway are expected to increase slightly up to 2025, and then sharply towards 2050 [23]. In small catchments areas (20–50 ha), the maximum flow will normally occur during the summer months [24]. It is estimated that it will continue to rise with an average of 13% in the period 2071–2100 compared to 1961–1999 [16]. In the period 2071–2100, the intensity of the heaviest summer rains in Oslo is estimated to be 20% higher than today [25], while corresponding rains in the autumn are expected to become 40% higher than today. A comparison of extreme rainfall events with 24 hour durations from the past 100 years [26], show only small variations between the Scandinavian countries regardless of the return period and season. The western coast and mid-Norway experience the greatest extreme weather conditions in Scandinavia. However, only small differences are found when comparing specific measurements from the capitals of each country.

Precipitation and flooding in cities result in a number of social costs such as traffic disturbance, damage to infrastructure and buildings, sick leave due to infectious water, lost sales for businesses, pollution of drinking water and local recipients [24]. The insurance companies believe that these costs could increase by 40% or more over the next ten years. This estimate does not include conditions that are defined as natural disasters. The insurance companies are therefore working on a strategy to handle the expected increase in damages. They consider transferring more risk to both private homeowners and municipalities, if they are not willing to adapt to the assumed climatic changes [17]. There have been several court cases regarding heavy urban flood damages in recent years (e.g., Fredrikstad, Stavanger, Alta) [27,28]. All these cases have emphasized that insurance companies in the future will hold the municipalities more liable for flooding related to insufficient capacity of the mains. Not all costs are easy to determine, but from 1992 to 2007, Norwegian insurance companies paid 3000 million EUR in compensation for water damages. The expenses rose each year during the period, most likely due to frequent torrential rains and more rain in general. It is estimated that approximately 25% of these payments were due to flooded houses caused by insufficient urban drainage system [29].

In recent years, there have been several damages caused by heavy rain in Norway, for instance in Fredrikstad (August 2008) and in Drammen (August 2012), which resulted in major damages. Sweden has been less exposed to urban flooding, but some extreme events have caused significant social costs. Copenhagen in Denmark had a major rainfall in the summer of 2011. This is one of the clearest examples of extreme rainfall, which have consequences both for housing and infrastructure. Total insurance payments amounted to about 800 million EUR, distributed among approximately 80,000 cases [30,31].

Even though it is not possible to make an exact comparison, the above shows that there are many common challenges, and focus on flood prevention measures in urban areas should then be ranked almost equally in the Scandinavian countries.

3. Theory

The capacity of stormwater systems may be increased by new and larger pipes when old pipes cause problems with flooding, pollution, *etc.* This conventional method is no longer seen as sustainable [32], and if possible, it is increasingly replaced by non-piped solutions in more and more countries. How far this trend has been developed in different countries, vary widely, and great diversity is seen even within countries. In urban areas, it is not realistic to establish stormwater systems that completely consist of non-piped solutions. However, it is important to plan for an ever-increasing flood risk, and take into account that this will be an even greater challenge in the future. For a city, optimal measures will rarely consist of one single method, but a selection of sustainable solutions adapted for local conditions and requirements.

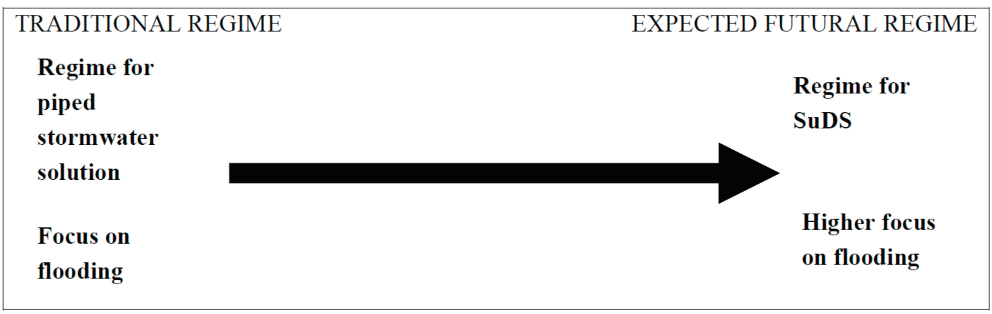
In the wastewater sector like many other sectors, a dominating way to solve a social subtask can be denoted as a *regime*, and such a regime is typical for the way we meet the societal needs [33]. Other regimes, which have a power are denoted *niche-regimes*, although they are not dominating the way that the societal needs are met. Niche regimes fundamentally challenge the dominant regime. A change in which a niche-regime emerges, and finally oust the dominant regime, may occur. The dominant regime will at any time be what protects the society's needs in the best way. This transitional change is denoted *regime shift*. The speed of this transition is influenced by a complex number of conditions, which drive the transition.

According to Ashley, *et al.* [34] the societal system is composed by a number of societal subsystems, and storm water management in cities is an example of such a social subsystem. The way to solve these challenges in cities, deals with two fundamentally different competing regimes. The developed part of the world is at different stages in the transition from the traditional storm water regime to other systems. The old regime, which in most cases also is the current regime, is to improve the system through piped solutions either by combined or separate systems. They state that the traditional piped solutions for handling storm water are the dominant regime in most cities. Changes in boundary conditions (*i.e.*, more flooding as a consequence of climate changes) may change the society's opinion and help the niche to develop. But a sudden increase in flooding events may be met by the decision makers by conventional renewing methods, because there is no time for untested methods as SuDS. Thus, the uptake of this niche may be delayed. However, the development of SuDS has come with an increasing focus at the possible impact of climate changes [35]. It is then assumed a transition towards the new and more flexible regime for storm water management will occur.

4. Methods

A general theoretical model [33], adapted by Ashley *et al.* [34], is used in this context. The increased attention to flooding as a target and SuDS as a preferred method to solve this is illustrated in Figure 1 as a transition line between the old and the new regime. According to Geels [36], the conceptual characteristics of a regime transformation is that the regime insiders gradually change their cognitive beliefs and behavioural norms.

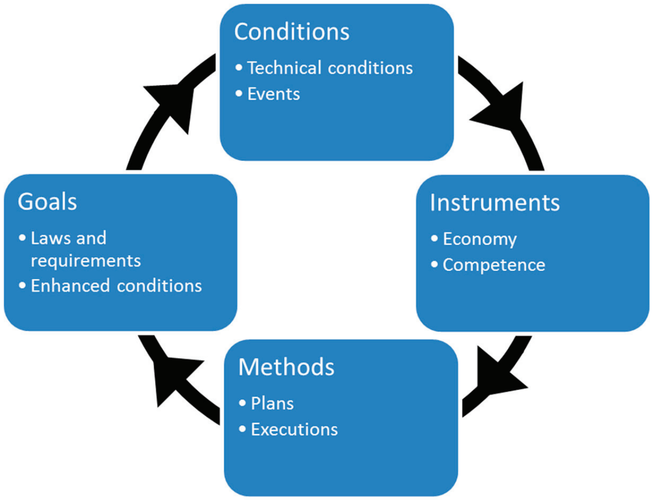
Figure 1. Transition line toward a sustainable urban drainage system (SuDS)-focused regime.



In step 1 of this survey a comparison between Norway and other Scandinavian countries was made, both in regard to the target for the improvement and the methods used.

The next step of this study was to make a model of factors that influence the present regime. These are the factors that combined can provide an explanation for the situation in each country, as shown in Figure 2. The factors are interrelated, and can be viewed as a continuous improvement process. Bos and Brown illustrated this in a broader perspective as "*Phases of governance experimentation leading to adaption in water governance structures…*". They mention this as strategic, tactical, operational and reflexive activities [12]. When a goal and a desired condition are achieved, new goals will be set and the process starts over again. The purpose of the model is to identify relationships between individual factors that may explain the differences, which are found in step 1.

Figure 2. Factors affecting flood and SuDS–focus—illustrated as a continuous process.



The model in Figure 2 can be used to compare any urban wastewater systems, (e.g., cities or companies). In this study, however, the model was used to compare the SuDS-focus in the Scandinavian countries. Within each factor, some quantitative and relevant parameters were identified and compared. In Figure 2, the term *Conditions* is used to describe the state of the technical facilities and the consequences of this condition. Renewal rate, the rate of combined systems or the amount of infiltrated water are all indicators for the conditions of the drainage and sewerage systems. In addition, water leaks are used as an indicator because this causes more water to infiltrate the drainage system, and affects the choice of method for repairing the system. In this study, the term *events* includes registered damages at insurance companies and economic costs of extreme rainfalls. *Instruments* are factors that can be utilized to change the conditions, e.g., the financial resources the owner is willing to spend and available expertise. This will mainly include professionals, but in an initial phase it may also include politicians and the citizens as well. The term *Methods* is used for the possible physical measures. These are again seen as a result of choices and strategies that have been taken to improve the condition of the system. The primary *Goal* in relation to this will be to reduce the risk of flooding. Within the wastewater sector, many of these goals are regulated by the EU Framework Directive, which is current legislation in all Scandinavian countries.

The survey was made out to capture trends, and it was designed to create a holistic view for the largest cities in Scandinavia. This study did not deal with the rate of change or the actual transition to a new regime. The results of the study were viewed in the light of the models described in Figures 1 and 2.

The wastewater plan, like other urban development plans, does not give a complete picture of how and why the cities prioritize new projects in practice [37]. The plans do not always show the preceding ideas and internal discussions among professionals. Therefore, the personnel managing the wastewater sector in each city were contacted and asked to take part in the survey. It was assumed that these persons have a great influence on the decisions for planning and implementing renewal projects. The largest cities are supposed to be the most relevant selection when it comes to urban flooding [38]. Smaller communities might be less vulnerable to flooding due to a higher proportion of natural green areas in the vicinity. However, they might also lack engineers to provide adequate solutions to flooding problems. Accordingly, small cities were excluded from the study, since these are expected to encounter different challenges than larger ones. In addition, the major cities in each country are expected to reflect the "national best practice" in relation to urban flooding. The current study analyses drivers and methods used by system owners for improving the drainage and sewerage systems, based on completed projects in the chosen reference year 2010.

Initially 10 Norwegian cities were visited in May–June 2012 and interviewed based on a qualitative study. This was done to get an overview of the state and to confirm the validation of the questions. Then the remaining 15 of the 25 largest cities were contacted and accepted to receive a questionnaire, which later was sent by mail. Respondents were asked questions about the improvements of existing drainage and sewerage system in a given reference year (2010). The key questions were triggering reasons and used methods when improving the system. In addition, they were asked questions about the condition of the system, availability of staff, and financial constraints. A similar study was done in Sweden and Denmark during winter 2012/2013. Based on the experience from Norway, three cities were visited and interviewed to confirm the questions. The rest of the cities among the 25 largest, were contacted and accepted participation in the questionnaire, which later was sent by mail.

From the survey in Norway, 22 of 25 cities (88%) responded. Similar numbers in Sweden were 14 of 25 (56%) and in Denmark 16 of 25 (64%). In addition to the questionnaire, quantitative data from national registers (Bedre VA (Norway), VASS (Sweden) and Danva benchmarking (Denmark)) for the reference year 2010 were collected. Even though the study was limited to the largest cities in the considered countries, the difference in population in the cities in the survey was substantial. Accordingly, weighting the results by the economy or population of the cities would result in a bias towards the trends in the largest cities (weighted answers from the smallest cities would have counted only 5% to 10% relative to the largest cities). Since the goal was to capture trends, the use of non-weighted averages for each country was selected.

There are obvious differences between the Scandinavian countries that must be taken into account before analyzing the results of the survey. The median number of inhabitants in the Norwegian cities that responded was approximately 47,300, while the corresponding numbers in Sweden and Denmark were 98,900 and 94,800, respectively. It is not reasonable to assume that the results from the larger cities are representative to smaller cities with less manpower, less financial resources and less population density. However, in this study there was no significant trend that the larger cities used other methods and had different reasons to improve the system than the smaller ones.

The results were related to the theory described above and presented in two steps. Step 1 was based on the responses to the questionnaire of selection process and methods for improvement projects in a given reference year. The results of this were used to calculate Norway's position in the transition towards a more sustainable storm water regime compared to Sweden and Denmark.

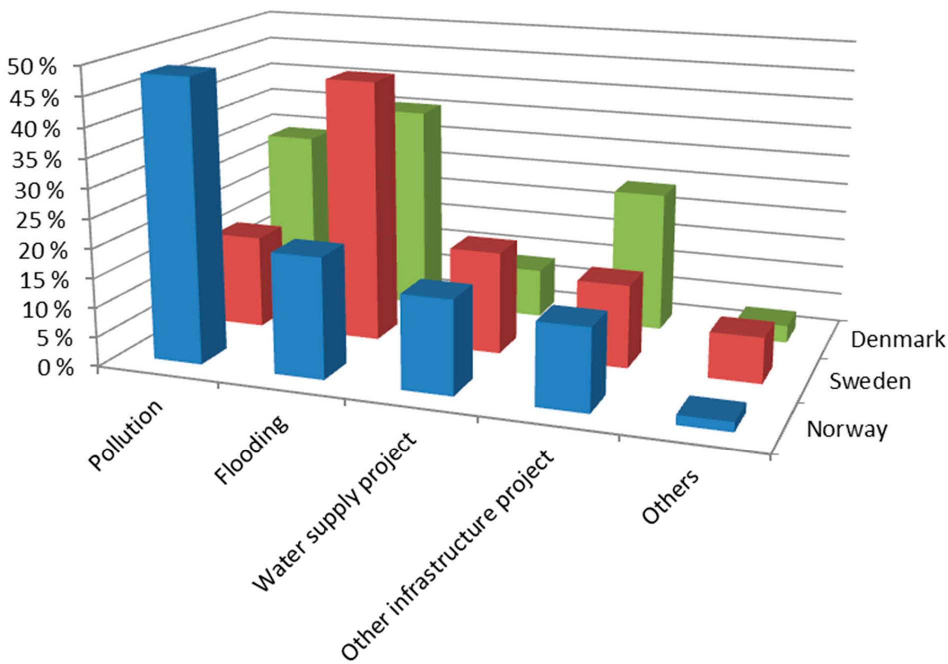
In step 2 additional results from the survey, national benchmarking and literature review were used to find the underlying reasons for the differences between the considered countries.

5. Results

This study primarily investigates how cities were dealing with flood prevention. However, it also included an investigation regarding how measures in relation to existing drainage and sewerage system were undertaken. Measures are planned and conducted by the same professionals, and often carried out at the same time and need to be within a given budget. It was therefore relevant to compare the different triggers for improvement projects.

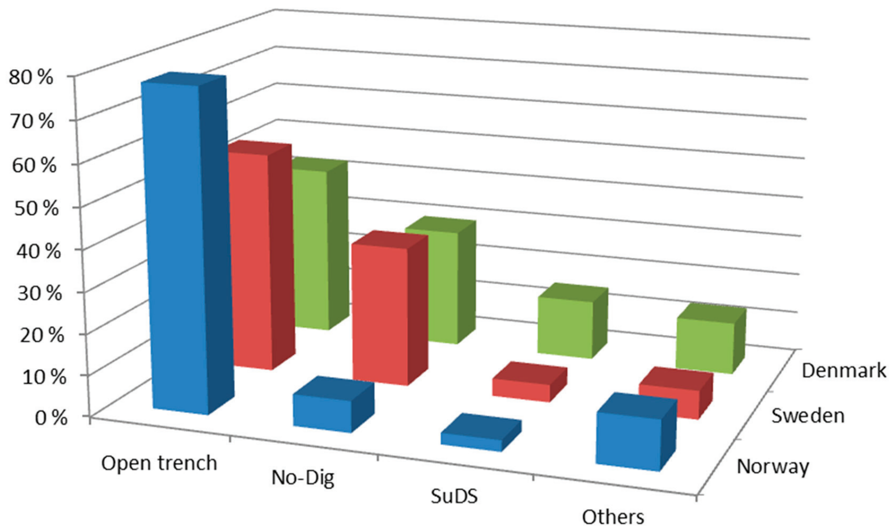
In step 1 of the survey, the engineers in the cities evaluated both the triggering cause and method in the reference year 2010. A project can have multiple purposes, and therefore the triggers could be somewhat more difficult to determine than the methods. However, they were requested to state what they believed were the main triggers. It is reasonable to assume that some causes require specific methods, thereby providing a close connection between them. It is accordingly appropriate to discuss these answers together. The distribution of causes triggering projects in the existing drainage and sewerage system in the largest Scandinavian cities in 2010, are shown in Figure 3.

Figure 3. Causes triggering improvement projects in existing drainage and sewerage systems in the largest Scandinavian cities in 2010.



When comparing this, life-cycle analysis (LCA) or other tools could have been useful [39], but in Figure 4 the projects are ranked by the financial investments. Open trench means digging up and replacing old sewers, while No-Dig covers relining, blocking or other possible methods for renewing the old pipe without digging. SuDS include non-piped solutions as ponds and open ditches trench, mainly built for flood protection. Compared to many other methods, SuDS are normally less capital intensive, and the amount spent on sustainable solutions is expected to be far lower than other methods such as open trench.

Figure 4. Methods used to improve existing drainage and sewerage systems in the largest Scandinavian cities 2010.



Some clear trends in relation to flooding were found in the survey and are shown in Figures 3 and 4:

- Compared to Sweden and Denmark, there were fewer cases in Norway where prevention of flooding was the triggering factor to wastewater projects. Pollution was reported to be the main reason for most drainage projects in Norway, far more important than in the other countries.
- Sustainable methods of stormwater management were used more frequently in Denmark than in the other countries.

In Figure 4 it is shown that SuDS was rarely used in Norway, in average it is only 3% which confirms previous research [18]. More than 80% of the Norwegian cities report that they did not use SuDS at all in 2010. Approximately 45% of the Swedish and 10% of the Danish cities reported the same. The findings indicate that both Denmark and Sweden are more focused on flood prevention measures.

Based on the results shown in Figures 3 and 4 it is not possible to see a correlation between focus on flooding and the use of SuDS. However, it seems to be a trend that Norwegian cities are more one-sided and traditional both in their targets and choice of methods to improve the drainage and sewerage system.

The limited focus on SuDS indicates that Norway is placed to the far left in Figure 1. Based on the same criteria, the survey indicates that Danish cities have made most progress in the development towards a more sustainable stormwater regime.

In step 2 of the study, the model in Figure 2 was discussed with an intention to explain the differences in step 1. Factors assumed to be relevant are shown in Table 1.

Table 1. Comparison of factors that may affect flooding and SuDS-focus.

Factors	Characteristics	Characteristics for Norwegian cities (N)	Characteristics for Swedish cities (S)	Characteristics for Danish cities (DK)
Conditions	Rate of combined sewers (2010) [1]	31%	13%	48%
	Renewal rate (2010) [1] per. Year [1]	0.74%	0.38%	1.07% (2000–2010)
	Number of basements flooding in houses caused by the drainage and sewerage system 2008–2010 [2]	6,000–6,500	6,000	6,000–9,000 (2008–2009), 20,000 (2010)
	Infiltrated water in the largest treatment plants in 2009 [3]	68%	58%	23%
	Leakage from drinking water networks 2010 [1]	43%	23%	9%
	Cities reporting lack of capacity [4]	32%	7%	7%
Instruments	Fee for a standard residential (2010) [1]	225 EUR per year	173 EUR	359 EUR
	Cities reporting good or adequate financial frames to improve the systems [4]	95%	42%	80%
	Cities reporting shortage of internal professionals [4]	59%	64%	23%
	Cities reporting shortage of available external expertise [4]	26%	29%	0%
Methods	Use of methods (ref. Figure 4) [4]	Most use of open trench	Less use of open trench, more use of No-Dig compared to N	Less use of open trench, more use of No-Dig compared to N
	Number of cities invested in SuDS (2010) [4]	18%	54%	92%
Goals	EU Water Framework Directive is the most relevant international legislation in the sector and is basically the same in all Scandinavian countries. In S the EU Flood directive is implemented for urban flooding, in contrast to N and DK.			
	N reports activity in the voluntary national benchmarking (Bedre VA) and required national reporting (KOSTRA). Both S and DK report the activities as in N. No reporting of emissions from transport system is required in N. Most of the cities in S and DK report emissions from all CSOs. In S this is reported to the regional, and in DK to national environmental authorities.			

Notes: [1] Data from national benchmarking (Bedre VA, VASS, DANVA benchmarking) for the 25 largest cities in each country which have registrated data; [2] Comparable insurance data. For Norway and Denmark 2008–2010, for Sweden 2010 [40–42]; [3] According to Lindholm, *et al*. [43]; [4] Survey of the largest cities in Norway, Sweden and Denmark related to this paper.

6. Discussion

6.1. Conditions

When evaluating the technical condition of the drainage and sewerage systems in relation to flooding, it is relevant to compare the share of combined sewers. From Table 1 it can be seen that both Norway and Denmark have significantly more combined sewers than Sweden, and from Table 1 it can be seen that leakage from drinking water network is significantly higher in Norway compared to Sweden and Denmark. Even if leaks from water pipes into sewers are unaffected by precipitation, it is relevant in this context, because it causes reduced capacity to handle extreme rainfall.

Infiltrated water is defined as any unwanted water entering the sewers and is, according to Lindholm *et al.* [43], higher in Norway than in the other Scandinavian countries. Much infiltrated water results in extra large flow during periods with heavy rainfall. As an additional question, the cities were requested to make subjective evaluations of the sewers. The responses fit well with the study of infiltrated water. Evaluated on the basis of capacity, the Norwegian cities are rather more pessimistic than in the other countries, and approximately 30% state capacity as poor/reduced. Among the Swedish and Danish cities, less than 10% report this.

An effect of poor condition of the systems is a high number of registered flood damages after large rainfall events. To identify challenges from urban flooding in Scandinavia, the number and cost of flooding from sewers registrated by insurance companies can be compared. From the Norwegian register of water related damages [40], the number of damages from 2008 to 2010 were about 6000–6500 per. year and with an estimated cost of *ca.* 35–40 million EUR each year. Statistics from Sweden the recent year [41] have estimated that these costs are 30–35 million EUR. Sweden is almost twice as densely populated as Norway. The number of damages due to lack of capacity of the drainage systems is low from the Swedish insurance companies' point of view [44]. Even if it is an increasing problem, it is not yet seen as a big challenge compared to other kind of damages. In Denmark there are statistics for cloudbursts [42], but this is not separated into the different kind of damages. In Denmark, the number and cost of damages was estimated to be at same level as Norway in 2008–2009, but it was more than doubled in 2010. However, this increase is probably linked to differences in spesific events, and not to the conditions of the systems.

Comparison of several parameters describing the current state indicate that Denmark has experienced more damages caused by some spesific incidents, while Norway has significantly greater challenges in terms of the technical conditions of the sewers than Sweden and Denmark.

6.2. Instruments

According to the selected instruments, the survey generally showed a more positive trend in Danmark. They were less conserned about the capacity and had fewer challenges in recruiting professionals than Norway and Sweden.

Both Sweden and Denmark have an opportunity to levy a separate stormwater fee [45,46], which may lead to consciousness for sustainable stormwater treatment. Sweden and Norway have significant lower fees than Denmark. The cities were asked whether they had sufficient financing to improve the drainage and sewerage systems in the reference year 2010. Although the Norwegian cities had lower fees than Denmark, the professionals in Norway are more positive to the available financial resources than the largest Danish cities. A comparison of instruments indicates that Norway has a challenge in recruiting enough professionals. There are also strong indications that they have lower ambitions in relation to what is sufficient economic framework to improve the system.

For the Swedish cities, it is a more significant correlation between low fees and dissatisfaction of the financial frames of the drainage and sewerage systems.

6.3. Methods

The results presented in Figure 4 indicate that replacing old pipes is far more common in Norway than in the other Scandinavian countries. This means that old combined systems were dug up and replaced with separate sewers. The method is both expensive and time consuming in urban areas, but is a safe method to reduce pollutant emissions, provided that all private service pipes in the area is in good condition or replaced at the same time. The municipal engineers in Norway are more satisfied with the financial framework than in the other countries. This may be the reason why they often choose to improve the system by open trench. Moreover, Table 1 shows that water leaks is such a big problem that in many ways the use of full digging is preferred and thus it is suitable to separate the system too.

In the survey, No-Dig-methods seemed to be little used as a renovation method in Norwegian cities in contrast to Sweden and Denmark. According to Lindholm [47] the largest cities in Norway have an ever increasing use of No-Dig as the preferred renovation method. Apart from that, water leaks can enforce open trenches; a possible explanation may be that Norway is less densely populated. Otherwise, there are no clear technical reasons why No-Dig-methods are less used in Norway than in Sweden and Denmark.

As mentioned above, SuDS are found to be significantly more frequently used in Denmark than Norway. One explanation may be that Denmark traditionally has greater need to restore stormwater to the natural environment, since 99% of drinking water sources in Denmark are groundwater. Accordingly, Denmark already has a tradition of SuDS planning since the 1990s, before the climate changes came into focus.

Methods for improving the wastewater system vary less in Norway than in the other countries. Uniform use of methods may mean that Norway has some extraordinary challenges which only can be solved by open trench. The water leaks from water supply network may be such a challenge. Another possibility is that the current and past requirements do not encourage varying methods in relation to the challenges that arise. As previously mentioned [10], the wastewater sector in Norway is known to be conservative. It may, in addition to shortage of professionals, be the reason why testing of more sustainable methods are prioritized lower than in Denmark.

6.4. Goals

EEC and national laws regulate flooding and damage from surface water in all Scandinavian countries. The Water Framework Directive aims at ensuring that all watercourses are returned to a natural state. The Flood Directive requires the responsible authority to do risk analysis to identify potential flood incidents. Actions that ensure the achievement of an acceptable level of risk should be taken by 2015. In Sweden, the EU Flood directive is implemented for urban flooding, in contrast to Norway and Denmark. In addition, there may be differences in national requirements and particularly in how they are practiced.

Both in Sweden [46] and Denmark [48], separate laws for the water- and wastewater sectors have been passed. In Norway, relevant acts governing the wastewater sector are integrated in several laws. The Planning and Building Act, the Water Resources Act and the Pollution Control Act are the most relevant laws [24,49]. Although sector laws have given the wastewater management increased attention in Sweden and Denmark, the short time since these laws were passed suggest that this is probably not the main explanation for why Norway has different priorities.

In terms of preventing flooding, it is particularly interesting to compare the requirements from the national authorities regarding the impact of stormwater to the drainage and sewerage system. The way in which the requirements from the authorities have been given and controlled appears to have varied since the 1990s. The investigation indicates that Norwegian cities, in the reference year 2010, have the same priority as they had before climate change became an issue.

Interestingly, the Norwegian pollution authority has not demanded monitoring or modeling the efficiency of the improvements in the network during the last 20 years. Accordingly, Norwegian cities have never had any incentives to monitor these themselves. Thus, it has not been possible to evaluate the impact of the measures that has been taken, nor is it clear whether the main reason for improvement was to achieve reduced pollution or flood control. Ever since the 1990s, the National authorities in Sweden and Denmark have had a greater focus on monitoring combined sewer overflows (CSO) from sewers than Norway. In Sweden, the overflow values were made public through the EMIR registry to the county administrative board [50]. It was demanded that the overflow volume from sewers which served WWTP designed for more than 500 pe (population equivalents), should be monitored [51]. In Denmark, this is reported by Danish Nature Agency [52]. It appears that the requirements to monitor overflow from transport systems have been the focus of the national authorities in both Sweden and Denmark. In contrast to Norway, this might have made the cities more aware that the emissions from transport systems should affect the priorities when deciding where and how measures are taken.

6.5. Considerations Concerning Improvement as a Continuous Process

In Figure 2, the development process is drawn as a circle, which illustrates that this is a continuous process. Accordingly, when a goal has been reached, for example by an implemented wastewater plan, better conditions are achieved. Thus, the process will commence with a new starting point, and new choices and priorities based on changed conditions will emerge. How to measure and compare the original and the improved condition of the drainage and sewerage system is significant, since this confirms whether the instruments and methods have been optimized.

An indication of the focus Norwegian authorities had in the 1990s is given by Bull [53]. In 1996, it was articulated in a speech by the junior minister in the Royal Norwegian Ministry of the Environment that the goal was to clean up the sewage sector in Norway by the year 2000. It was focused on how to finalize the separation of combined systems, and improving treatment plants within a few years. Guidelines from the regional environmental authorities [54,55] show that the quantitative requirements through the 1990s and 2000s applied only to overflow from wastewater treatment plants. According to Farestveit [56] the Norwegian authorities were concerned about overflow from CSOs in the 1990s, but unfortunately this attention declined in the 2000s.

The survey showed that Norwegian cities have less variation in the use of improvement methods. Open trench, which is a traditional method, was more frequently used in Norway than in the other Scandinavian countries. This fits the findings that Norway has limited internal personnel resources, but acceptable economic constraints. When Norwegian cities specify triggers for a specific project, this is probably based on the intentions for the project. Since loss from transport systems is seldom monitored, the assumption that one method provides a better condition is prevailing, e.g., separation is synonym to pollution reduction. It is difficult to verify to which extent the intended goal is achieved. Improvement projects in the wastewater system in Norway have mainly been reported by activities, e.g., renewal rate (meter pipe per year or % restoration per year) or the investment (amount of money per year). This focus has probably appeared because it is both easy to register and explain to the society. When a significant number of Norwegian cities reported that they currently face major challenges related to infiltration of water into the transport systems, which are recently renewed, there are reasons to question how they register achievement of goals. Lack of requirements may have led to the fact that overflow and other loss from the system have been unknown. Accordingly, the condition and the need for improvements are defined by other, simpler criteria. This may have led to an impression that method and activity are the main goals.

The state of the wastewater system seems to be significantly lower in Norway than in the other Scandinavian countries. There are already considerable challenges to manage increased rainfall. For all countries, and particularly for Norway, it is important to quantify the impact of what has being carried out. More focus on the requirements of measuring the impacts of prioritized projects will probably lead to a more sustainable stormwater management in Norway.

7. Conclusions

Current practice for prioritizing new projects in existing drainage and sewerage system in Scandinavia is shown in Figures 3 and 4. The study, which applies to the reference year 2010, indicates:

- Flood prevention measures are less important target in Norwegian cities compared to the other Scandinavian countries. The most important reason when prioritizing projects in the existing systems is reduction of pollution. In both Sweden and Denmark flooding is more frequently given as the reason for initiating and conduct improvement projects;

- Methods for sustainable urban drainage system (SuDS) are rarely used in Norway. Based on the amount of money invested, Denmark seems to have a higher utilization of SuDS-methods than cities in Sweden and Norway, where the same low rate of SuDS-measures are found. There are also differences in the number of cities, which use SuDS. The respondents from Denmark reports 93%, while the corresponding numbers in Sweden and Norway are 54% and 18%, respectively. Both climate prognoses and increase in insurance damages should indicate that the challenges in Norway are almost the same as in Sweden and Denmark. The condition of Norwegian wastewater system seems to be worse than the other Scandinavian countries. It is therefore reasonable to question why flood prevention and sustainable stormwater handling have such a low priority. The survey was done with reference to the year 2010. The heavy rain in Copenhagen 2 July 2011 or other incidents do not seem to explain the differences.

There are several reasons why Norway has not progressed as far as the other countries in relation to this issue:

- Denmark use groundwater for water supply. Therefore, the return of stormwater to the natural environment has been part of the Danish engineering culture even before it became the focus of climate changes and extreme weather. To a lesser extent, the same could be the case in Sweden. Norwegian cities use surface water for water supply and have more water resources. Therefore, the initiative for taking such considerations is smaller in Norway;
- Shortage of enough competent personnel both internally and in the external consultancy market, may lead to limited resources for innovation and analysis to find the optimal measures. The survey showed that in Norway the prioritization of new projects are done on the basis of the same considerations, and probably with the same methods, as before climate changes became an issue more than 10 years ago;
- There are indications that the Norwegian authorities' interest and actual requirements for the leakage of wastewater in general, and from the transport system in particular, have been lacking compared to the other countries since the 1990s.

To get a better view and more consciousness about the problem, the Norwegian authorities should introduce stricter demands for documentation of total overflow and leakage from the transport system. This can encourage the Norwegian cities to be more focused on the *impacts* of improvement projects rather than the *activity*. Over time, this can lead to a more sustainable stormwater management.

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Conflicts of Interest

The authors declare no conflict of interest.

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An Innovative Approach for Drainage Network Sizing

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Abstract: In this paper, a procedure for the optimal design of rural drainage networks is presented and demonstrated. The suggested approach, exploring the potentialities offered by heuristic methods for the solution of complex optimization problems, is based on the use of a Genetic Algorithm (GA), coupled with a steady and uniform flow hydraulic module. In particular, this work has focused: on one hand, on the problems of a technical nature posed by the correct sizing of a drainage network; on the other hand, on the possibility to use a simple but nevertheless efficient GA to reach the minimal cost solution very quickly. The suitability of the approach is tested with reference to small and large scale drainage networks, already considered in the literature.

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1. Introduction

The problem of the optimal design of rural drainage channels can be approached from two distinct points of view, namely the optimal design of a single channel and the optimal design of an entire channel network. Historically, due to the lack of computers and adequate numerical techniques, the optimization of the single channel's shape and design has been considered first, and useful analytic solutions can be found in classic hydraulic engineering texts [1]. Despite the precocious availability of these results, researchers have also considered this theme recently. Guo and Hughes [2] presented an analytical procedure for the determination of the best configuration for a trapezoidal cross section of a single channel, able to minimize both frictional resistance and construction cost, taking into account the freeboard and bank slope. Mironenko *et al.* [3] studied the design of channels with parabolic cross-section. Loganathan [4] presented optimal conditions for a parabolic channel cross section accounting for freeboard and limitations on the velocity and channel sizes. Froehlich [5] used the Langrange's multiplier method to determine optimal channel cross sections, incorporating in his formulation of the optimization problem, as additional constraints, both limited flow top width and depth. Monadjemi [6] used Langrange's multipliers method to find the best hydraulic cross section area for different channel shapes. In particular, he solved the problem of optimizing the lining costs, and found that the minimization of the wetted perimeter and the minimization of the cross section area are mathematically equivalent. Swamee *et al.* [7,8] proposed an approach for optimal open channel design where seepage losses were also considered. Das [9] proposed an optimization model for the design of trapezoidal channels, which considers the flooding probability; the same author [10] proposed an optimization strategy to design open channels with composite lining along the perimeter. Jain *et al.* [11] considered spatial variations of the velocity across a proposed composite channel cross section, and approximated the solution to this problem using a Genetic Algorithm (GA). Chahar [12] faced the design of parabolic cross section channels using a nonlinear unconstrained optimization method. More recently, Reddy and Adarsh [13] used a Genetic Algorithm (GA) as Particle Swarm Optimization (PSO) to optimally design a composite trapezoidal irrigation channel.

Of course, in practical applications it is important to consider the optimal design of an entire drainage network consisting of multiple channels. With reference to this topic, few studies about the optimization of free surface rural drainage networks are available, while interest of researchers has been focused mainly on the optimal design of drainage networks. Despite their specific characteristics, there is an obvious conceptual link between these two problems. For this reason, and due to the scarcity of contributions on the topic of rural drainage network optimization, the literature available in the field of urban drainage networks will be also considered here. While numerous works focus on the optimal layout of urban drainage networks [14–23], the majority of research results concerns the optimal channel sizing of a network whose layout is already known. In other cases, the optimization procedures were oriented to solve more general problems. For instance, Lee *et al.* [24] proposed a methodology for efficient rehabilitation of sewer systems; Chill and Mays [25] and Zhang *et al.* [26] proposed different procedures to determine the optimal locations to place various types of developments in a watershed to reduce the negative impacts of urbanization on watershed stormwater systems, and then changes in flow rates and volume from natural to developed conditions; Oxley and Mays [27] proposed an optimization model, based upon the simulated annealing method, to optimize the size and location of detention basin systems including the outlet structures subject to design constraints. An interesting review of the optimal design procedures available for sewer networks has been made by Guo *et al.* [28].

Generally speaking, the techniques proposed for the optimal sizing of drainage networks differ by:

- the choice of the decision variables (longitudinal slopes, ground elevations, crown elevations, *etc.*);
- the constraints used during the optimization procedure;
- one or more Objective Functions (OF) considered within the optimization procedure;
- the optimization algorithm used;
- the hydraulic model used to evaluate the performances of the drainage network;
- the model used to evaluate the discharges through the network.

Classical nonlinear optimization methods, based on gradient techniques, are not satisfactory when applied to the optimal drainage network design problem, because they have a tendency to get stuck in local optima while searching for global solutions in a non-convex discrete search space. As a result of developments in Artificial Intelligence and Operation Research, different alternative optimization techniques, such as the Evolutionary Computation approaches, have emerged during the last 30 years. With reference to the ability to achieve fast results, Wang *et al.* [29] made a comparison between GA [30–34], Particle Swarm Optimization [35] and Ant Colony Algorithm [36–38], showing that the Ant Colony methods require minor computational burden. Afshar *et al.* [39] used Cellular Automata approaches, obtaining results comparable to other methods but with higher computational efficiency. Conversely, GA allows obtaining the most accurate solution [32]: this class of algorithms is very robust in handling complex problems that display large variability and intermittency in input parameters and a large degree of nonlinearity in functional relationships [40,41].

In this paper, we propose a GA procedure aiming at the optimal design of rural drainage networks, which enables the network channels to convey the required discharges with minimum construction and maintenance costs, achieving the best compromise between the numerous technical conflicting requirements. In order to develop the main structure of the optimization procedure, the network hydraulic performance is evaluated by means of a very simple hydraulic model, based on a uniform and steady state stage discharge formula, and the *a priori* knowledge of discharges flowing through each link of the network. However, these assumptions can be easily relaxed, considering realistic hydraulic simulators, coupled with hydrological models able to evaluate the surface runoff to the channel network [42,43].

Besides the main objective of providing a general methodology for the optimal sizing of rural drainage network channels, additional objectives are considered in this paper, namely:

- exploring the influence, on the optimal design of the network, of the value assigned to the invert elevation of the network ending node;
- the analysis of the influence of the technical constraint which imposes, at each junction node of the network, that the size of the channel downstream is not smaller than that of the channels upstream;
- exploring the influence of the mutation probability, which is a GA parameter to be tuned in order to achieve good solutions [44–46].

In the following sections, the problem of the optimal rural drainage network design is formulated, the assumptions made are described, and the optimization model is briefly recalled. Then, two case studies are presented and analyzed. Finally, a discussion of the results obtained is carried out, and general conclusions are drawn.

2. Methods

2.1. Problem Formulation

In practical cases, the problem of the rural drainage network design can have many competing solutions, and a criterion should be defined in order to choose a solution that is optimal. In the present case, we define the optimal network that minimizes the construction cost, and the OF is defined accordingly. The optimization process needs much input data, such as the layout of the system, the ground elevation at the network nodes, the location of the network outlet, the unit costs for construction, the shape of the cross sections, the range of variability of the decision variables, and the flow discharges through the network channels. Feasible solutions should satisfy a set of constraints, in order to take into account physical limitations, technical standards and good engineering practices.

With reference to Figure 1, the constraints that can be considered are summarized as follows:

c1: if h is the water depth corresponding to the design discharge Q, the design filling degree is defined as $\delta = h/(H_{exc} - c)$, where H_{exc} is the excavation depth and c is the ground subsidence. Overflow of the channels should be avoided: this constraint is represented by the condition $\delta \leq \delta_{max}$, where $\delta = 1 - f_b/(H_{exc} - c)$, and f_b is a convenient freeboard. The design discharge is defined as $Q = Q_{T_2}$, where $T_2 = 10 \div 20$ years is the design return period.

c2: a maximum excavation depth $H_{exc,max}$ has to be considered in order to limit the excavation costs and to avoid excessive drainage of sub-surface flow, with subsequent need for irrigation.

c3: in order to reduce the construction costs, the erosion of non-lined channels bottom and banks should be controlled, taking into account the effects of moderate return period flows Q_f. A criterion based on the definition of a threshold velocity V_{er} can be used to evaluate the start of erosion: if V_f is the velocity corresponding to the frequent flow discharge Q_f, the constraint is expressed as $V_f \leq V_{er}$. For the evaluation of V_{er}, the approach proposed by USDA [47] can be used, while $Q_f = Q_{T_1}$ is the flow corresponding to a moderate return period $T_1 \leq T_2$.

c4: sediment deposition should be avoided during flow conditions that have a frequency higher than $3 \div 6$ times per year. If V_{vf} is the velocity corresponding to the very frequent flow discharge Q_{vf}, the constraint is expressed as $V_{vf} \geq V_{dep}$. The limit velocity V_{dep} is a function of the diameter of the particles carried by flow, while $Q_{vf} \cong \left(\frac{1}{15} \div \frac{1}{10}\right) Q_{T_2}$.

c5: a sufficient freeboard f_{cr}, equal to the thickness of the crop-roots layer, has to be considered in order to protect crop even during flow conditions that have a frequency higher than $3 \div 6$ times per year. If h_{vf} is the water depth corresponding to Q_{vf}, and $\delta_{vf} = h_{vf}/(H_{exc} - c)$ is the filling degree corresponding to Q_{vf}, this constraint is expressed as $\delta_{vf} \leq \delta_{cr}$, where $\delta_{cr} = 1 - f_{cr}/(H_{exc} - c)$.

c6: at each node of the network, the dimensions of the channel downstream should not be smaller than those of the channels upstream [48,49]).

With reference to a network made up of N_r reaches and N_n nodes, let Ω_r be the set of the N_r reaches, Ω_n the set of the N_n nodes, and $\Omega_{up}(j)$ the set of the reaches whose downstream end coincides with the upstream end of the generic reach $j \in \Omega_r$. For first order channels, the set $\Omega_{up}(j)$ is empty. The problem of the optimal rural network design is formulated as the minimization of the following OF:

$$OF = \sum_{j \in \Omega_r} C_j\left(CS(j), Z_{exc}(j, up(j)), Z_{exc}(j, up(j))\right) \quad (1)$$

where C_j is the construction cost of the channel j, $CS(j) = \lfloor C_1(j) \quad C_2(j) \quad \ldots \quad CS_{N_{sp}}(j) \rfloor$ is the vector of the channel's geometric characteristics, $up(j)$ and $dw(j)$ are the upstream and downstream end nodes of the channel j, $Z_{exc}(j,n)$ is the bottom elevation at the end n of the channel j. In particular, the cost OF of the network is the sum of C_{exc} and C_{lin}, where C_{exc} refers to the cost of excavation, waste transport and landfill, while C_{lin} refers to the lining cost. In order to evaluate C_{exc}, the scheme of the trench considered in the calculations is shown in Figure 1.

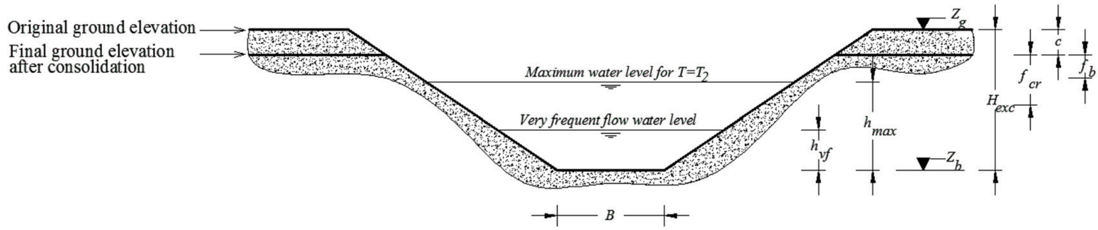

Figure 1. Rural drainage networks: definition sketch of the symbols used.

The OF is subject to the following constraints:

c1:	$\delta(j,n) \le \delta_{\max}(j,n)$	$\forall j \in \Omega_r, \quad n = up(j), dw(j)$	(2)
c2:	$H_{exc}(j,n) \le H_{exc,\max}(j,n)$	$\forall j \in \Omega_r, \quad n = up(j), dw(j)$	(3)
c3:	$V_f(j,n) \le V_{er}(j,n)$	$\forall j \in \Omega_r, \quad n = up(j), dw(j)$	(4)
c4:	$V_{vf}(j,n) \ge V_{dep}(j,n)$	$\forall j \in \Omega_r, \quad n = up(j), dw(j)$	(5)
c5:	$\delta_{vf}(j,n) \le \delta_{cr}(j,n)$	$\forall j \in \Omega_r, \quad n = up(j), dw(j)$	(6)
c6:	$CS_i(j) \ge CS_i(k)$	$i = 1,2,\ldots,N_{sp}, \quad \forall j \in \Omega_r, \quad \forall k \in \Omega_{up}(j)$	(7)

Though more general approaches and numerical models may be applied [50–59], in this work, for the sake of simplicity, in order to show the potential of the approach proposed for the optimal sizing of the drainage network, the actual hydraulic behavior of the whole network is neglected, and the performance of each channel is evaluated only by means of an appropriate state stage-discharge formula corresponding to uniform and steady state conditions. In particular, the Manning's equation $V = n_M^{-1} R^{2/3} i^{1/2}$ is adopted, where n_M is the Manning coefficient, R is the hydraulic radius, $i = \sin[\tan^{-1}(s)]$, and s is the channel's longitudinal slope.

2.2. The Genetic Algorithm

The Genetic Algorithm implemented by the authors has been described in Palumbo *et al.* [60]. For this reason, it will be only briefly depicted in this section. GAs are a class of heuristic techniques, inspired by the biological concepts of natural evolution and selection of individuals, which are used to sample the search space, in order to approximate the optimal solution. The *candidate solutions* of the optimization problem, called *individuals*, differ by their appearance (phenotype), *i.e.*, by the value of the decision variables. The phenotype is coded as a genotype string, which is in turn formed by sub-strings, each representing the binary Gray coding of the decision variables. The individual characteristics determine the individual's *Fitness Function* (FF) value, which depends both on the OF value related to the phenotype and on the degree of satisfaction of constraints.

At the beginning, an initial *population* of N individuals is randomly generated. The individuals are ranked in increasing order, according to their fitness, and a selection probability, which decreases with the ranking order, is assigned to each individual. Finally, the individuals are picked, according to their selection probability, and accumulated in a "mating pool", in order to form couples of parents of the subsequent generation individuals. In this work, "exponential ranking" is used to select the individuals to be inserted in the mating pool for the subsequent steps of the GA processes. After "selection", other operators can be introduced, namely "crossover", "mutation", and "elitism". When the decision variables satisfy the problem constraints, the FF value coincides with the OF. Conversely, the FF value is calculated by adding penalization terms to the OF value when one or more constraints are not satisfied. This mechanism biases the selection in favor of those individuals that satisfy the constraints.

In this work, trapezoidal cross sections with fixed bank slope are adopted, and then the vector $CS(j)$ degenerates to the bottom width $B(j)$ of the channel j. The trench bottom elevation continuity is considered at the nodes of the rural drainage network:

$$H_{exc}(j,up(j)) = H_{exc}(k,dw(k)) \qquad \forall j \in \Omega_r, \quad \forall k \in \Omega_{up}(j) \tag{8}$$

Under these hypotheses, the phenotype of a candidate network is completely characterized by a vector containing the height of the trench H_{exc}^{nen} at the downstream end of the network and, for each reach, the slope s of the channel together with the bottom width B.

The actual form of the FF adopted is the following:

$$\begin{aligned} FF = OF &+ p_{fb} \sum_{j\in\Omega_r} \max\left\{0, \max_{n=up(j),dw(j)} \left\{\delta(j,n) - \delta_{\max}(j,n)\right\}\right\} + \\ p_{exc} \sum_{j\in\Omega_r} \max\left\{0, \max_{n=up(j),dw(j)} \left\{H_{exc}(j,n) - H_{exc,\max}(j,n)\right\}\right\} &+ p_{er} \sum_{j\in\Omega_r} \max\left\{0, \max_{n=up(j),dw(j)} \left\{V_f(j,n) - V_{er}(j,n)\right\}\right\} + \\ p_{dep} \sum_{j\in\Omega_r} \max\left\{0, \max_{n=up(j),dw(j)} \left\{V_{dep}(j,n) - V_{vf}(j,n)\right\}\right\} &+ p_{cr} \sum_{j\in\Omega_r} \max\left\{0, \max_{n=up(j),dw(j)} \left\{\delta_{vf}(j,n) - \delta_{cr}(j,n)\right\}\right\} + \\ &p_{sz} \sum_{j\in\Omega_r} \max\left\{0, \max_{k\in\Omega_{dw}(up(j))} \left\{B(l) - B(k)\right\}\right\} \end{aligned} \tag{9}$$

In Equation (9), the symbols p_{fb}, p_{exc}, p_{er}, p_{dep}, p_{cr} and p_{sz} represent the unit penalties corresponding to the constraints of Equations (2)–(7), respectively.

The following GA parameters have been used during the numerical experiments: N = 300 individuals during each generation; I = 5000 generations; crossover probability c_p = 1; N_e = 5 individuals preserved by the elitism operator; the values of the unit penalization coefficients p_{fb}, p_{exc}, p_{ser}, p_{dep}, p_{cr}, p_{sz} may vary from an application to another, and usually fall in the interval (10^6, 10^{15}) when the relevant constraint is activated, while the value zero is used if the constraint is discarded. The mutation probability m_p is variable in the range (0.01 ÷ 6.0) %.

3. Results and Discussion

The optimization procedure discussed in this work is applied to two case studies. The first application is taken from the existing literature about the drainage networks' optimal design, and is used to test the GA adopted for the optimization. The second application is used to demonstrate the feasibility of the approach for real world applications.

3.1. Genetic Algorithm Verification: World Bank Network (1991)

In the literature, there is a general lack of case studies referring to the optimization of rural drainage networks, while many case studies are available for urban drainage networks. For this reason, the model implemented is easily adapted to solve the problem of the optimal urban drainage network, and then is applied to an urban drainage network with circular pipes taken from the literature [33,61,62].

The network layout is shown in Figure 2. The characteristics of this test case (network geometry, pipe diameters allowed, pipes costs, excavation costs) are summarized by Afshar and Zamani [62], and they are not repeated here. The following constraints are assumed: the maximum filling degree of the pipes is $\delta_{\max} = 0.82$; the maximum excavation depth considered is $H_{exc,\max} = 4.5$ m; the maximum allowed flow velocity is $V_{max} = 2.5$ m/s; the minimum allowed flow velocity is $V_{min} = 0.5$ m/s; the minimum soil cover depth is $H_{cov,\min} = 1.5$ m. A set of $2^9 = 512$ longitudinal slopes is considered in the range (0.01 ÷ 0.08) m/m, with a step equal to 1.36986×10^{-4} m/m. Finally, the diameters considered in the calculations are $2^4 = 16$.

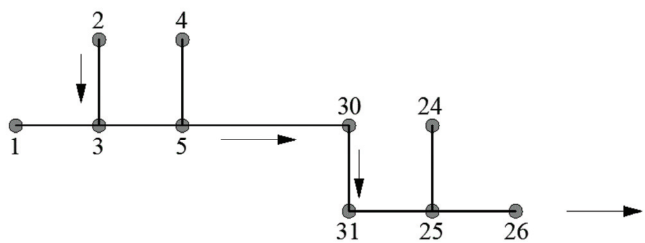

Figure 2. World Bank (1991) [61] case study. Layout.

The mutation probability m_p must be intended here as the number N_{bm} of bits involved in the mutation process, divided by the total number N_{bt} of bits which constitute the genotype of the generic individual. Different analyses are performed in order to evaluate how the optimization process is influenced by the values assigned to the network ending node excavation H_{exc}^{nen} and to the mutation probability m_p. Aiming at this, two sets of runs are considered:

- Case WB-1: H_{exc}^{nen} is not a decision variable, and its value is taken equal to 2.00 m;
- Case WB-2: H_{exc}^{nen} is left free to vary in the range (0.45 ÷ 2.00) m with step 0.05 m.

For each set of runs, the algorithm is restarted using different initial populations, in order to assess the robustness of the optimization model outcome, and considering variable values of the mutation probability m_p.

The results obtained for the case WB-1 are summarized in Table 1.

In particular, the information reported in the generic row are as follows: the number N_{bm} of bits involved in the mutation process, the optimal cost obtained for different initial populations (*Pop1*, *Pop2*, …) with fixed N_{bm}, the minimum cost obtained (*Min*), the maximum cost (*Max*), the average cost (*Ave*), and the Root Mean Square error (*RMS*) of the costs. Note that the solutions are not penalized: the constraints are satisfied, and OF coincides with *FF*. The best solution is

OF = 199,088.63, and it is obtained for N_{bm} = 2, corresponding to m_p = 0.017. It is interesting to observe that the average optimal cost *Ave* attains its minimum value for N_{bm} = 2 as well, while the maximum cost *Max* and the root mean square error *RMS* of the costs are close to their minimum for N_{bm} = 2. This ensures that, for the present application, the most important numerical parameter is m_p: a good choice of m_p leads to reliable solutions.

Table 1. World Bank (1991) [61] case study. Optimal results for the case WB-1.

N_{bm}	*Pop 1*	*Pop 2*	*Pop 3*	*Pop 4*	*Pop 5*	*Min*	*Max*	*Max*	*RMS*
1	199,381.54	208,480.70	221,530.28	199,337.83	199,288.43	199,288.43	221,530.28	205,603.76	4866.32
2	**199,088.63**	199,108.37	199,125.11	199,097.66	199,097.79	**199,088.63**	199,125.11	**199,103.51**	8.68
3	199,095.89	199,108.37	199,166.08	199,118.22	199,105.76	199,095.89	199,166.08	199,118.87	17.45
4	199,109.00	199,105.76	199,108.50	199,097.52	199,111.85	199,097.52	**199,111.85**	199,106.53	**8.30**
5	199,098.83	199,124.74	199,128.56	199,245.57	199,169.35	199,098.83	199,245.57	199,153.41	36.96
6	199,158.12	199,213.63	199,235.26	199,242.11	199,154.04	199,154.04	199,242.11	199,200.63	52.83
7	199,324.87	199,383.58	199,599.30	199,287.05	199,247.62	199,247.62	199,599.30	199,368.48	136.85

The results obtained for the case *WB-2* are summarized in Table 2. Again, no optimal solution is penalized: the best value for the objective function is OF = 199,088.63 and it is found for N_{bm} ranging between 2 and 4, corresponding to $m_p \in (0.013 \div 0.027)$. The functions *Ave*, *Max* and *RMS* attain their minimum values in the same range.

Table 2. World Bank (1991) [61] case study. Optimal results for the case WB-2.

N_{bm}	*Pop 1*	*Pop 2*	*Pop 3*	*Pop 4*	*Pop 5*	*Min*	*Max*	*Ave*	*RMS*
1	202,802.76	199,320.22	199,289.02	199,312.14	199,299.74	199,299.74	202,802.76	200,004.78	747.88
2	**199,088.63**	199,098.47	199,183.11	**199,088.63**	199,098.47	**199,088.63**	199,183.11	199,111.46	19.10
3	199,105.76	**199,088.63**	**199,088.63**	199,128.11	199,135.48	**199,088.63**	**199,128.11**	199,109.32	12.72
4	199,095.89	**199,088.63**	199,129.97	199,111.93	199,118.18	**199,088.63**	199,129.97	**199,108.92**	**11.27**
5	199,136.47	199,240.47	199,139.26	199,202.66	199,089.27	199,089.27	199,240.47	199,161.62	40.45
6	199,199.19	199,206.59	199,133.30	199,123.84	199,220.07	199,199.19	199,220.07	199,176.60	43.20
7	199,180.98	199,145.15	199,170.09	199,114.00	199,227.68	199,145.15	199,227.68	199,167.58	39.16
8	199,198.38	199,201.65	199,264.49	199,203.22	199,822.13	199,198.38	199,822.13	199,337.97	155.81
9	199,260.81	199,304.59	199,396.04	199,297.51	199,267.17	199,260.81	199,396.04	199,305.23	99.26
10	199,258.31	199,326.11	199,963.77	199,972.76	199,318.42	199,258.31	199,963.77	199,567.88	259.66

In Table 3, the results obtained for this set of runs are compared with those obtained by other authors.

Table 3. World Bank (1991) case study. Optimal results obtained by various researchers.

Model	**Cost ($)**
SEWER (World Bank 1991) [62]	199,480
Afshar and Zamani (2002) [63]	199,320
Afshar *et al.* (GA-TRANS2, 2006) [36]	199,244
Proposed Model	**199,088.63**

By inspection of the results listed in the Tables 1–3, it is possible to state that:

- the best result obtained for this test case is better than those found by previous authors (Table 3);
- for this test case, there is no difference between the best results obtained considering H_{exc}^{nen} fixed and equal to 2.00 m, or left free to vary in the range (0.45 ÷ 2.00) m;
- the best solutions for OF are obtained for N_{bm} ranging in the interval (2 ÷ 4), which corresponds to m_p ranging approximately in the interval (0.013 ÷ 0.027). This result is in agreement with the values of m_p often suggested in the GA literature, with reference to hydraulic engineering applications [28,63];
- the functions *Ave*, *Max* and *RMS* attain their minimum values in the same range of m_p where OF is minimized. This fact ensures the reliability of the optimal solution found.

The characteristics of the optimal network obtained with the proposed approach are listed in Table 4. It is interesting to observe that, in the case under examination, the constraint c_6 (no decreasing size of the channel in the downstream direction) is automatically satisfied and then superfluous.

Table 4. World Bank (1991) [61] case study. Optimal decision variables and hydraulic characteristics.

Branch	Crown Elevation (m)		Diameter	Slope	Velocity	Filling Degree
	Upstream	Downstream	(mm)	(m/m)	(m/s)	(m/m)
1–3	1394.5963	1387.0884	150	0.072	2.063	0.456
2–3	1393.8938	1387.0884	250	0.028	2.057	0.624
3–5	1385.4855	1380.2767	300	0.027	2.307	0.684
4–5	1376.6060	1374.4658	150	0.076	2.499	0.739
5–30	1387.0884	1380.2767	300	0.030	2.453	0.674
30–31	1380.2767	1378.3178	450	0.018	2.496	0.711
31–25	1378.3178	1377.4986	450	0.018	2.496	0.711
24–25	1377.4986	1374.4658	450	0.017	2.437	0.727
25–26	1374.4658	1371.0000	500	0.016	2.494	0.681

3.2. Case Study: Biggiero and Pianese Network (1996)

The model is applied to a case study available in the literature [64,65], which is used to demonstrate the feasibility of the approach for real world applications. The test considered is a rural drainage network consisting of 37 reaches, whose total length is 8310 m, and 38 nodes (Figure 3). The characteristics of the network are reported in Table 5. For the sake of simplicity, though without loss of generality, the value of the frequent discharge Q_f has been taken equal to the value of the very frequent discharge Q_{vf}.

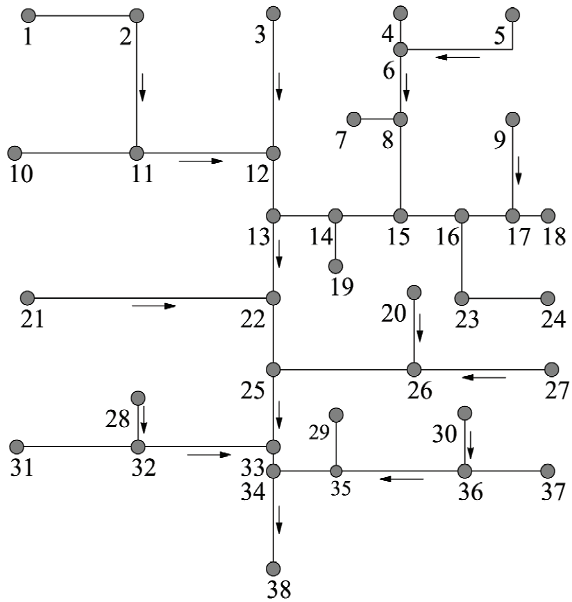

Figure 3. Biggiero and Pianese (1996) [64] case study. Layout.

Table 5. Biggiero and Pianese (1996) [64] case study. Geometric and hydraulic characteristics of the problem.

Branch	Ground Elevation (m)		Horizontal Length	Q	$Q_f \equiv Q_{vf}$
	Upstream	Downstream	(m)	(m^3/s)	(m^3/s)
1–2	13.604	13.204	200	0.10373	0.010373
2–11	13.204	12.204	400	0.19977	0.019977
10–11	12.654	12.204	250	0.14310	0.014310
11–12	12.204	11.694	300	0.44535	0.044535
3–12	12.454	11.694	400	0.15754	0.015754
4–6	12.819	12.534	150	0.095607	0.0095607
5–6	13.129	12.534	350	0.15382	0.015382
6–8	12.534	12.160	220	0.30989	0.030989
7–8	12.320	12.160	100	0.051418	0.0051418
8–15	12.160	11.840	200	0.41000	0.041000
18–17	12.285	12.173	70	0.049872	0.0049872
9–17	12.515	12.173	190	0.096821	0.0096821
17–16	12.173	12.008	110	0.16984	0.016984
24–23	12.408	12.138	180	0.079993	0.0079993
23–16	12.138	12.008	260	0.12276	0.012276
16–15	12.008	11.840	120	0.32731	0.032731
15–14	11.840	11.645	150	0.76748	0.076748
19–14	11.705	11.645	150	0.059884	0.0059884

Table 5. *Cont.*

Branch	Ground Elevation (m)		Horizontal Length	Q	$Q_f \equiv Q_{vf}$
	Upstream	Downstream	(m)	(m^3/s)	(m^3/s)
14–13	11.645	11.405	200	0.85356	0.085356
12–13	11.694	11.405	170	0.64189	0.064189
13–22	11.405	10.925	300	1.5406	0.15406
21–22	11.860	10.925	550	0.23869	0.023869
22–25	10.925	10.645	200	1.8285	0.18285
20–26	11.441	11.041	250	0.095221	0.0095221
27–26	11.521	11.041	320	0.14660	0.014660
26–25	11.041	10.645	330	0.32110	0.032110
25–33	10.645	10.370	250	2.1774	0.21774
31–32	11.245	10.820	250	0.12171	0.012171
28–32	11.067	10.820	130	0.093266	0.0093266
32–33	10.820	10.370	300	0.32767	0.032767
37–36	11.011	10.595	320	0.14874	0.014874
30–36	10.791	10.595	140	0.062599	0.0062599
36–35	10.595	10.391	170	0.27880	0.027880
29–35	10.547	10.391	120	0.081949	0.0081949
35–34	10.391	10.270	110	0.37467	0.037467
33–34	10.370	10.270	100	2.4675	0.24675
34–38	10.270	10.000	300	2.8255	0.28255

The cross section shape is assumed trapezoidal, with bottom width B, while the angle between the banks and the horizontal plane is $\alpha = 45°$. The values allowed for B range from 0.30 to 4.00 m, and are reported in Table 6.

Table 6. Biggiero and Pianese (1996) [64] case study. Bottom width B and network ending node excavation. H_{exc}^{nen}: the values.

ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
B (m)	0.30	0.50	0.80	1.00	1.50	2.00	2.50	3.00	3.50	4.00	-	-	-	-	-	-
H_{exc}^{nen} (m)	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.80	0.90	1.00	1.10	1.20	1.30	1.40	1.45	1.50

In order to evaluate the network construction cost, the waste transport and landfill are neglected, while only excavation costs are considered. In particular, the unit excavation costs are equal to 9.97 €/m^3 for $H_{exc} \leq 2.00$ m, and are equal to 10.29 €/m^3 for $H_{exc} > 2.00$ m.

The parameters used for the evaluation of Equations (2), (4) and (5), corresponding to constraints c_1, c_3 and c_5, are chosen as follows: $f_b = 0$ m (and then $\delta_{max} = 1$), $c = 0$ m, $f_{cr} = 0.30$ m. Without loss of generality, the constraints c_2 and c_4 about the maximum excavation and the deposition velocity, respectively, have been discarded. The limit velocity V_{er} is evaluated considering silt gravels, characterized by Plastic Index value $PI = 16$ and porosity $p = 0.35$, while the sediment concentration in the water flowing through the channels is assumed to be equal to 0.7%. Under these assumptions, the approach proposed in USDA [47] allows evaluation of the erosion velocity V_{er} as a function of the water depth h_{vf} corresponding to the very frequent discharge Q_{vf}, using the formula $V_{er} = 2.44 \cdot h_{vf}^{0.19}$.

Four different series of tests are performed:

- Case BP-1A: H_{exc}^{nen} is not a decision variable, and its value is taken equal to 1.50 m, while the constraint c_6 is effective;
- Case BP-1B: H_{exc}^{nen} is not a decision variable, and its value is taken equal to 1.50 m, while the constraint c_6 is discarded;
- Case BP-2A: H_{exc}^{nen} is considered as a decision variable, and it is left free to vary in the range (0.40 ÷ 1.50), while the constraint c_6 is effective;
- Case BP-2B: H_{exc}^{nen} is considered as a decision variable, and it is left free to vary in the range (0.40 ÷ 1.50), while the constraint c_6 is discarded.

In each reach, a set of 2^9 = 512 longitudinal slopes is considered, variable in the range (0.0001 ÷ 0.0064) m/m with step equal to 0.00001233 m/m, while the 2^4 values allowed for the decision variable H_{exc}^{nen} are reported in Table 6. In order to evaluate the FF in Equation (9), the unit penalization coefficients are chosen as follows: $p_{fb} = p_{er} = p_{cr} = 10^9$, and $p_{exc} = p_{dep} = 0$. The value used for the unit penalty coefficient p_{sz} is 10^9 for the cases BP-1A and BP-2A, while it is zero for the cases BP-1B and BP-2B. For each case, the algorithm is restarted from different initial populations (*Pop1*, *Pop2*, …), and considering variable mutation probability values m_p.

The results obtained for the cases BP-1A and BP-1B are reported in Table 7. With reference to the case BP-1A, the best solution is OF = 98,972.09€, and it is obtained for N_{bm} = 5, corresponding to m_p = 0.0075. For the same case, the average optimal cost *Ave* attains its minimum value for N_{bm} = 9, corresponding to m_p = 0.0150, together with the maximum cost *Max* and the root mean square *RMS* of the costs. With reference to the case BP-1B, the best solution is OF = 85,539.03€, and it is obtained for N_{bm} = 5, corresponding to m_p = 0.0075: due to the absence of the constraint about the channel width, a degree of freedom is added, and the best result obtained for the case BP-1B is not greater than the best result for BP-1A. The optimal values for *Ave*, *Max* and *RMS* are obtained for m_p ranging in the interval (0.0075 ÷ 0.0225).

The results for the cases BP-2A and BP-2B are reported in Table 8.

With reference to the case BP-2A, the best solution is OF = 94,343.22€, and it is obtained for N_{bm} = 5, corresponding to m_p = 0.0075: due to the absence of the constraint about the excavation at the network ending node of the network, a degree of freedom is added, and the optimal solution is not greater than that obtained for the case BP-1A. For the same case, *Ave* and *RMS* attain their minimum values for N_{bm} = 5, corresponding to m_p = 0.075, while *Max* is minimized using m_p = 0.015. With reference to the case BP-2B, the best solution is OF = 73,353.32€, and it is obtained for N_{bm} = 9, corresponding to m_p = 0.015: as expected, the best result obtained for the case BP-2B is not greater than the best results for BP-1B and BP-2A. The optimal values for *Ave*, *Max* and *RMS* are obtained for m_p = 0.0075.

Table 7. Biggiero and Pianese (1996) [64] case study. Optimal results for the cases BP-1A and BP-1B.

Case	m_p	N_{bm}	Pop_1	Pop_2	Pop_3	Pop_4	Pop_5	*Min*	*Max*	*Ave*	*RMS*
BP-1A	0.001	1	150,114.97	173,070	141,479.11	145,121.82	135,643.23	135,643.23	173,070	149,085.82	28,989.15
	0.0075	5	122,515.75	100,911.57	109,885.81	108,321.04	98,972.09	**98,972.09**	122,515.75	108,121.25	10,754.46
	0.015	9	121,415.61	108,932.41	101,410.7	99,967.7	102,357.2	99,967.7	**121,415.61**	**106,816.72**	**10,145.37**
	0.0225	14	131,916.7	147,795.94	191,307.27	141,376.08	143,637.42	131,916.7	191,307.27	151,206.68	30,785.86
	0.03	19	153,819.02	145,978.72	170,365.79	209,552.47	134,984.9	134,984.9	209,552.47	162,940.18	36,507.51
	0.0375	24	209,401.27	152,818.34	221,438.39	216,116.22	157,821.18	152,818.34	221,438.39	191,519.08	49,230.73
BP-1B	0.001	1	104,690.99	96,109.18	132,357.17	110,152.17	122,785.97	96,109.18	**132,357.17**	113,219.1	12,986.66
	0.0075	5	135,483.19	100,798.7	91,897.17	85,539.03	93,541.7	**85,539.03**	135,483.19	**101,451.96**	10,159.75
	0.015	9	87,205.04	106,330.03	101,287.47	103,712.54	135,992.85	87,205.04	135,992.85	106,905.58	11,343.42
	0.0225	14	106,298.84	90,893.8	109,810.09	99,335.51	96,656.07	90,893.8	109,810.09	100,598.86	**6710.11**
	0.03	19	109,417.35	100,446.6	108,456.51	103,556.18	99,369.49	99,369.49	109,417.35	104,249.23	7837.56
	0.0375	24	126,757.81	104,155.58	101,745.82	102,967.66	104,154.92	101,745.82	126,757.81	107,956.36	10,195.95

Table 8. Biggiero and Pianese (1996) [64] case study. Optimal results for the cases BP-2A and BP-2B.

Case	m_p	N_{bm}	Pop_1	Pop_2	Pop_3	Pop_4	Pop_5	*Min*	*Max*	*Ave*	*RMS*
BP-2A	0.001	1	114,994.54	129,701.43	123,222.31	121,269.94	110,152.71	110,152.71	129,701.43	119,868.19	11,874.28
	0.0075	5	99,147.22	111,655.69	98,806.23	104,279.08	94,343.22	**94,343.22**	111,655.69	**101,646.29**	**4255.66**
	0.015	9	104,551.42	99,804.07	108,322.84	106,931.04	104,584.26	99,804.07	**108,322.84**	104,838.73	4935.65
	0.0225	14	131,596.06	142,020.01	126,446.31	137,005.42	148,466.35	126,446.31	148,466.35	137,106.83	19,500.35
	0.03	19	149,737.04	132,467.33	197,257.66	157,626.5	198,950.1	132,467.33	198,950.1	167,207.72	34,740.54
	0.0375	24	205,444.33	204,105.65	176,471.38	181,740.54	264,992.84	176,471.38	264,992.84	206,550.95	52,179.77
BP-2B	0.001	1	105,278.18	88,998.65	100,389.65	109,368.9	116,342.27	88,998.65	**116,342.27**	104,075.53	14,338.91
	0.0075	5	84,640.15	77,488.21	79,381.09	77,382.45	90,499.754	77,382.45	90,499.75	**81,878.33**	**4431.97**
	0.015	9	82,917.73	73,353.32	74,360.86	92,763.82	87,478.171	**73,353.32**	92,763.82	82,174.78	5172.12
	0.0225	14	88,965.44	86,126.22	82,616.31	101,479.54	125,238.16	82,616.31	125,238.16	96,885.14	12,610.8
	0.03	19	85,571.29	96,805.84	91,319.61	83,952.82	97,789.09	83,952.82	97,789.09	91,087.73	8322.36
	0.0375	24	99,792.81	112,730.92	94,426.29	99,286.95	111,468.22	94,426.29	112,730.92	103,541.04	13,883.82

The optimal network characteristics are reported in Table 9 for all the cases examined. From the inspection of this Table, it is clear that the optimal decision variables are strongly sensitive to the constraints applied. For instance, with reference to the network ending reach 34–38, its bottom width B lies in the range (1.00 ÷ 1.50) m, depending on the case examined. The same is true for the first order channels. For example, the bottom width B of reach 1–2 lies in the range (0.30 ÷ 0.50) m, while the slope lies in the range (0.00145 ÷ 0.00247) m/m.

By exploring the results listed in the Tables above, it is possible to draw the following observations:

- the optimal results depend strongly on the constraints that are applied. In particular, the optimal result of the most constrained case (BP-1A) is 35% greater than that of less constrained case (BP-2B);
- when the constraint c_6 is not explicitly enforced (cases BP-1B and BP-2B), it may happen (Table 9) that the channel bottom width decreases downstream, despite the increase of the design discharge *Q*. This is true when the decrease of the channel width is sufficient to compensate, from an economical point of view, the increase of the channel longitudinal slope;
- differently from the World Bank case study, there is a significant difference between the cases of H_{exc}^{nen} fixed or variable in a range. As expected, the optimal results for the cases BP-2A and *BP-2B* are not greater than those related to the cases BP-1A and BP-1B;
- the best solutions for *OF*, *Ave*, *Max* and *RMS* are obtained for m_p ranging in the interval (0.0075 ÷ 0.0225), and again this result is in agreement with the values of m_p often suggested in the GA literature.

Comparing the best solution cost obtained, in this work, for the case BP-2A, in which the technical constraint c_6 is effective, with the cost of the network considered in [64], obtained using the same unit costs and value of H_{exc}^{nen} (H_{exc}^{nen} = 1.4 m) (see the following Table 10 and Figure 4, in which the geometric characteristics reported in [64] and the geometric characteristics obtained for the case BP-2A have been reported), it is possible to observe that the minimum cost network obtained by the proposed optimization procedure is € 94,343.22/€ 275,339.25 = 34.3% of the cost of original network, designed just to be effective from a technical point of view, but without considering the need to reduce the intervention costs. In order to show the convergence properties of the presented approach, the behavior of the fitness function for the case BP-2A has been reported in Figure 5.

Table 9. Biggiero and Pianese (1996) [64] case study. Optimal decision variables.

Reach	Case BP-1A		Case BP-1B		Case BP-2A		Case BP-2B	
	B	s	B	s	B	s	B	s
	(m)	(m/m)	(m)	(m/m)	(m)	(m/m)	(m)	(m/m)
1–2	0.5	0.00195	0.3	0.00247	0.3	0.00179	0.3	0.00145
2–11	0.8	0.00308	0.3	0.00237	0.5	0.00248	0.5	0.00267
10–11	0.3	0.00254	0.3	0.0018	0.8	0.00188	0.5	0.00227
11–12	0.8	0.00311	0.8	0.00315	1	0.00262	0.7	0.00177
3–12	0.5	0.00354	0.3	0.00303	0.3	0.00257	0.3	0.00194
4–6	0.3	0.00195	0.3	0.00382	0.8	0.00349	1.3	0.00334
5–6	0.3	0.00172	0.3	0.00247	0.3	0.00215	0.3	0.00207
6–8	0.5	0.00274	0.3	0.00279	0.8	0.00116	0.5	0.00154
7–8	0.3	0.00469	0.3	0.00629	0.8	0.00276	0.4	0.00111
8–15	0.5	0.00262	0.3	0.0013	0.8	0.00591	0.5	0.00246
18–17	0.8	0.00281	0.3	0.00328	0.3	0.00365	0.5	0.00023
9–17	0.8	0.00232	0.3	0.00215	0.3	0.00379	0.4	0.00131
17–16	0.8	0.00343	0.3	0.00455	0.3	0.00257	0.5	0.00277
24–23	0.8	0.00181	0.8	0.00157	0.8	0.00121	0.4	0.0018
23–16	0.8	0.00154	0.8	0.00223	0.8	0.00249	0.3	0.00111
16–15	0.8	0.00291	0.8	0.00174	0.8	0.00515	0.3	0.00188
15–14	1.5	0.00047	0.3	0.00303	1	0.00019	0.6	0.00228
19–14	0.8	0.00297	0.3	0.0055	0.3	0.00576	0.3	0.00278
14–13	1.5	0.00237	0.3	0.0012	1	0.00123	0.5	0.00149
12–13	0.8	0.00132	0.3	0.00319	1	0.00456	0.7	0.0039
13–22	1.5	0.00147	0.8	0.00139	1	0.00158	0.8	0.00203
21–22	0.3	0.00253	0.8	0.00292	0.3	0.00301	0.3	0.00274
22–25	1.5	0.00306	0.8	0.00211	1	0.00112	1.5	0.00091
20–26	0.3	0.00158	0.3	0.0017	0.3	0.0025	0.3	0.00145
27–26	0.5	0.00151	0.3	0.00149	0.3	0.00211	0.4	0.00127
26–25	0.5	0.00376	0.8	0.00354	0.8	0.00268	0.5	0.00264
25–33	1.5	0.00155	0.8	0.00159	1	0.00167	1.1	0.00217
31–32	0.3	0.00165	0.8	0.00226	0.5	0.00174	0.4	0.00196
28–32	0.8	0.00207	0.3	0.00276	0.3	0.00192	0.5	0.0027
32–33	0.8	0.00471	0.3	0.00421	0.8	0.00432	0.3	0.00388
37–36	0.5	0.00137	0.3	0.00141	0.3	0.00222	0.4	0.00122
30–36	0.3	0.0018	0.3	0.00223	0.3	0.00387	0.3	0.00223
36–35	0.5	0.00501	0.3	0.00472	0.3	0.00164	0.3	0.00443
29–35	0.8	0.00629	0.3	0.00623	0.8	0.00482	0.3	0.00399
35–34	0.8	0.00483	0.8	0.00462	0.8	0.00639	0.3	0.00281
33–34	1.5	0.00216	0.3	0.00252	1	0.00223	1	0.00137
34–38	1.5	0.00094	0.8	0.001	1	0.00101	1.1	0.00111
H_{exc}^{nen} (m)	1.5		1.5		1.4		1.3	

Table 10. Geometric characteristics reported in Biggiero and Pianese (1996) [64] *vs.* geometric characteristics obtained for the case BP-2A.

Reach	Biggiero&Pianese (1996)		Case BP-2A	
	B	s	B	s
	(m)	(m/m)	(m)	(m/m)
1–2	0.5	0.00200	0.3	0.00179
2–11	0.5	0.00250	0.5	0.00248
10–11	0.5	0.00180	0.8	0.00188
11–12	0.8	0.00170	1.0	0.00262
3–12	0.8	0.00190	0.3	0.00257
4–6	0.5	0.00190	0.8	0.00349
5–6	0.8	0.00170	0.3	0.00215
6–8	0.8	0.00170	0.8	0.00116
7–8	0.5	0.00160	0.8	0.00276
8–15	1.0	0.00160	0.8	0.00591
18–17	0.5	0.00160	0.3	0.00365
9–17	0.5	0.00180	0.3	0.00379
17–16	0.5	0.00150	0.3	0.00257
24–23	0.5	0.00150	0.8	0.00121
23–16	0.8	0.00050	0.8	0.00249
16–15	0.8	0.00140	0.8	0.00515
15–14	1.5	0.00130	1.0	0.00019
19–14	0.5	0.00040	0.3	0.00576
14–13	1.5	0.00120	1.0	0.00123
12–13	1.5	0.00170	1.0	0.00456
13–22	2.0	0.00160	1.0	0.00158
21–22	1.0	0.00170	0.3	0.00301
22–25	2.0	0.00140	1.0	0.00112
20–26	0.5	0.00160	0.3	0.0025
27–26	0.8	0.00150	0.3	0.00211
26–25	1.0	0.00120	0.8	0.00268
25–33	2.0	0.00110	1.0	0.00167
31–32	0.5	0.00170	0.5	0.00174
28–32	0.5	0.00190	0.3	0.00192
32–33	1.0	0.00150	0.8	0.00432
37–36	0.8	0.00130	0.3	0.00222
30–36	0.5	0.00140	0.3	0.00387
36–35	0.8	0.00120	0.3	0.00164
29–35	0.5	0.00130	0.8	0.00482
35–34	1.0	0.00110	0.8	0.00639
33–34	2.5	0.00100	1.0	0.00223
34–38	2.5	0.00090	1.0	0.00101
H_{exc}^{nen} (m)	1.4		1.4	

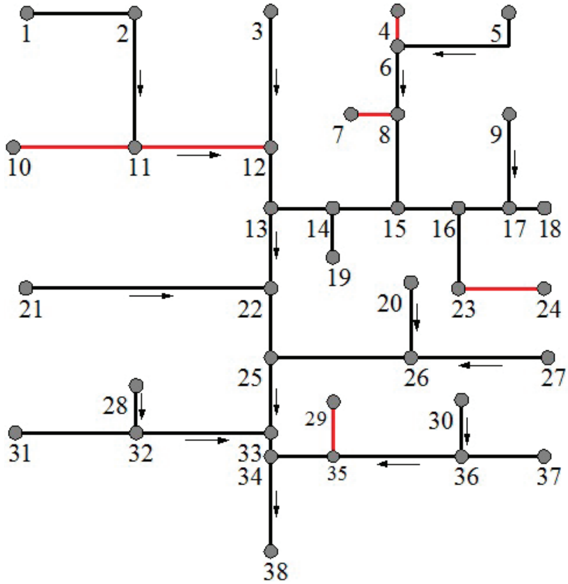

Figure 4. Biggiero and Pianese (1996) [64] *vs.* case BP-2A. Layout.

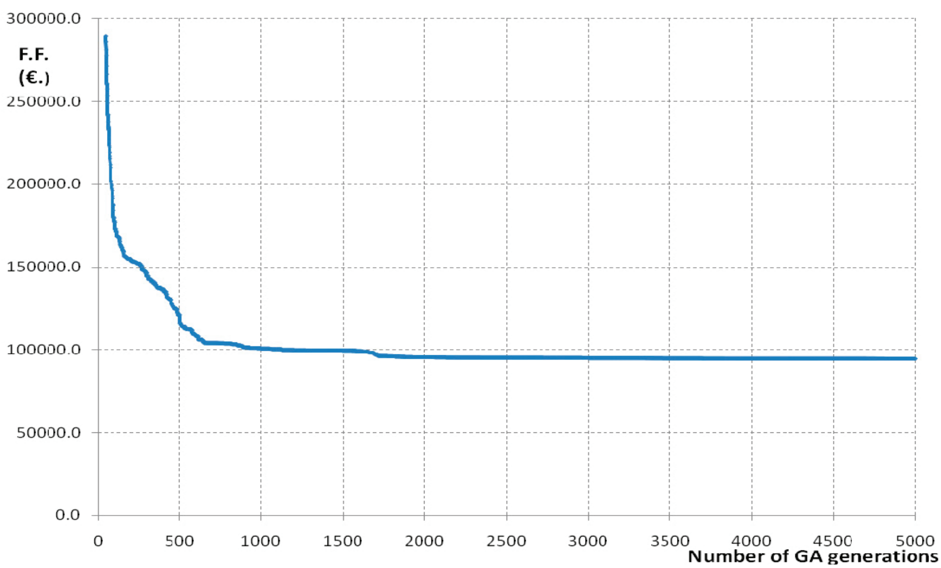

Figure 5. The behavior of fitness function for the case BP-2A.

4. Conclusions

In this work, an automated tool for the optimal design of rural drainage networks is proposed and its application and effectiveness are demonstrated. The optimization procedure makes use of a *GA* for the choice of the channels' geometric characteristics that minimize the construction cost, while a uniform flow stage–discharge formula is used to evaluate the hydraulic performance of the channels and the degree of satisfaction of constraints.

Two case studies are considered. The first application, taken from the literature about the optimal design of urban drainage networks, is used to demonstrate the ability of the *GA* to approximate the optimal solution of the drainage network problem. The second application refers to a realistic large rural drainage network. The results of this application show that:

- the cost of the optimal rural drainage network can be very sensitive to the choice of the value to assign to the ending node excavation depth. In particular, the optimal solution obtained fixing the ending node elevation can be much more expansive than the optimal solution obtained with the ending node excavation left free to vary in a given interval. For this reason, fixing *a priori* the network outlet elevation should be avoided, when possible, technically valid solutions could be obtained by exploiting the possibility that the network outlet channel leaps into the receiving water body;
- in many cases, the optimization procedure tries to find the optimal solution by increasing the channels slope and reducing the channel width; consequently, the channels' width may decrease in the downstream direction, despite the fact that the design discharges increase downstream. Of course, the solutions with decreasing channels' cross section in the downstream direction are not desirable, because they are inefficient when backwater effects are present during on-stationary conditions. For this reason, the constraint c_6 should be always enforced in practical cases;
- the optimal values of the mutation probability *mp* fall in the range (0.0075, 0.0225) for the cases examined. This result is in good agreement with the values of *mp* often suggested in the GA literature, with reference to hydraulic engineering applications.

The approach proposed in this work is based on the preventive knowledge of the discharges flowing through each channel of the drainage network, and on the hypotheses of steady and uniform flow conditions. These limitations, though unable to help in establishing very different minimum cost solutions (Cimorelli *et al.* [43]), can be removed considering a hydrologic model for the evaluation of the discharges, and using a hydraulic model (De Saint Venant Equations or their parabolic approximation) in order to evaluate the hydraulic performance of the channels.

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Author Contributions

All authors contributed equally to this work. In particular:
Luca Cozzolino contributed to the article writing.

Luigi Cimorelli wrote the computer programs used for the computations.

Carmela Mucherino performed the computations and the analysis of results.

Carmine Covelli contributed to the article organization and with the elaboration of figures and tables.

Domenico Pianese had the basic idea of the present work and coordinated the research group.

Conflicts of Interest

The authors declare no conflict of interest.

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Hydrodynamic Performances of Air-Water Flows in Gullies with and without Swirl Generation Vanes for Drainage Systems of Buildings

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Abstract: As an attempt to improve the performances of multi-entry gullies with applications to drainage system of a building, the hydrodynamic characteristics of air-water flows through the gullies with and without swirl generation vanes (SGV) are experimentally and numerically examined. With the aid of present Charge Coupled Device (CCD) image and optical systems for experimental study, the mechanism of air entrainment by vortex, the temporal variations of airflow pressure, the trajectories of drifting air bubbles and the self-depuration process for the gullies with and without SGV are disclosed. The numerical simulations adopt Flow-3D commercial code to attack the unsteady two-phase bubbly flows for resolving the transient fields of fluid velocity, vorticity and pressure in the gullies with and without SGV. In the twin-entry gully without SGV, air bubbles entrained by the entry vortex interact chaotically in the agitating bubbly flow region. With SGV to trip near-wall flows that stratify the drifting trajectories of the air bubbles, the air-bubble interactions are stabilized with the discharge rate increasing more than 7%. The reduction of the self-depuration period by increasing discharge rate is observed for the test gullies without and with SGV. Based on the experimental and numerical results, the characteristic hydrodynamic properties of the air-water flows through the test gullies with and without SGV are disclosed to assist the design applications of a modern drainage system in a building.

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1. Introduction

To facilitate the efficient water supply and discharge for a building remains as a difficult task due to the complex flow bifurcations in water supply networks as well as the dynamic and unsteady interfacial air-water flow mechanisms developed in a drainage system. For preventing odor transmissions into habitat spaces through a drainage network, the gullies that reserve a water seal for many discharge branches have demonstrated their convenience for installation and maintenance, with opportunities to simplify the drainage system. A recent growing rate for the usage of gullies in Taiwan has proven their potential benefits for building industries. For each device installed in a drainage system, its impacts on the system stabilities, in particular on the variations of airflow pressures responsive to the intermittent discharge(s) through a drainage piping system, have to be identified prior to its widespread applications.

Unlike a siphonic roof drainage system, the random and intermittent falling water into the vertical stack via the various discharge branches in a drainage system is not generally at the full water condition but entrains airflow to formulate a variety of complex air-water flows with various two-phase flow patterns. The interfacial air-water flow structures are affected by the geometries of the pipe-line and appliances, the flow rate and the location in a drainage system. In a branch and the vertical stack of a building drainage network, the interfacial flow structures are typical of intermittent stratified, wavy and annular flows [1]. The momentum changes of air-water flows caused by varying flow direction, expansions and contractions, bifurcations and/or chocking the airways incur the locally positive or negative transient airflow pressures that propagate throughout the entire drainage system at the sonic speed [2]. The impacts of such transient propagation—including the effects on acoustic resonances, discharging capacities and local negative or positive pressures—depend on the air-water interfacial structures and on the reflection and transmission of pressure waves on the interfacial and solid boundaries. Following a transient water discharge from the branch into the vertical stack of a drainage system, the considerable pressure oscillations at the elbow bend of the vertical stack were demonstrated to affect the entire drainage network [2]. At locations where the water curtain or excursion develops to intermittently block a high momentum air stream, the trap seal is often diminished by the raised positive airflow pressure due to the water hammer effect [3]. As the water seal prevents the transmission of foul odors ingress into the habitable spaces through the interconnected drainage network in a building, the survival of each water seal during random discharges is of primary importance. The various design codes for architectures normally request a trap seal with about 50 mm water height corresponding to the permissible pressure excursion of $\pm 375\ \mathrm{N \cdot m^{-2}}$ [1,2]. To achieve this design goal, the relevant experimental and numerical works have being carried out. As an attempt to suppress the positive pressure surges in a drainage system, the propagations of air pressure transient in a simulated drainage system by solving the St. Venant equations using the finite difference scheme was numerically performed [4]. With the complex two-phase air-water flow structures in a drainage system, the suppression of undesirable pressure transients still remains as a formidable task. In particular, the air-water flow phenomena in the various types of components and appliances of a drainage system are interdependent, leading to complicated interactive hydrodynamic responses [1–4]. As an attempt to moderate the positive airflow pressure surges initiated from the bottom elbow bend of a vertical stack [3,4], the pressure accumulator was installed to provide additional expansion space for alleviating the positive airflow transients [4]. The streamlined vortex fin(s) with sidewall grooves [3] was installed at the elbow-bend of a vertical stack to induce longitudinal swirls for penetrating the downstream water curtain developed in the elbow-bend. With the numerical schemes for attacking the two-phase flow problems in a drainage system [5–8], the entrainment model was developed for solving the hydrodynamic characteristics of multi-phase flows involving hydraulic jumps with air entrainments [7,8]. With the presence of entrained air to add the damping effect on the collapsing bubbles, the damages caused by cavitation were alleviated, thus recommending the installation of aeration devices to entrain air for alleviating the cavitation effect.

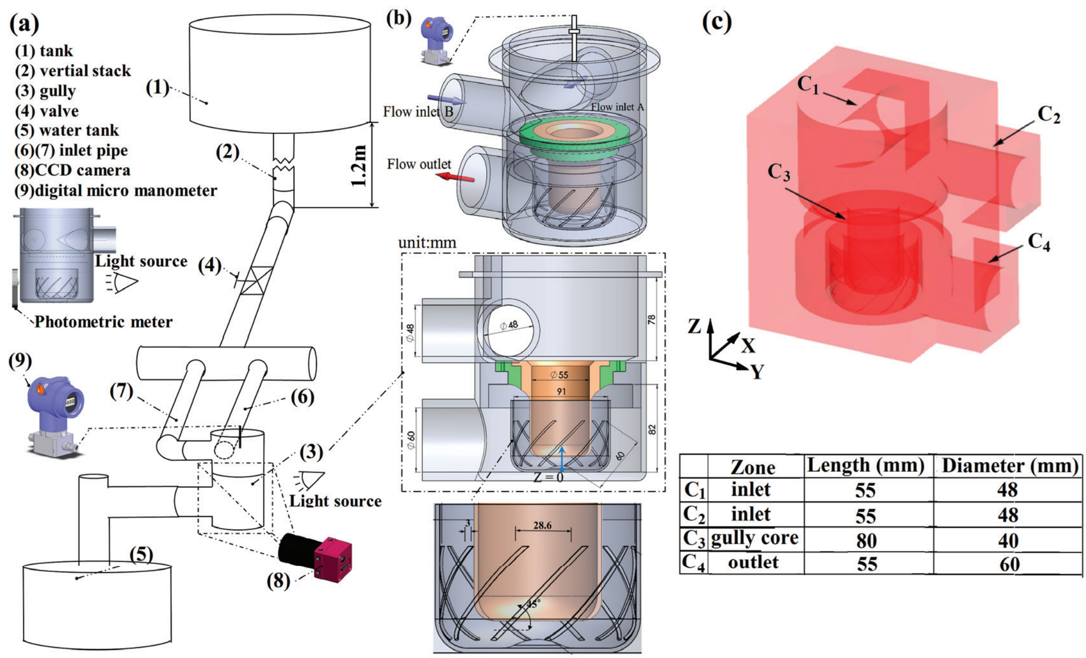
In view of a gully within which the common water seal for many discharge branches is trapped, the hydrodynamic characteristics for the through air-water flow are further complicated and dependent on the geometries of the flow pathways. In [9,10], the experimental measurements for the flow dynamics and the numerical simulations for the dynamic responses in the multi-outlet siphonic roof drainage systems were respectively reported. The fundamental air-water flow phenomena in the multi-entry gully were illustrated using a set of numerical results simulated by Flow 3-D code [11]. Based on the assumption of lumped bubbly flow for the multi-entry gully, the geometries of entry and discharge ports as well as the plenum chamber were shown as the predominant factors to affect the hydrodynamic performances for this type of multi-entry gullies [11]. Driven by the need to miniaturize the multi-entry gully for building applications, a streamlined bump [12] was fitted at the location downstream the discharge port. With the locally siphonic effects at the throat of the partitioned discharge port, the upstream air-water flows were substantially stabilized; while the maximum flow rates were limited by the choking nozzle effect at the discharge port. In order to raise the maximum discharge capacity for the shallow type multi-entry gully, a ring of SGV (swirl generation vanes) is fitted in the annular flow pathway for stabilizing the air-water flows by stratifying the air-bubble drifting trajectories along the swirl induced by the SGV. This study adopts experimental and numerical methods to probe into the air-water flow phenomena taking place in the shallow-type twin-entry gullies without and with SGV. The flow phenomena, in particular for the dynamic air-water interfacial flow structures, disclosed by this work are beneficial for gully design practice with the follow-on researches directing toward the acoustic aspect of flow induced vibrations and the miniaturization of gully with optimized discharge rate. In what follows, the experimental and numerical methods are briefly illustrated and followed by a set of selective results to comparatively examine the SGV effects on the hydrodynamic performances for this type of gully.

2. Research Methods

2.1. Experimental Apparatus and Test Details

Figure 1 depicts (a) test facilities with the optical device measuring the self-depuration performance (b) a twin-entry test gully with SGV. As depicted by Figure 1a, the supplied water from tank (1) is located at second floor of the in-house fifth-floor height drainage test facility, giving rise to the pressure potential of 1.2 m of water height to facilitate the required flow rates for experimental tests. As indicated in Figure 1a, the fresh water fed from tank (1) flows through a vertical stack (2) to the twin-entry test gully (3) via two horizontal entry pipes tangent to the gully drum. The present drainage system is complied with the new construction method using the single-pipe vertical stack with the Air Admitting Valve (AAV) (4) installed on top of the vertical stack. Airflow pressures are controlled in the typical range of ±375 Nm-2 via the auto air entrainments through the AAV (4) shown in Figure 1a. The net volume of water flow through the test gully (3) in Figure 1a is measured by the downstream water tank (5) with the time span detected by the electronic timer for accounting the averaged water flow rate through the test gully (3). A scale attached along the inner periphery of each transparent inlet pipe (6), (7), as indicated in Figure 1a, detects the water flow level for the stratified entry air-water flow in the horizontal branches (6), (7). The void fraction (α) of each entry mixed water stream can be accordingly determined. The air-water flow structures in the gully at each tested water flow rate at single- and/or twin-entry flow conditions are visualized from the snapshots imaged by the Charge Coupled Device (CCD) system. This imaging system records the flow snapshots at 300 fps with 600 pixels per gully width. The CCD camera (8) shown in Figure 1a is aimed at the angle normal to the test gully (3) with a constant focal length. The static airflow pressure is detected by a computerized digital micro manometer (9) in Figure 1a with the precision of 0.01 mm H_2O. As indicated in Figure 1b, the pressure tap measuring the airflow pressure above the entry vortex of the test gully is located on the frame attached on the top plane of the test gully with the probing depth to be precisely measured. Another port of the digital micro manometer is vented to atmosphere so that the static airflow pressures at the measuring locations above the entry vortex are detected. This type of pressure measurement device utilizes the piezoelectricity to convert pressure signal into electrical potential. The pressure measurements are synchronously recorded with the flow images taken by the CCD system, which are constantly monitored by the on-line data acquisition system. The detailed temporal variations of the airflow pressure and the corresponding flow images detected at each test condition are simultaneously recorded for post data processing. The test gully is made from a transparent arctic block. At each pre-defined flow test condition, a light sheet is emitted toward the dyed test gully behind which the photometric receiver is installed to detect and record the temporal lumen variations. By way of analyzing the temporal photometric variation, which is responsive to the temporal variation of dye concentration within the test gully, the self-depuration performance is revealed.

Figure 1b depicts the twin-entry test gully with SGV. As shown in Figure 1b, the test gully is configured by a vertical primary drum that directs the entry mixed water streams from the horizontal twin-entry ports in the downward direction toward the gully base. The radial spreading air-water stream then sharply turns and flows upward in the annular pathway between the primary and secondary drums. Over the circumferential band on the outer cylindrical wall of the secondary drum, ten SGV are in-line arranged and oriented at 45 deg. relative to the upward stream. These vanes are fitted to trip the anti-clockwise annular swirl between the primary and secondary drums. The cross-section area of discharge port is equal to the sectional annular area between the primary and secondary drums. The upward air-water stream is spilled out of the annular pathway toward the discharge port. As the overlapping height between the primary and secondary drums is 50 mm, the minimum water seal height in the test gully is ensured above than 50 mm. A replaceable filter leaf is installed above the cylindrical core on the top of the test gully, which permits the air entrainments from the surrounding atmosphere. As the two entry ports are in tangent with the outer rim of the gully casing, a central vortex is induced in the primary drum after feeding the mixed water flow into the gully. The free surface of the entry vortex formulates the airway to entrain air into the liquid pool, which will be later demonstrated. It is noticed that present orientation for the SGV is attempted to induce the co-current swirl at the same direction as the free vortex formulated in the primary drum. With the co-current swirl in the annular flow pathway in which the air-water stream flows in upward direction, the drifting air bubbles are guided by the near-wall flows over the roughened cylindrical wall on the secondary drum.



	Zone	Length (mm)	Diameter (mm)
C_1	inlet	55	48
C_2	inlet	55	48
C_3	gully core	80	40
C_4	outlet	55	60

Figure 1. (**a**) test facilities; (**b**) twin-entry test gully with swirl generation vane (SGV); (**c**) layout of numerical model.

In order to examine the self-depuration performance of a gully, a set of optical device [12] is adopted to detect the temporal variations of the lumen level shaded by the dyed test gully. In the attempt to measure the self-depuration performance for the bulk flow of a test gully, the relative self-depuration properties are comparatively evaluated by measuring the temporal L/L_1 variations. The photometric meter adopted by this work is a two-dimensional device, which is attached on the transparent cylindrical casing of the test gully as indicated in Figure 1a. As the air entrained into a test gully transforms into the air bubbles taking various shapes, the received photometric levels behind the test gully with fresh water flow are affected by the light scattering through these agitating air bubbles. Thus, the normalized lumen level through the test gully at each test condition with fresh water is initially detected by present computerized optical system. The photometric receiver transmits the received light signal to the Personal Computer (PC), giving rise the lumen reference to determine the completion of self-depuration process at each test condition. By way of feeding the mixed water at the particular test condition defined by Re_L and α, namely the interfacial Reynolds number and void fraction of entry flow, the reference lumen levels at the pure water flow conditions (L_1) are pre-determined. It is interesting to note that, as the resolving air bubbles in the test gully reflect and scatter light, the instant lumen values at each pure-water test conditions oscillate about the corresponding L_1 reference. With self-depuration tests, the water trap stored in the test gully is dyed by the black ink to give the pre-defined lumen level (L_0) for a particular set of tests. The L_0 level at each "dark" test condition is controllable by adjusting the ink concentration and appears as a stable value due to the absence of air bubble prior to feeding the mixed water into the test gully. After charging the mixed water into the test gully, the instant lumen level (L) starts rising from L_0 toward L_1. The detailed temporal lumen (L) variation from L_0 to L_1 reflects the self-depuration performance for the test gully. For the test gullies with different geometries or different entry flow conditions, the L_0 and L_1 references are accordingly varied and measured. The temporal variation of normalized lumen in terms of L/L_1 is used to quantitatively characterize the self-depuration performance for each test gully. The time lapse taken for L/L_1 approaching 0.99 is defined as the self-depuration period correspond to the particular test condition.

2.2. Numerical Method and Simulation Details

With the Flow-3D code, the continuity equation and Navier-Stokes equation which describes the momentum conservation law for incompressible viscous flow within the fluid domain Ω surrounded by a piecewise smooth boundary Γ are described by Equations (1) and (2) respectively:

$$\nabla \cdot u = 0 \tag{1}$$

$$\frac{\partial u}{\partial t} + (u \cdot \nabla)u = -\nabla p + \frac{1}{\mathrm{Re}}\nabla^2 u + f \tag{2}$$

In Equations (1) and (2), u, p, f, Re, t respectively denote the fluid velocity vector, pressure, additional force source terms, Reynolds number and time. The solution in Ω domain satisfies the initial condition of $u = u_0$ and the non-slip boundary conditions on the solid boundary Γ. The geometries for numerical simulations are identical with the experimental test models using the scaling factor of unity as shown by Figure 1c. The origin of present XYZ coordinate system locates at the center of the bottom plate. Within the calculation Ω domain, the numerical solutions are obtained using the fine grid cells of length 1.5 mm. The air pressures for the voids in the water stream are assumed as 1.013 × 105 Pa (1 atm). Flow entry conditions for both gullies with and without SGV are identical with the total discharging rate of 30 L/min. For each entry port, the water flow rates, Q_A and Q_B, are set at $Q_A = Q_B = 15$ L/min with the void fraction of unity. This numerical study simulated the temporal variations of the interfacial air-water flow structures, including the 3-D distributions of Fr, vorticity and static pressure, for disclosing the complex two-phase flow phenomena in the test gullies without and with SGV. For the present numerical model, the intensity of non-linearity and convective effects are sensitive to the magnitude of volume flow rate from each inlet. The Sommerfeld radiation boundary conditions are selected as the outlet flow boundary conditions so that the study for the effects of wave interactions with the solid surfaces is permissible. Justified by the experimental observations, the lumped bubbly flows are selected as the interfacial flow structures throughout the calculations.

3. Results and Discussion

3.1. Flow Structures

For establishing the comparative reference results, the flow structures in the test gully without SGV are detected against which the flow structures detected from the test gully with SGV are compared to disclose the SGV impacts on the hydrodynamic performances. The basic flow structures identified from the flow snapshots detected at all the tested flow rates (Q) of 10, 20, 30 and 40 L/min with single and twin entry flows remain similar for each type of test gullies. The basic flow structures in the test gullies without and with SGV are comparatively presented in Figure 2 at the maximum discharging rates. Having charged the mixed water from the twin entry ports, an entry vortex is formulated to convect the downward air-water stream into the primary drum. Justified by the convex curvature along the free surface of the entry vortex, the regional hydrodynamic performances for this type of test gullies are governed by the free vortex flow. However, near the center of the entry vortex, the contour of vortex reverts to be concave, featuring the forced vortex. The entry vortex in the primary drum is thus a mixed vortex. After the downward vortical air-water stream impinging onto the base plate of the test gully, the radially spreading air-water flow turns to be up-lifted through the 180°. sharp bend into the annular pathway between primary and secondary drums. Air bubbles entrained by the entry vortex are formed and drifting in this annular flow pathway, emerging the noticeably differential air-water interfacial activities between the tested gullies without and with SGV as compared by Figure 2. Clearly, the near-wall flows tripped by the angled SGV stratify the air bubbles to drift in the direction along the SGV orientation. In the test gully without SGV, the chaotic interactions among the up-drifting air bubbles take place in the annular passage, triggering considerable flow instabilities to amplify the air-pressure oscillations above the free surface between the primary and secondary drums. With the stabilized air-bubbles drift in the annular pathway among the upward flows for the test gully with SGV, the maximum discharging rates at present pressure potentials tested are increased more than 7% from those through the test gully without SGV.

Numerical simulations successfully capture all the dominant flow structures detected by the experimental study for the test gullies with and without SGV. The numerical test results obtained at water inflow rate for each entry port at 15 L/min show favorable agreements with the experimental measurements, thus confirming the calculated flow and pressure fields at the air-water flow conditions. The distributions of instant fluid velocity and pressure over the middle vertical planes of $Y = 0$ and $X = 0$ at $t = 10$, 20 and 30 s with $Q_A = Q_B = 15$ L/min are collected in Figure 3. In primary drum and the annular pathway between primary and secondary drums, the typical gravity-driven hydrostatic pressure variations are observed. When the upward air-water stream spills out of the annular pathway, the radial spreading water screen emitted from the top rim of the secondary drum envelops air bubbles. The free surface surrounding the outer wall of the secondary drum takes the unsteady wavy pattern for both gullies as shown by Figure 3. In the annular pathway between the primary and secondary drums and at the wavy free surface outside the secondary drum, the agitating bubbly air-water flows formulate the unstable flow region in this type of gully. Except in the agitating bubbly air-water flow region among which the air-bubble drifts are considerably affected by SGV as seen in Figure 2, the air-water flows in the gullies with and without SGV as shown by Figure 3 share the similar pattern. Many small-scale vortices with short life cycles are intermittently developed and resolved in both gullies with and without SGV.

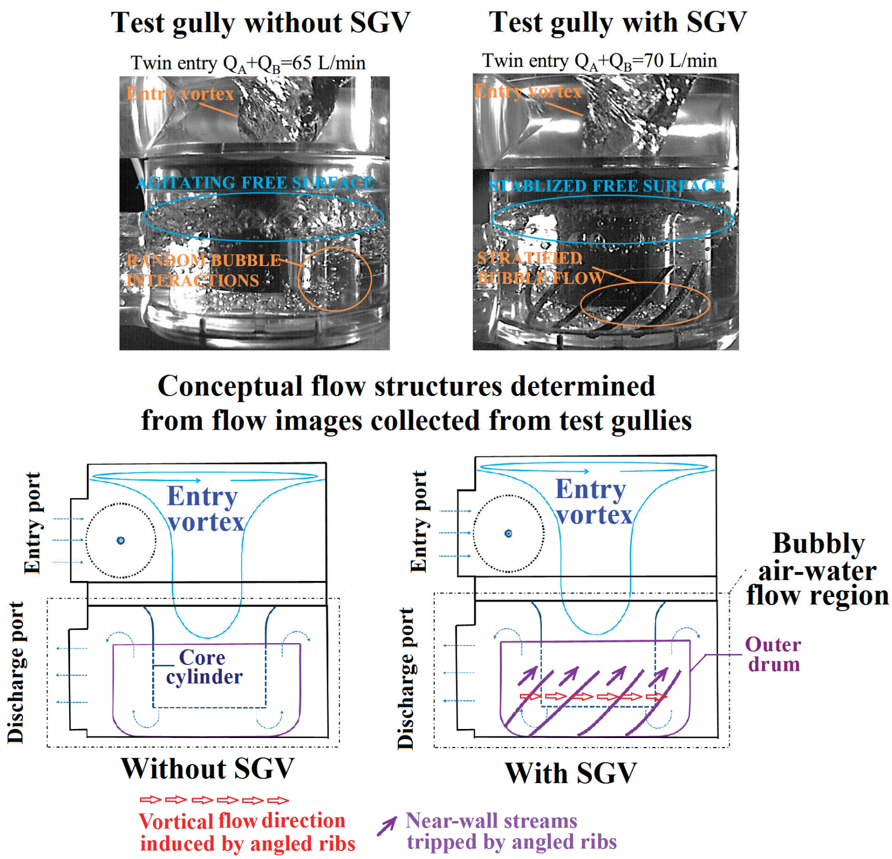


Figure 2. Air-water flow structures in test gully without SGV at $Q_A + Q_B = 65$ L/min andin test gully with SGV at $Q_A + Q_B = 70$ L/min.

To depict the complex unsteady air-water flow structures in present gullies without and with SGV, the three dimensional distributions of instant Froude number (*Fr*) at t = 5 and 30 s are calculated and collected in Figure 4. Present *Fr* is defined as the ratio of fluid velocity to the gravitational wave velocity to physically respond the ratio of inertial to gravitational forces for indicating the relative resistances of submerged air bubbles moving through the water stream. As compared with Figure 4, the *Fr* levels among the agitating bubble flow region in the gully without SGV are higher than the counterparts in the gully with SGV. Even with the protruding SGV to add the associated frictional and form drags along the flow pathway in the gully with SGV, the flow resistances attributed to the chaotic air bubble agitations in the gully without SGV still supersede the additional flow resistances added by the SGV; which leads to the increased maximum flow rates under the same pressure heads from the discharges for the gully with SGV. In Figure 4, the complete 3-D flow structures formulated by the entry vortex, agitating bubbly flow region along the serpentine flow pathway and the discharge flow with unsteady wavy free-surface are similar for both gullies without and with SGV to signify the characteristic flow pattern for this type of gully.

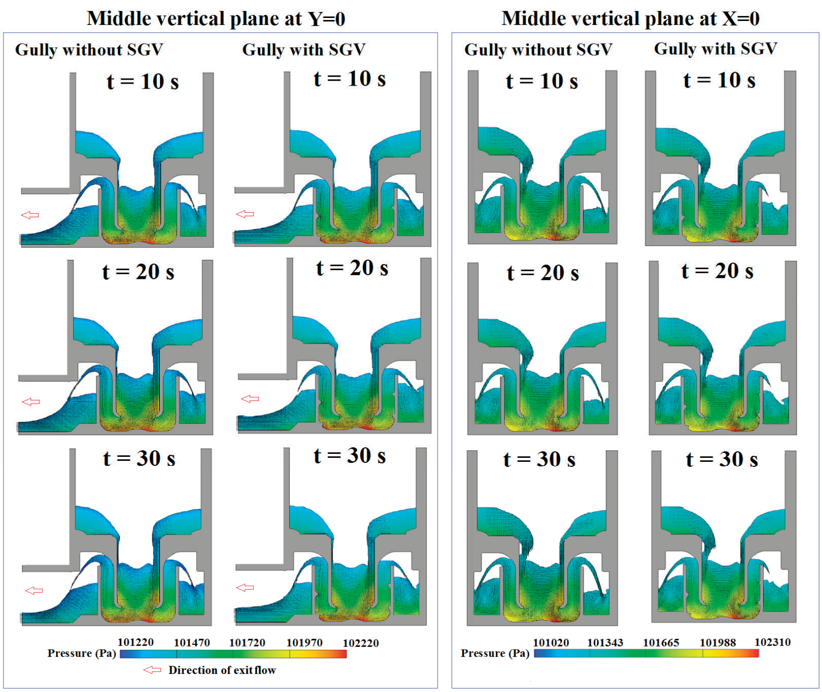


Figure 3. Distributions of instant fluid velocity and pressure over middle vertical planes of Y = 0 and X = 0 at t = 10, 20 and 30 s with $Q_1 = Q_2$ = 15 L/min.

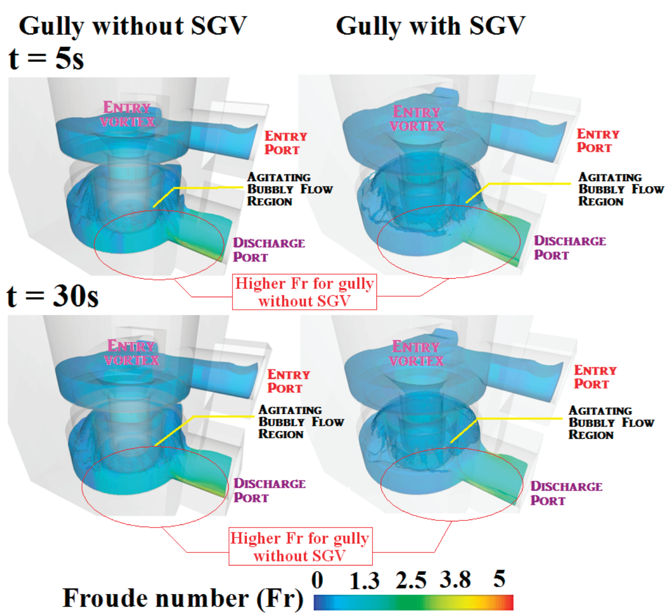


Figure 4. Three dimensional distributions of instant Froude number reflecting the overall flow structures in gullies with and without SGV.

3.2. Air Entrainments by Entry Vortex

For this type of gully, the downstream air-water flow structures are affected by the flow phenomena caused by the entry vortex, which include the considerable air entrainments. Following the conventional vortex theory, considerable radial pressure variations over the free surface and among the vortex are generated and affected by local fluid velocities. This is demonstrated by Figure 5, which compares the distributions of instant velocity and pressure contours between the gullies with and without SGV over three horizontal XY planes at $Z = 22$, 34 mm that are sectioned through the annular pathway between the primary and secondary drums and at $Z = 46$ mm under the primary drum. As Z increases, the gravitational effect increases the hydrostatic pressures in general, which is evidently shown by sequentially examining the three pressure contours obtained at $Z = 22$, 34, 46 mm at each t selected shown by Figure 5. At $Z = 22$ mm, the XY section through the exit port is fully occupied by the airflow; whereas the evident anti-clockwise vortex circulation are already emerged to fully occupy the primary drum. At $Z = 34$ mm, the pressures along the vortex outer edge are further elevated but moderated at $Z = 46$ mm. When the downward vortex stream is radially spread on the XY plane at $Z = 46$ mm, the characteristic signatures for vortex are according weakened for both gullies as demonstrated by Figure 5. With all the flow fields sectioned through the XY planes at $Z = 22$, 34 and 46 mm, the vortex core consistently show the lowest pressure levels due to the high fluid velocities. As the fluids approach the center of vortex, the increased fluid velocities are accompanied with the reduced static pressures. Once the static pressures over the free surface of the entry vortex fall less than the atmospheric level, the surrounding air above the entry vortex is entrained into the swirling liquid pool and converted to the air-bubbles by the surface tension effect. With the air entrainments by the entry vortex, a considerable amount of drifting air bubbles in the flow pathways is consistently observed even if the void fraction (α) over the flow entry ports is zero at the a full-water conditions. Although the resolving air bubbles in the present test gully are partially attributed to the local pressure reductions along the flow pathway, the air entrainment by the entry vortex is considered as the manifesting mechanism responsible for introducing air bubbles into the water stream. This is demonstrated by Figure 6 in which a series of continuous flow snapshots are selected to illustrate the process of air entrainment by the entry vortex.

To experimentally verify and visualize the mechanisms for the air entrainment by the entry vortex, the temporal variations of the airflow pressures, starting from charging the mixed water into the test gully, are individually detected at the various Z locations along the vertical central core ($X = Y = 0$) as depicted in Figure 6a. At $Z = 74$ mm, the probe of pressure sensor is about 1 mm above the liquid surface of the entry vortex-core. All the temporal variations of the airflow static pressures collected in Figure 6a from the different Z locations follow a similar varying trend. Within an initial period about 30 s after feeding mixed water into the test gully at the single entry condition of $Q = 30$ L/min, the entry vortex remains as developing; whereas the liquid level in the gully is up-rising to compress the trapped air within the gully drum, leading to the positive pressure heads along the central core as shown by Figure 6a. At the instant that the discharge of mixed water flow is partially choked, the upstream pressure waves generate an abrupt pressure increase at all the measured Z locations as shown by Figure 6a. Followed by the sudden airflow pressure rises shown by Figure 6a, the growing strength of the entry vortex keeps accelerating and dragging the airflow adjacent to the free surface of the entry vortex, leading to the subsequent reducing trend of pressure reductions at all the Z locations seen in Figure 6j. The negative airflow pressures at the locations close to the free surface of entry vortex are then emerged and stayed to trigger the process of air entrainment as demonstrated by the following Figure 6b–j. Due to the complex and interactive air-water interfacial mechanisms among the vortex core region, the static airflow pressures start oscillating about the atmospheric level to promote the unsteady air entrainments by the entry vortex as $t > 70$ s for this particular test condition.

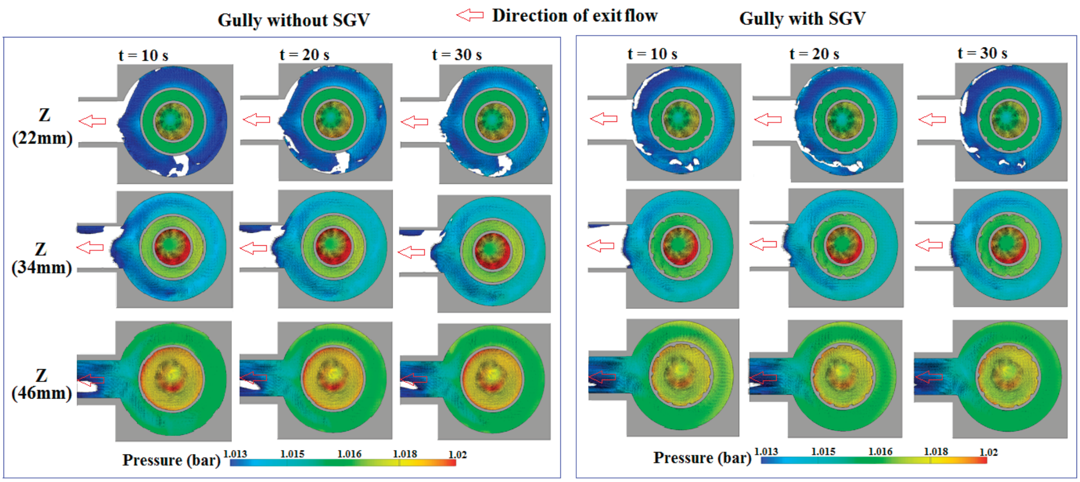


Figure 5. Distributions of instant velocity and pressure contour for gullies with/without SGV over horizontal XY planes at $Z = 22$, 34, 46 mm.

The process of vortex deformation is initially observed at instants seen in Figure 6b–c by sharpening the vortex core in downward direction seen in Figure 6c. As a result of the driven pressure gradients on the free surface of the entry vortex, a lumped air bubble is formulated at the vortex core; but still coherently attached on the free surface of the entry vortex as shown by Figure 6d. After a short time lapse, the separation of air bubble into the liquid pool is observed as seen in Figure 6e; which can be occasionally followed by another sequence of vortex-core deformation and air-bubble separation seen in Figure 6f. The large-scale separated air bubble that submerges into the swirling liquid pool is generally broken into small air bubbles which scatter underneath the vortex core as indicated by Figure 6g–h. The interfacial air-bubble evolutions disclosed by sequentially viewing the flow snapshots detected at the instants shown by Figure 6b–h are followed by the subsequent vortex-core deformation as typified in Figure 6i to complete an air-entrainment process induced by the entry vortex. The successive process for another air entrainment is initiated with the flow image shown by Figure 6j. It is noticed, with present test gullies, the entire air entrainment process by entry vortex, as typified by Figure 6b–j, is completed within 1 s.

In addition to the considerable flow resistances by the air bubbles in the flow passages formulated in the gullies without and with SGV as demonstrated by Figure 4, the entrained air into the water stream also affect the vorticity distributions in the gullies. To explore the impact of entrained air on vorticity distributions, the instant vorticity contours for the gullies with/without SGV over horizontal *XY* planes at $Z = 22, 34, 46$ mm at $t = 10$, 20 and 30 s with $Q_A = Q_B = 15$ L/min, which corresponding to the Computational Fluid Dynamics (CFD) scenarios collected in Figure 3, are compared by Figure 7.

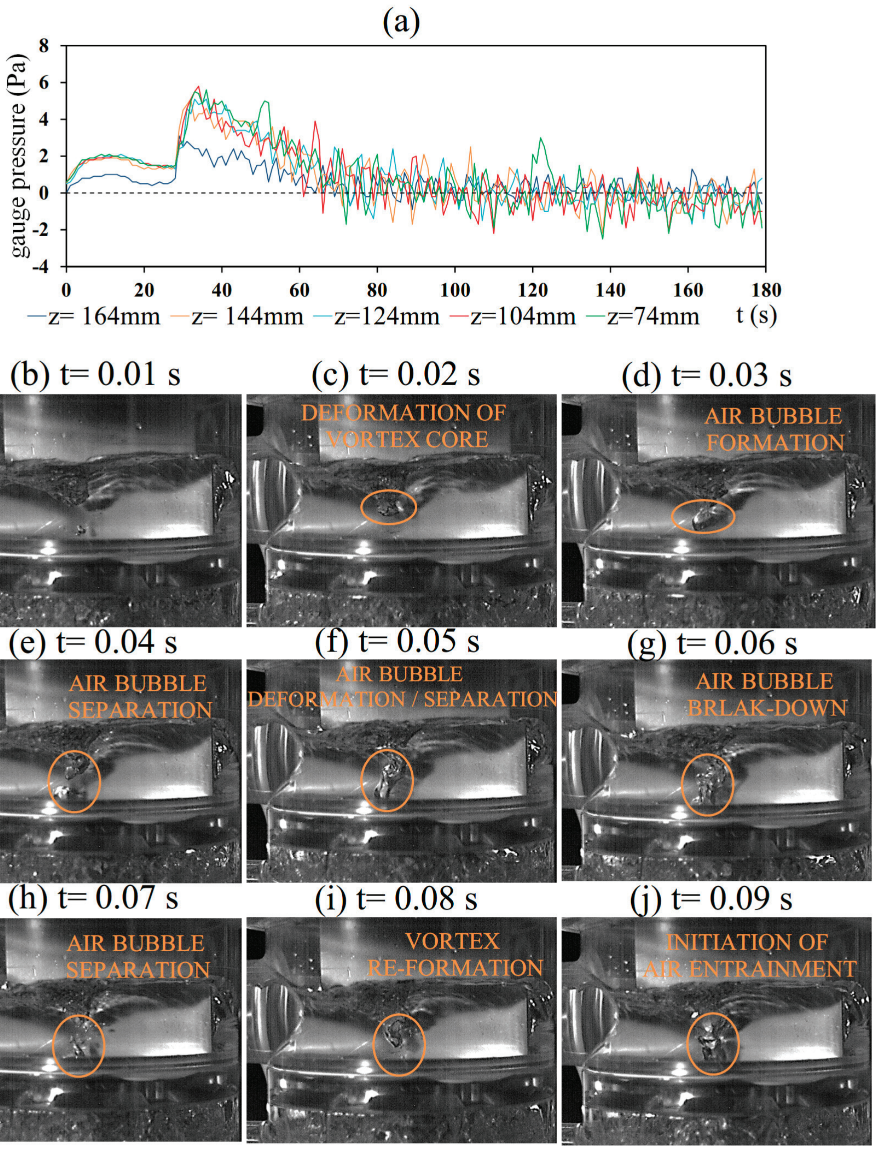


Figure 6. Temporal airflow pressure variations and corresponding flow snapshots demonstrating the process of air entrainment by entry vortex.

It is interesting to note the ring of high vorticity circling around the center of entry vortex. Due to the air-entrainment taking place at the center of the entry vortex, the development of local angular momentum by the shearing action resulting from the particular fluid velocity field is interfered. As a result, the local vorticity at the center region of the entry vortex is weakened to be less than those emerging along the surrounding rim shown by Figure 7. Over the annular sections between the gully casing and the secondary drum, several spots show the negative vorticites, in particular along the air-water interfacial boundaries marking as the black solid lines in Figure 7. The counteracting circulations for the air bubbles in the water stream are suggested by present numerical results. Above all, with applications to drainage systems, present type of gullies can be classified as the appliance capable of entraining air into the drainage system. Flow instabilities are mainly attributed to the air bubble interactions in the agitating bubbly flow region specified by Figure 4.

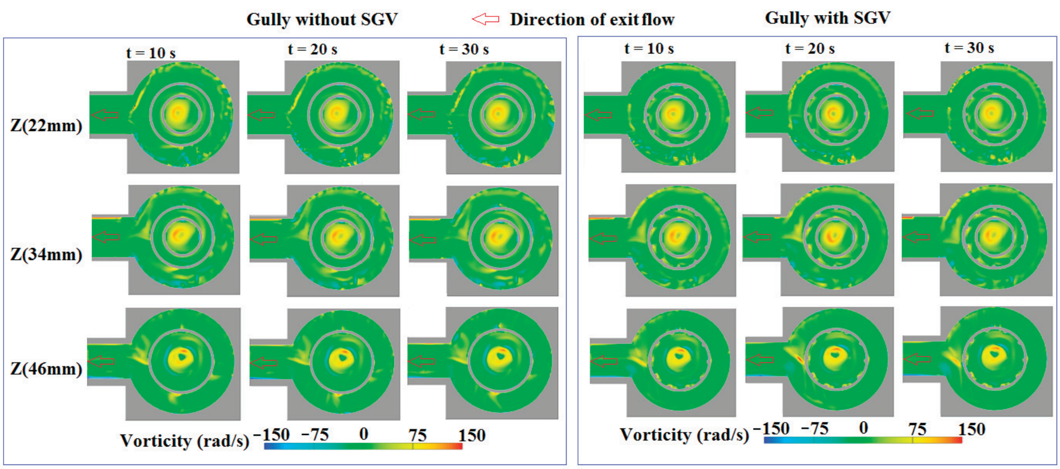


Figure 7. Distributions of instant vorticity for gullies with/without SGV over horizontal *XY* planes at $Z = 22$, 34, 46 mm.

3.3. Air Bubble Drifts in Test Gullies with/without SGV

As the primary contributions of present SGV for improving the hydrodynamic performances of this type of gullies, the near-wall water streams tripped by the angled SGV assist to guide the drifting air bubbles over the agitating bubbly flow region. This is demonstrated by Figure 8 in which the trajectories of drifting air bubbles in the test gullies without and with angled SGV are compared. The instant flow snapshots adopted to identify the drifting trajectories for the air bubbles in the agitating bubbly flow region are also shown in Figure 8. As summarized in the conceptual flow diagram for the test gully without SGV in Figure 8, the drifting trajectories of air bubbles mainly follow three routes indicated by the A, B, C traces in the flow snapshots as shown in Figure 8. Along the drifting routes A and B in the test gully without SGV, the complex bubble collisions and coalescences and oscillations are observed. Relative to the gully with SGV, the highly agitated free surface between the secondary drum and the gully outer cylindrical casing is observed for the test gully without SGV. By fitting the angled SGV along the cylindrical wall of the secondary drum, the air bubbles are drifting along with the near-wall water streams tripped by the angled SGV so that all the A, B, C trajectories for air bubble drifts in the agitating bubble flow region are guided/stratified along the angled SGV direction to moderate the flow instabilities caused by the random air bubble collisions and coalescences and oscillations.

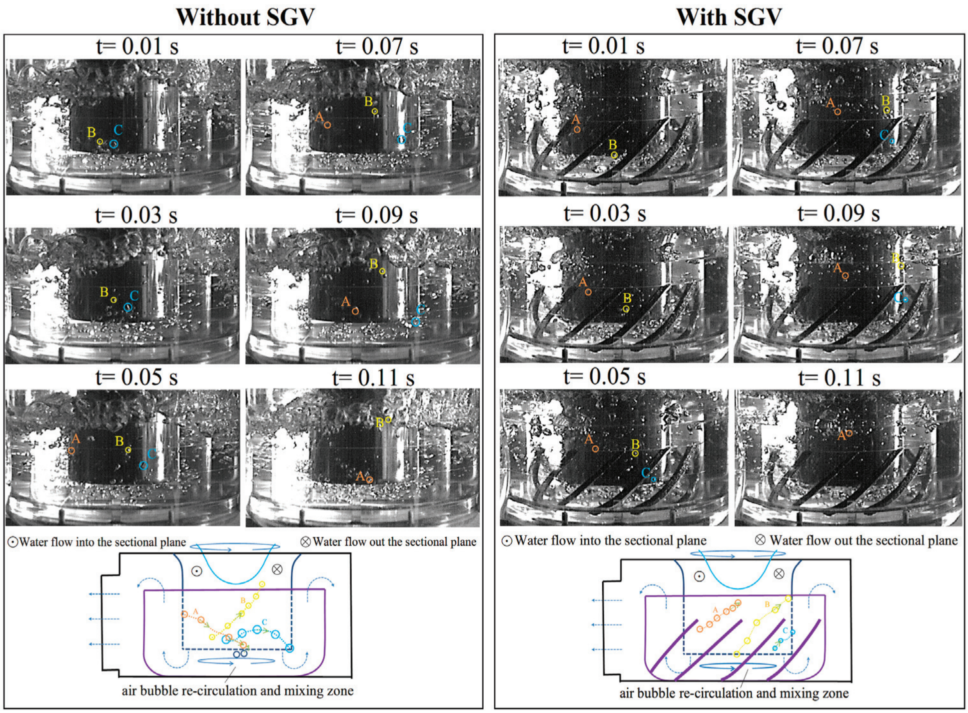


Figure 8. Drifting trajectories of air bubbles in test gullies without/with SGV.

3.4. Self-Depuration Performances

While the agitations of air-bubbles and the motion of free-surface in present test gullies without and with SGV are considerably different, the performances of self-depuration are similar. Figure 9 compares the temporal variations of L/L_1 ratios at all the tested flow rates with single and twin flow entry conditions for the test gullies without and with SGV. As compared by Figure 9, the temporal variations of L/L_1 ratios at all the tested flow conditions with single and twin flow entries follow the similar pattern. Prior to charging the mixed water into each test gully, the dye concentration is controlled to provide the referenced L/L_1 ratios at about 0.4. After feeding the mixed water into each test gully, an initial start-up period with stable L/L_1 levels proceeds about 10 s. Following the stable period with L/L_1 ratios at about the reference "dark" condition, the L/L_1 ratios increase sharply within a short period about 5 s. The physical implication for such rapid L/L_1 increase is the significant improvement for the self-depuration performance attributed to the development of entry vortex which effectively discharges the dyed water and replenishes with the supplied fresh mixed water. While the air entrainment is mainly caused by the entry vortex, the self-depuration performance is considerably improved by the entry vortex which rapidly replaces the dyed water by the mixed water. After the period of rapid L/L_1 increase, an exponential-like period of moderate L/L_1 increase over the period about 3–10 s is followed. As Q increases, the initiation of the rapid L/L_1 increase is advanced as shown by Figure 9. Thus, the consistent reduction of self-depuration period by increasing the discharge capacity is observed in Figure 9. The variations of self-depuration time for each test gully against total entry water flow rate at single and twin entry conditions are summarized in Figure 10. As compared with the three data trends obtained at single and twin flow entry conditions, the self-depuration time for the test gullies without and with SGV at the two single flow entry conditions labeled as QA and QB in Figure 10 are similar. By feeding the air-water mixed flows from present two perpendicular flow entry pipes in tangent with the gully drum, the flow momentums required to formulate the entry vortex are likely to be raised from the conditions with single entry flow. With the enhanced vortical strength for the entry vortex at the twin-entry flow conditions, the self-depuration time is consistently less than the single-entry counterpart for the test gullies without/with SGV as shown by Figure 10. Justified by the data trends revealed in Figure 10, the empirical correlations for the self-depuration time are devised as Equations (3)–(5) and (6)–(8) for present test gullies without and with gullies:

$$T = -14.38 \ln(Q_A) + 73.77 \quad \text{(single flow entry A for gully without SGV)} \tag{3}$$

$$T = -13.83 \ln(Q_B) + 73.33 \quad \text{(single flow entry B for gully without SGV)} \tag{4}$$

$$T = -15.01 \ln(Q_A + Q_B) + 72.75 \quad \text{(twin flow entry A+B for gully without SGV)} \tag{5}$$

$$T = -13.19 \ln(Q_A) + 70.35 \quad \text{(single flow entry A for gully with SGV)} \tag{6}$$

$$T = -13.1 \ln(Q_B) + 68.81 \quad \text{(single flow entry B for gully with SGV)} \tag{7}$$

$$T = -12.55 \ln(Q_A + Q_B) + 63.37 \quad \text{(twin flow entry A+B for gully with SGV)} \tag{8}$$

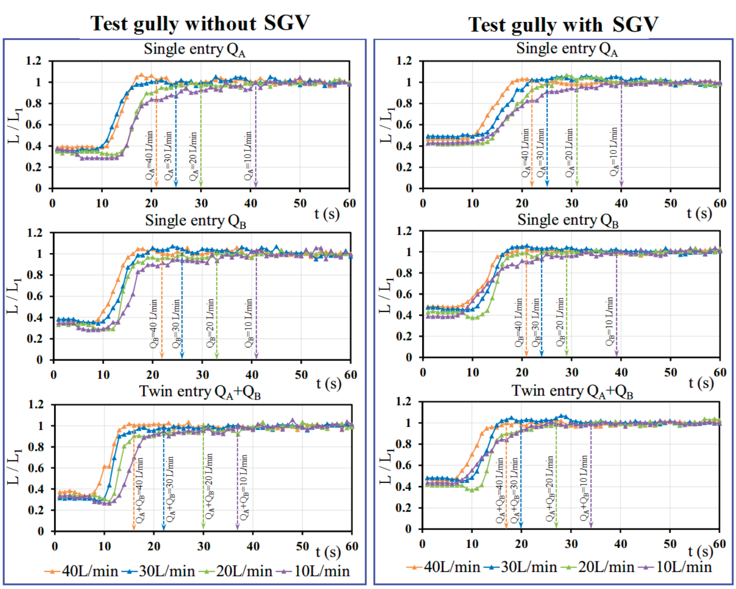


Figure 9. Temporal *L/L1* variations for test gullies without/with SGV at single/twin entry conditions.

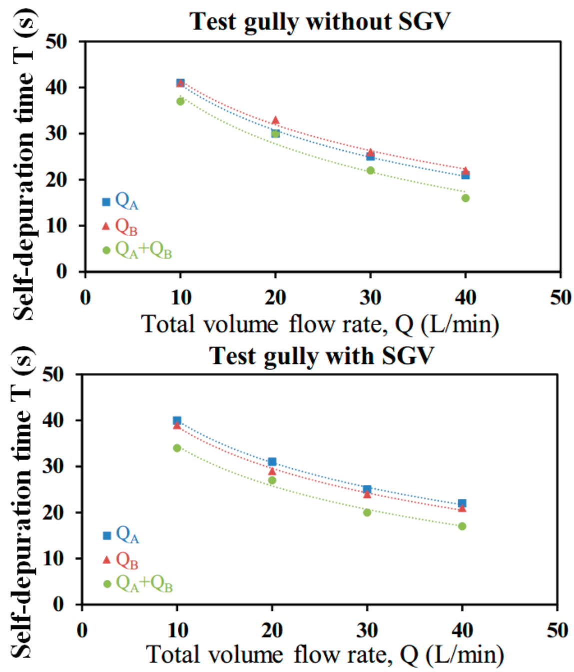


Figure 10. Variations of self-depuration time for test gullies without/with SGV at single/twin entry conditions.

4. Conclusions

This experimental and numerical work comparatively examined the hydrodynamic performances of two test gullies without and with SGV to enlighten the air-water flow structures, air entrainment mechanisms, air-bubble drifts and self-depuration properties. The conclusions emerge from this study are served as the design considerations with the applications to drainage systems in buildings. With the entry vortex formulated in the primary drum; this type of gullies is classified as the appliance that entrains air into the drainage system.

Air bubbles entrained by the entry vortex in present test gully without SGV interact chaotically in the agitating bubbly flow region. With SGV on the cylindrical wall of test gully, the near-wall flows tripped by the angled SGV stratify the drifting trajectories of the air bubbles, leading to the stabilized air-bubble interactions. Justified by the 3-D *Fr* distributions, the flow resistances attributed to the chaotic air bubble agitations in the gully without SGV supersede the flow resistances caused by the SGV. The maximum discharging rates for the test gully with SGV at present pressure head of 1.2 m water-height are increased more than 7% from the discharges by the gully without SGV.

After an initial short period of stable low L/L_1 levels during which the entry vortex is under development, the rapid L/L_1 increase followed by the exponential-like moderate L/L_1 increase reflects the characteristic self-depuration property for this type of gullies with entry vortex. The consistent reduction of self-depuration period by increasing the discharge capacity is consistently observed for present test gullies without and with SGV. Two sets of empirical correlations that permit the estimation for self-depuration periods at single and twin entry flow conditions for present test gullies without and with SGV are devised to assist the relevant industrial applications.

Author Contributions

Der-Chang Lo performed the Computational Fluid Dynamics (CFD) simulation to reveal the fundamental air-water flow phenomena in the multi-entry gully. Jin-Shuen Liou carried out the experimental tests and data analysis. Shyy Woei Chang wrote the paper, proposed the SGV and formulated the experimental method to investigate the effects of SGV on the hydrodynamic performances for the multi-entry gully.

Nomenclature Nomenclature

English Symbols

c	water wave propagation velocity (ms^{-1})
d	entry tube diameter (m)
Fr	Froude number = u/c
L	lumen
Q	volume flow rate (L min^{-1})
Re_L	Reynolds number of liquid flow for mixed entry water = $\rho_L V_L d/\mu_L$
T	self depuration time for test gully (s)
u	fluid velocity (ms^{-1})
X,Y,Z	coordinate (m)

Greek Symbols

α	void fraction of entry flow
ρ_L	density of liquid flow for mixed entry water ($kg \cdot m^{-3}$)
μ_L	dynamic viscosity of liquid flow for mixed entry water ($kg \cdot s^{-1} \cdot m^{-1}$)

Subscripts

A	flow entry A
B	flow entry B

Conflicts of Interest

The author declares no conflict of interest.

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Quantifying the Fecal Coliform Loads in Urban Watersheds by Hydrologic/Hydraulic Modeling: Case Study of the Beauport River Watershed in Quebec

Amélie Thériault and Sophie Duchesne

Abstract: A three-step method for the identification of the main sources of fecal coliforms (FC) in urban waters and for the analysis of remedial actions is proposed. The method is based on (1) The statistical analysis of the relationship between rainfall and FC concentrations in urban rivers; (2) The simulation of hydrology and hydraulics; and (3) Scenario analysis. The proposed method was applied to the Beauport River watershed, in Canada, covering an area of 28.7 km^2. FC loads and concentrations in the river, during and following rainfall events, were computed using the Storm Water Management Model (SWMM) hydrological/hydraulic simulation model combined with event mean concentrations. It was found that combined sewer overflows (CSOs) are the main FC sources, and that FC from stormwater runoff could still impair recreational activities in the Beauport River even if retention tanks were built to contain CSOs. Thus, intervention measures should be applied in order to reduce the concentration of FC in stormwater outfalls. The proposed method could be applied to water quality components other than FC, provided that they are present in stormwater runoff and/or CSOs, and that the time of concentration of the watershed is significantly lower than their persistence in urban waters.

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1. Introduction

Fecal coliforms (FC) in urban waters are indicators of recent fecal contamination, and thus of a potential pathogen contamination [1]. This is why FC concentrations are often used in water quality standards for recreational activities, such as bathing, canoeing and fishing, especially since they are relatively easy to monitor. Sources of FC in urban areas are numerous and often difficult to track [1]. For example, point sources include wastewater treatment plant effluents and combined sewer overflows (CSOs), while nonpoint sources include stormwater runoff. Nonpoint sources have been demonstrated to be more important sources of contamination than point sources in many studies conducted in urban areas (e.g., [1]). Indeed, high concentrations of FC can be found in stormwater runoff [2–5].

Due to the numerous and varied potential FC sources in urban areas, modeling is useful to identify the main origins of FC contamination in urban watercourses before the proposal of remedial actions. Many different mathematical models exist to simulate water quality in urban areas. Some are based on linear regressions and correlations with explanatory variables [6–8], while others are less difficult to apply, like the Schueler's simple method [9] or the annual load method proposed by Shaver *et al*. [10]. Other models are based on the simulation of hydrology and hydraulics, such as DR3M—QUAL

(Multi-Event Urban Runoff Quality Model) [11], HSPF (Hydrological Simulation Program-Fortran) [12], MIKE [13], HEC-RAS (Hydrologic Engineering Centers River Analysis System) [14] and SWMM (Storm Water Management Model) [15]. With these models, water quality can be estimated by specific build-up/wash-off models, or by event mean concentrations (EMC).

Modeling studies focusing on the estimation of FC are less common than for other pollutants. Studies on the estimation of FC include Servais *et al*. [16], Bougeard *et al*. [17], Manache and Melching [18], Smith [19] and ADEC [20]. Recently, McCarthy *et al*. [21] developed a model designed specifically for the simulation of microorganisms in urban stormwater (Micro-Organism Prediction in Urban Stormwater (MOPUS)).

In this paper, we propose a three-step method for the identification and quantification of the main FC sources in urban areas and for the analysis of remedial actions, based on the simulation of hydrology and hydraulics. The three steps include preliminary statistical analysis, computation of FC loads from various potential sources and analysis of remedial scenarios. The methodology is applied, as an example, to the Beauport River watershed (Canada), an urban watershed where high FC concentrations often impair aquatic recreational activities.

2. Materials and Methods

2.1. Study Area

The Beauport River watershed is located in the Quebec City region (Canada) and covers an area of 28.7 km². The Beauport River flows through the watershed over a length of 22 km. The outlet of the river is situated in the Beauport Bay, a favored location for swimming and other secondary contact activities, such as fishing, kite surfing and kayaking. The area is divided into five large occupational classes: Residential, commercial, industrial, agricultural and undeveloped, which represent, respectively, 51%, 2%, 6%, 4% and 36% of the total area, as shown in Figure 1. The different drainage systems and facilities are shown in Figure 2. Precipitations were recorded every 5 min at the location shown in Figure 2. Data concerning flow rates were available in the form of daily averages. The daily average flow rate from 2006 to 2011 was 0.74 m^3/s and the minimum recorded for those years was 0.18 m^3/s. Two types of drainage networks exist in the watershed. First, from the upstream to the center of the watershed, runoff is drained trough ditches and stormwater pipes that conduct flow to various watercourses, among which the Beauport River is the principal. Fifteen retention basins are located in this area of the watershed. Second, in the downstream part of the watershed (*i.e.*, in the subwatersheds illustrated in blue and green in Figure 2), runoff is drained through combined sewer pipes. Combined sewer overflows (CSOs) can occur in this area during rainfall, as detailed in Section 2.2.2.

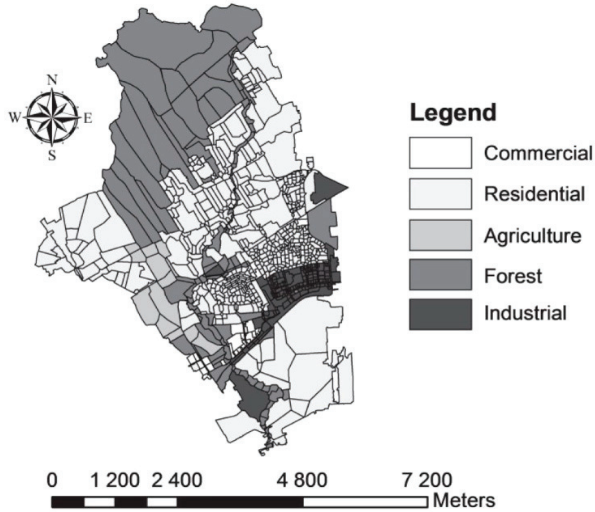

Figure 1. Land use in the Beauport River watershed.

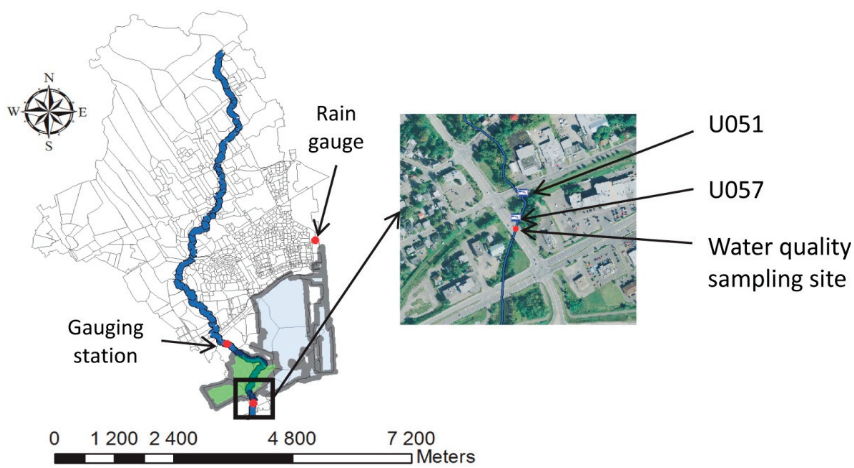

Figure 2. Separate (hollow) and combined (colored) subcatchments superposed with the location of the rain gauge, the river gauging station, the water quality sampling site and the combined sewer overflows (U051 and U057).

2.2. Available Data

2.2.1. Fecal Coliform Concentrations

Since water-related activities in the Quebec region occur mainly during the summer period, FC concentrations are tracked in the Beauport River from May to August. Data from 2008 to 2011 were analyzed. In Quebec, quality standards for FC are 200 CFU/100 mL for bathing and 1000 CFU/100 mL for secondary contact activities [22]. Measurements of FC concentrations were provided by the Quebec

City's Environmental Services department. A total of 148 daily measurements were available for the four years analyzed. The dispersion of measurements is represented in Figure 3 in the form of boxplots. All of the concentration medians were below the 1000 CFU/100 mL standard, for secondary contact activities. However, we observed a high variability in concentrations for a given year.

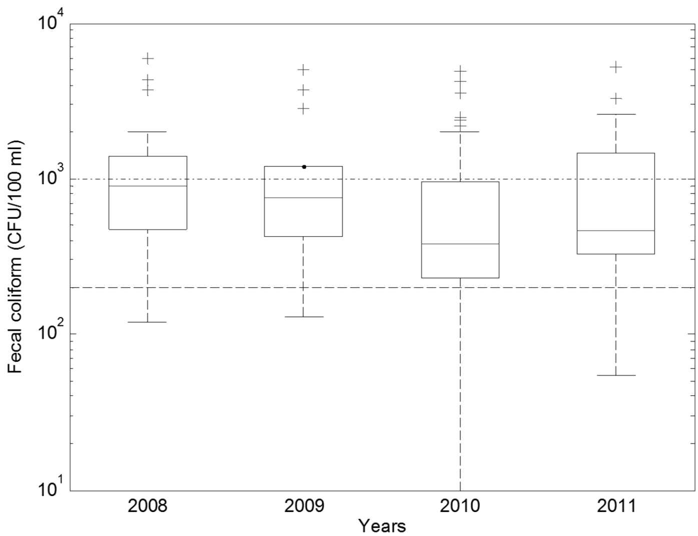

Figure 3. Boxplots of fecal coliforms (FC) concentrations for summer 2008 to 2011. The dashed line represents the 200 CFU/100 mL water quality standard and the line composed of mixed dashes and dots represents the 1000 CFU/100 mL standard.

2.2.2. Rainfall and Combined Sewer Overflows Observations

Table 1 presents the total rainfall from May to August for the four years analyzed, as measured by the rain gauge illustrated in Figure 2. Precipitations were recorded every 5 min. The 2009 measurements were the closest to the 1971 to 2000 precipitation average for the same months, which corresponds to 465 mm according to Environment Canada [23].

Data related to the CSOs were taken from the SOMAE database (*Suivi des Ouvrages Municipaux d'Assainissement des Eaux*, Monitoring of Municipal Water Drainage Structures). This program was started by the *Ministère des Affaires municipales et de l'occupation du territoire* (Quebec Ministry of Municipal Affairs and Land Use), with a main objective to conduct follow-ups of all CSO facilities in the province of Quebec. Four of the overflow facilities from the studied watershed are listed in SOMAE. From these four, only two overflowed during rainy periods in the monitored period, namely unit U051 and unit U057. The structure U051 tends to overflow less often than the structure U057. In fact, by applying the Schroeder's method [24], the critical daily rainfall height causing overflow is 1.4 mm for U057 and 4.4 mm for U051. The SOMAE database lists the date and duration of each CSO. No information on CSO volume or discharge is recorded in the database. Consequently, as specified in the next section, it was necessary to estimate the overflow volumes by simulation. The number of CSOs recorded at each facility is presented in Table 1.

Table 1. Number of combined sewer overflows (CSOs) for the two combined overflow units and rainfall data for each season (from 1 May to 31 August).

Year	Number of CSOs Caused by Rainfall		Rainfall (mm) May to August	Number of Rainfall Events May to August	
	U051	U057		>0.1 mm	>5 mm
2008	25	55	560.0	64	31
2009	34	41	507.8	61	26
2010	13	30	243.2	54	16
2011	15	50	627.4	61	25

2.2.3. River Flow

The hydrometric gauging station on the Beauport River is located more than one kilometer upstream of the river outfall, where it is not affected by tides. Flows at this location are recorded by the *Centre d'expertise hydrique du Québec* (CEHQ, Quebec Water Expertise Center) every fifteen minutes and the data are made available as mean daily values. CSOs do not affect the recorded flows since the overflow structures are located downstream from the hydrometric station. Table 2 presents the maximal, minimal, median and mean monthly flow rates for years 2006 to 2010, for the May to August period.

Table 2. Historical flow rates on Beauport River (from 2006 to 2010).

Flow Rate (m^3/s)	May	June	July	August
Maximal	2.950	3.225	4.578	6.708
Minimal	0.217	0.207	0.162	0.119
Median	0.628	0.315	0.339	0.270
Mean	0.741	0.636	0.618	0.543

2.3. Preliminary Statistical Analysis

To verify if a relationship existed between rainfall and FC concentrations in the Beauport River watershed, concentration data were divided into groups according to the total rainfall observed on the same day (day_0) as the FC measurement, the day before (day_{-1}) and two days before (day_{-2}). An ANOVA test was performed to compare the geometric mean (GM) of FC concentrations observed on days with rainfall and without rainfall, at day_0, day_{-1} and day_{-2}. Days with and without rainfall were defined using two different thresholds, which are 0.1 and 5 mm. This means that, in a first analysis, days during which less than 0.1 mm of rainfall was recorded were considered without rainfall and, in second analysis, days were considered without rainfall if less than 5 mm of rainfall was recorded.

2.4. Comparison of Load Estimation Methods

FC loads coming from the Beauport River subwatersheds were computed using two different methods, namely the simple method and a method based on the simulation of hydrology and hydraulics. The first method, as stated by its name, has the advantage of being very simple to apply, but cannot be used in the area drained by a combined sewer network. Indeed, in this kind of network, a part of runoff is drained to the wastewater treatment plant, and this cannot be taken into account by the simple method. Also, as opposed to the second method, the simple method cannot be used to assess the impact of various intervention scenarios on the FC loads discharged to the Beauport River. For both methods, the fecal coliform loads were computed for the summer period, from 1 May to 31 August, for the four years under study.

The simple method (developed by Schueler [9] and also used, among others, by the Center for Watershed Protection [25]) provides and estimation of the order of magnitude of the pollutant loads produced by rainfall runoff in an urban area over a year. The total load for a given pollutant is computed using:

$$L = R \times C \times A \tag{1}$$

where: L = annual load (M); R = annual total runoff (L); C = mean concentration (M/L^3); A = drained area (L^2).

In the work presented here, the annual runoff (R) was assessed with:

$$R = P \times RC \tag{2}$$

where: P = annual precipitation (L); RC = runoff coefficient.

The RC values vary according to land use. For the Beauport River watershed, the values proposed by Brière [26] were used (see Table 3).

Table 3. Runoff coefficients applied to the Beauport River watershed (from [26]).

Land Use	Runoff Coefficient (RC)
Residential	0.40
Commercial	0.70
Industrial	0.75
Undeveloped	0.10
Agriculture	0.15

As for the second method, the water volumes discharged to the river, from the separated and combined sewer networks, were computed using the USEPA SWMM model [15]. For both methods, loads were then estimated by multiplying the discharged water volumes by the event mean concentrations (EMC) presented in Table 4. For the stormwater outfalls, the selected EMCs are the median values proposed in [27], except for the agricultural land use. For this land use as well as for the CSOs, the EMCs are the mean order of magnitudes issued from a broad literature review, including [27–33].

Table 4. Event Mean Concentrations (EMC) values for the different land uses.

Source		EMC (CFU/100 mL)
Stormwater	Residential	7,750
	Commercial	4,500
	Industrial	2,500
	Undeveloped	3,100
	Agriculture	10,000
CSOs		1,000,000

SWMM is a dynamic rainfall-runoff simulation model used for single events or long-term continuous simulation of runoff quantity and quality, primarily from urban areas. For the purpose of this study, the separate stormwater and combined sewer systems were modeled distinctly. Both of these SWMM models were previously calibrated and validated by the Quebec City's Engineering Services department [34,35]. Some minor adjustments have also been brought to the models by the authors. More details are given in Section 3.2.

Both SWMM models solve the St-Venant's equations by dynamic wave routing and use Horton's formula for infiltration. The different parameters of the models, established by the Quebec City's Engineering Services department [34,35], are listed in Tables 5 and 6.

Table 5. Characteristics of the subcatchments in the SWMM models.

Physical Characteristics	Stormwater Model	Combined Model	Unit
Total area	25.5	3.2	km²
Number of subcatchments	914	52	–
Average slope of subcatchments	2.0	2.0	%
Average imperviousness	31	76	%
Conduit length	91	23	km
Beauport River length	21.4	–	km

Table 6. Parameters of the SWMM models.

Infiltration Model (Horton)	
Maximal infiltration rate	75–150 mm/h
Minimal infiltration rate	2–15 mm/h
Infiltration rate decay	0.001–4 h^{-1}
Manning Roughness Coefficient	
Pervious surfaces	0.25–0.28
Impervious surfaces	0.013–0.016
Pipes	0.013–0.3

2.5. Analysis of Scenarios

The objective of this analysis was to identify more efficient intervention methods to reduce the FC loads discharged to the Beauport River during and after rainfall events. To do so, the discharged FC loads were simulated according to six different scenarios, described below, for the 26 July 2011 rainfall event (from 0:00 to 23:55). Simulation of one day instead of a whole season allowed for a more precise analysis of the impacts of each scenario on the discharged FC loads, and the FC concentrations in the Beauport River. On 26 July 2011, a total of 33.9 mm of rainfall was recorded, with a maximal 5-min intensity of 25.2 mm/h (see hyetograph in Figure 4). This event was chosen as it was the 21st in importance, in terms of total runoff as simulated with SWMM, for the 2008 to 2011 summers. This means that there were, on average, five events each summer that provided more FC loads to the Beauport River than the 26 July 2011 event.

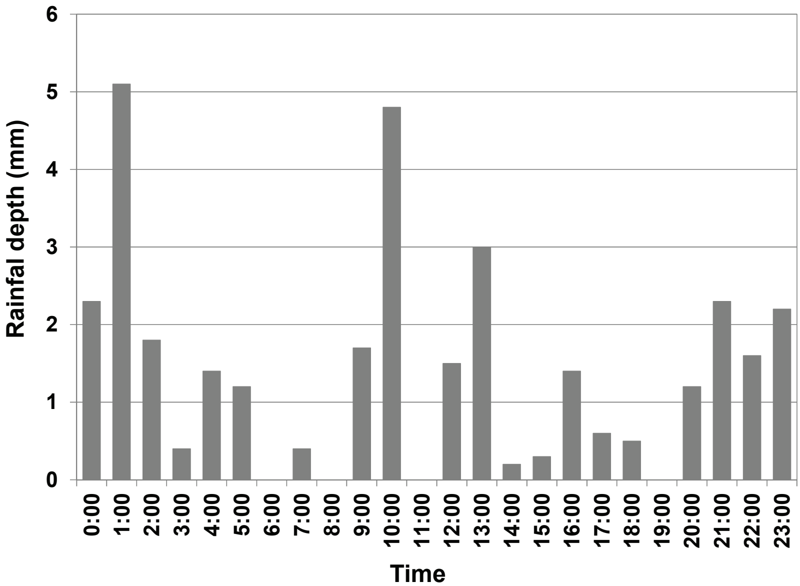

Figure 4. Recorded hyetograph on 26 July 2011.

To assess the FC concentrations in the Beauport River, a 0.36 m^3/s base flow was added to the flow simulated by the SWMM stormwater model, since this model was elaborated, calibrated and validated to properly simulate urban drainage only; consequently, it does not integrate groundwater flow nor headwater lakes, that provide water to the Beauport River during the periods without rain. The selected value of 0.36 m^3/s corresponds to the mean daily flow in the river the day before the simulated event, namely 25 July 2011, a day during which no rainfall occurred.

The six scenarios that were simulated are the following:

(1). Reference scenario (S1): Simulation of the watershed and drainage networks as they were in 2011.

(2). Retention scenario (S2): Similar to scenario 1, but with the addition of CSO retention tanks with sufficient capacities to contain all CSOs that occurred on 26 July 2011 (1935 m^3 for unit U051 and 2772 m^3 for unit U057, as simulated with SWMM).

(3). Primary treatment at some stormwater outfalls (S3): Similar to scenario 2, but with a proper retention time in the 15 stormwater retention basins already in place in the watershed, in order to achieve a 60% FC removal rate.

(4). Reduction of imperviousness (S4): Similar to scenario 2, but with a 1% decrease in the percentage of imperviousness for each subwatershed (meaning that the imperviousness of each subwatershed was multiplied by 0.99).

(5). Optimal management of stormwater (S5): Similar to scenario 2, but with a reduction in the EMC values for stormwater outfalls (respectively, 2500, 200, 500, 1.5 and 4.5 CFU/100 mL for the residential, commercial, industrial, agricultural and undeveloped land uses). These values are the minimal values observed by Wong ([36], cited in [27]). They correspond to EMCs that could be obtained with a very rigorous management of the urban surfaces and stormwater network, including correction of sewer cross connections, frequent road sweeping, regular cleaning of stormwater pipes, increase and promotion of infiltration, *etc*.

(6). Compilation (S6): Compilation of all scenarios presented above.

3. Results and Discussion

3.1. Preliminary Statistical Analysis

Results of the ANOVA tests comparing the FC concentrations in the Beauport River for days with and without rainfall (for the day of FC measurement, day_0, the day before the measurement, day_{-1}, or two days before the measurement, day_{-2}) are presented in Table 7. The ANOVA test confirmed that the geometric mean (GM) of FC concentrations observed on days with rainfall was significantly different from those observed during days without rainfall (day_0). Also, the GM of FC concentrations were different between days with and without rain the day before (day_{-1}). However, this difference was not observed for day_{-2}.

Table 7. Geometric mean of FC concentrations as of function of rainfall height for day_0, day_{-1} and day_{-2} and results of the ANOVA test (the given *p*-values are valid for both thresholds, *i.e.*, > 0.1 and > 5 mm).

Rainfall Day	Geometric Mean [FC] (CFU/100 mL)		Geometric Mean [FC] (CFU/100 mL)		ANOVA (*p*-Value)
	Daily Rainfall		Daily Rainfall		
	<0.1 mm	≥0.1 mm	<5 mm	≥5 mm	
day_0	445	781	502	1030	<0.001
day_{-1}	436	767	493	1061	<0.05
day_{-2}	539	640	432	771	>0.05

These analyses demonstrate the influence of rainfall on the FC concentrations in the Beauport River (influence that is still noticeable up to one day after the rainfall occurred). This demonstrates that runoff has a major influence on FC concentrations in the river and supports the comparison of FC loads for different scenarios using a hydrological/hydraulic model conceived for the simulation of the rainfall-runoff processes (such as SWMM in our case).

3.2. Calibration of the SWMM Models

As stated previously, the SWMM models were previously calibrated by the Quebec City's Engineering Services department and afterwards slightly modified by the authors. Some partial results are presented here; more details can be found in [34,35,37].

3.2.1. Calibration of the Model for the Separate Stormwater System

To calibrate this model, flow rates were measured at four points in the separate sewer system and at two points in the river, from 17 August to 31 October 2009. Data from the CEHQ river gauging station (shown in Figure 2) were also used for calibration and validation of the model. Four rainfall events were selected to calibrate the model. Figure 5 shows an example of calibration results for a measuring point located in the separate sewer system.

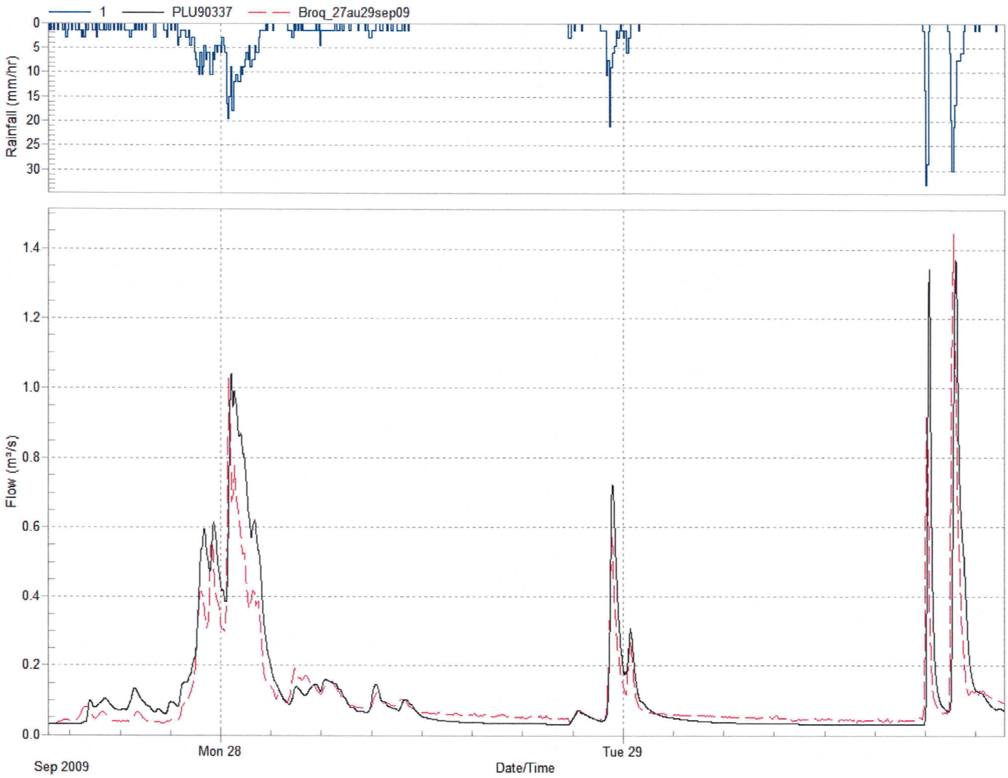

Figure 5. Example of calibration results at the Broqueville measuring point (black line = measured flow rate; red dashed line = simulated flow rate) (taken from [35]).

Since the SWMM model was conceived, calibrated and validated specifically for the modeling of urban runoff drainage, a base flow was added in the river by the authors in order to take into account the contribution of groundwater flow and headwater lakes. River flows simulated by the model were then compared to river flows measured at the CEHQ river gauging station using the Nash-Sutcliffe coefficient [38]:

$$NS = 1 - \frac{\sum_{i=1}^{n}(O_i - S_i)^2}{\sum_{i=1}^{n}(O_i - \overline{O})^2} \tag{3}$$

where: O_i = observation at time step i; S_i = simulated value at time step i; $\overline{O}$ = mean value of all observations; n = total number of time steps (NS may vary from $-\infty$ to 1 and is considered better when it gets closer to 1). Results of this comparison are summarized in Table 8.

Table 8. Nash-Sutcliff coefficient for the separate stormwater system at the CEHQ gauging station for the 1 May to 30 September period.

Year	Nash-Sutcliffe Coefficient
2008	0.63
2009	0.74
2010	0.89
2011	0.68

3.2.2. Calibration of the Model for the Combined Sewer System

As detailed in [34], the combined system model was calibrated based on flow rate measurement at 19 points in the sewer system from 29 May to 27 August 2009. Figure 6 shows an example of validation results.

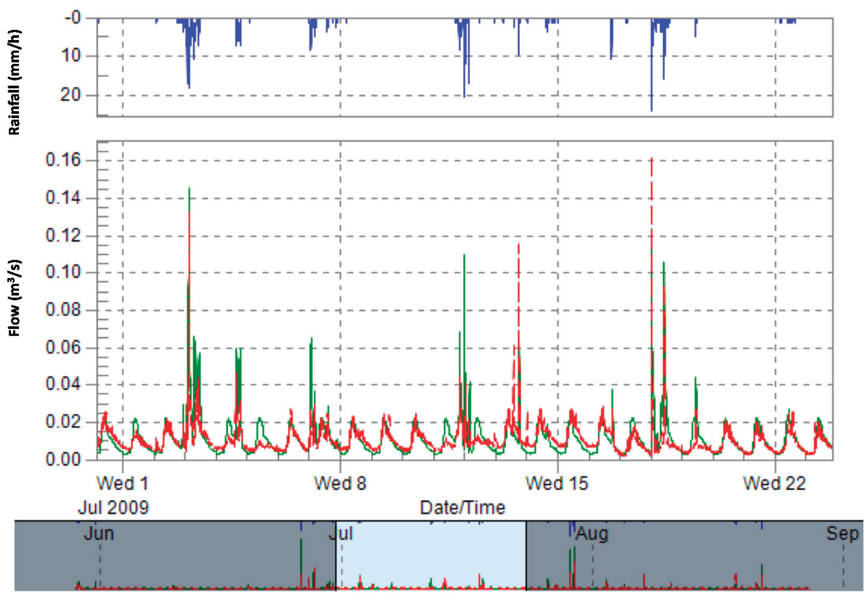

Figure 6. Example of validation results at the Giffard measuring point (red line = measured flow rate; green line = simulated flow rate) (taken from [34]).

The average absolute difference between simulated and observed flow at the 19 measurement points during the summer of 2009 was 18%. However, for the purpose of the analysis presented here, the model output that should be better calibrated is the total volume of CSOs that is discharged to the river. Consequently, some water level thresholds triggering overflows in the model were adjusted by the authors in order to match as closely as possible the number of simulated CSOs with the number of observed CSOs (recall that the volumes of CSOs were not recorded). Results are presented in Table 9.

Table 9. Comparison of the number of simulated CSOs with the number of observed CSOs for the 1 May to 30 September period.

Year	Number of CSOs			
	U051		U057	
	Simulated	Observed	Simulated	Observed
2008	30	28	64	62
2009	26	33	55	50
2010	17	15	48	47
2011	38	17 *	65	63

Note: * Errors are suspected in the number of observed overflows for the summer of 2011 based on a comparison with observed rainfall (see Table 1).

3.3. Comparison of Load Estimation Methods

The FC loads discharged to the Beauport River from the subwatersheds drained by the combined and separated sewer networks, as computed with the hydrologic/hydraulic simulation model, are illustrated in Figure 7. In this figure, it can be seen that the estimated contributions of the separate stormwater systems varied between 6.0×10^{13} and close to 1.6×10^{14} CFU per season. The contribution of the combined sewer system was higher, and varied from 5.1×10^{15} to 2.3×10^{16} CFU per season. From 2008 to 2011, the FC contribution from CSOs was as much as 100 times greater than the contribution from the stormwater drainage system, even though the total area drained by the combined sewer network (3.2 km^2) is much smaller than that covered by the separate stormwater drainage system (25.5 km^2). This means that priority intervention measures should be directed to the reduction of CSOs.

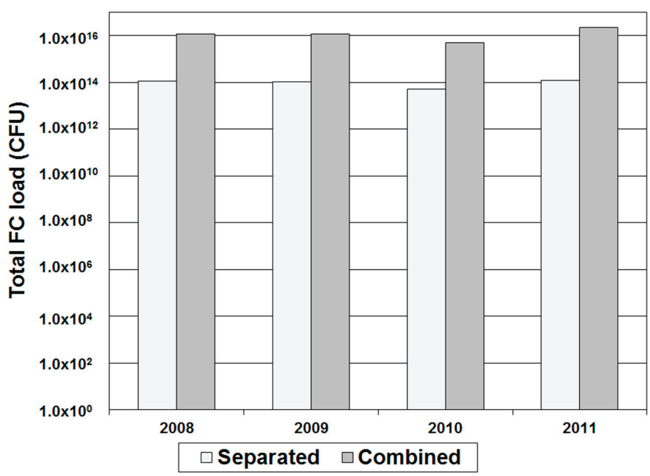

Figure 7. Estimated FC loads from the separate stormwater and combined sewer systems, for the May to August period.

As stated before, the Schueler's simple method [9] cannot be used in areas drained by a combined sewer network. Consequently, the loads evaluated by the two evaluation methods were compared only for the most upstream subwatersheds (illustrated in white in Figure 2 and covering a total area of 25.5 km²). Results of this comparison are given in Table 10.

Table 10. FC loads estimated by two methods for the 1 May to 31 August period.

Evaluation Method	Loads (CFU/Season)			
	2008	2009	2010	2011
Simple method	2.95×10^{14}	2.67×10^{14}	1.28×10^{14}	3.42×10^{14}
Hydrological/hydraulic model (SWMM)	1.14×10^{14}	1.07×10^{14}	5.05×10^{13}	1.26×10^{14}

Results in the previous table show that the FC loads estimated by the two methods are of the same order of magnitude. The simple method overestimates the loads by a factor of about 2.5 as compared with the hydrological/hydraulic modeling method (meaning that the runoff was overestimated in the simple method since the same EMCs were used with both methods). This demonstrates that the simple method is appropriate for a rapid estimation of the FC loads discharged by an urban drainage stormwater network. Indeed, one should recall that FC concentrations in urban waters commonly vary by many orders of magnitudes, and thus the computation of the same order of magnitude with the two methods is satisfactory, especially since the simple method is very easy and rapid to apply. However, the simple method cannot be used to evaluate intervention scenarios, as was done with the hydrological/hydraulic modeling method in the next section.

3.4. Analysis of Scenarios

Results presented in the previous section show that the FC discharged to the Beauport River mostly come from the combined sewer network (CSOs), but that the separate drainage network also contributes a significant quantity of FC to the river. The first step, to improve the water quality of the Beauport River to a level acceptable for recreational activities, should be the construction of retention tanks to reduce CSOs. However, this change may not be sufficient to reduce the FC concentrations below 1000 FCU/100 mL in the Beauport River during and after rainfall events. For this reason various stormwater management scenarios should be considered.

Figure 8 provides a visual comparison of the simulated FC loads discharged to the Beauport River on 26 July 2011 for scenarios S2 to S6. The contribution of scenario S1, not shown in Figure 8, is 5.18×10^{13} CFU (51.8×10^{12} CFU).

In decreasing order of total FC loads discharged to the river, the scenarios are ranked as follows: (1) The *status quo* (S1); (2) The retention of CSOs alone (S2); (3) the reduction in imperviousness (S4); (4) The primary treatment at some stormwater outfalls (S3); (5) The optimal management of stormwater (S5); and finally, (6) the compilation of all these intervention methods (S6). The last scenario reduced the total FC loads discharged to the river by a factor of 100 as compared with the reference scenario (S1) and by a factor of 10 for the reference scenario with the construction of retention tanks for CSOs (S2).

The simulated impacts of scenarios S2 to S6 on the FC concentrations in the Beauport River are illustrated in Figure 9. This figure shows that the compilation of all intervention methods (S6) is the only scenario to have reduced the FC concentrations below 1000 FCU/100 mL for 26 July 2011. The implementation of optimal measures for the management of stormwater combined with the construction of CSO retention tanks (S5) also reduced concentrations to near the 1000 FCU/100 mL objective.

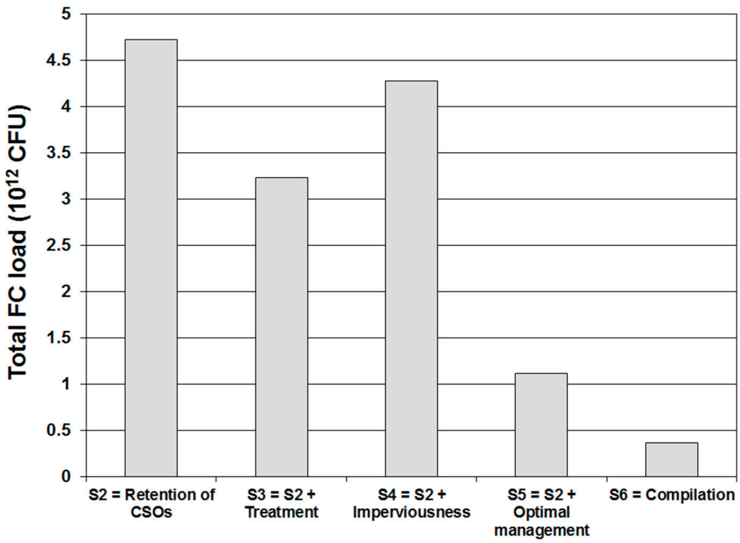

Figure 8. Comparison of the FC loads discharged to the Beauport River on 26 July 2011 according to various scenarios.

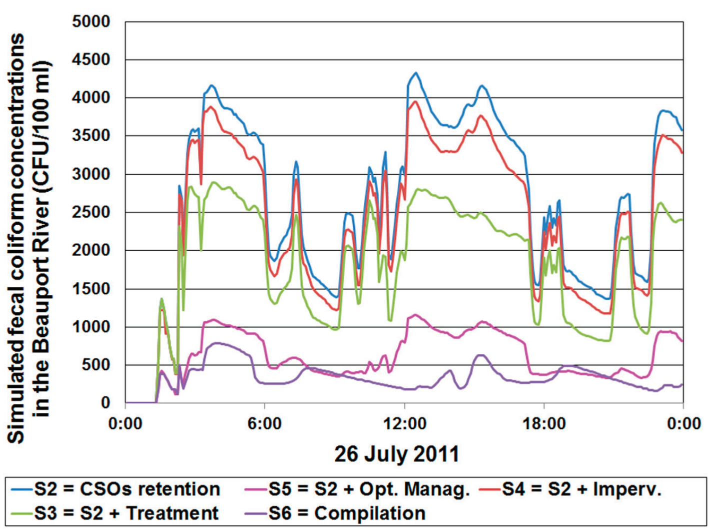

Figure 9. Simulation of water quality in the Beauport River on 26 July 2011 according to various scenarios.

These results demonstrate that although the construction of retention tanks for CSOs would be a major improvement, it alone would not be sufficient to guarantee suitable FC concentrations in the Beauport River during and after rainfall events. Many different best management practices should be combined and implemented in the watershed in order to reduce FC concentrations, as evidenced by the reduction provided by scenario S6.

It is important to note that the estimated FC loads and concentrations for scenario S3 are probably optimistic, since a 60% removal rate is assumed for FC in the stormwater retention basins, and this removal rate has been found to be null and even negative for FC in dry stormwater retention basins by many authors (e.g., [39]). Also, since EMCs may vary by many orders of magnitude for the same type of land use, the loads and concentrations that are estimated in this paper are subject to a high level of uncertainty and should be used only as a basis for comparisons between the various scenarios.

4. Conclusions

A three-step method for the identification of the main sources of fecal coliforms (FC) in urban waters and for the analysis of remedial actions was proposed. This method is based on the statistical analysis of the relationship between rainfall and FC concentrations in urban rivers, on the simulation of hydrology and hydraulics and on scenario analysis. The proposed method was applied, as an example, to the Beauport River watershed in Canada. Stormwater runoff in this watershed is drained by a separate sewer system in the upstream region and by a combined sewer system downstream. From this application we determined:

(1). In this watershed, there is a significant statistical relationship between the FC concentrations in the river and the amount of rainfall observed for the same day of the FC measurement and for the day before.

(2). Application of the Schueler's simple method [9] to the upstream part of the watershed led to seasonal FC loads of the same order of magnitude as those computed with a hydrological/hydraulic model combined with event mean concentrations (EMC).

(3). Combined sewer overflows (CSOs) are the main sources of discharged FC to the river.

(4). If retention tanks were built to contain CSOs on the watershed, FC from stormwater runoff would still impair recreational activities in the Beauport River.

(5). According to the scenario analysis, the major improvement that should be applied in the watershed to reduce FC concentrations in the Beauport River is the construction of retention tanks to contain CSOs (as planned by the City of Quebec).

(6). Optimal management of stormwater runoff, in order to reduce EMC at stormwater outfalls (e.g., correction of sewer cross connections, frequent road sweeping, regular cleaning of stormwater pipes, *etc.*) would provide the highest reduction in FC loads discharged to the river among the analyzed scenarios (including reduction of imperviousness and primary treatment at some stormwater outfalls). However, various intervention measures should be combined in order to reduce FC concentrations to a level acceptable for recreational activities in the Beauport River during and after rainfall events.

These conclusions were obtained using simulation models to compute FC loads and concentrations in the watershed. An important limit of these evaluations is that no FC concentrations were available in the Beauport River watershed other than in the river itself, in its downstream region. Consequently, EMC taken from the literature were used. Since EMC in urban runoff can vary by many orders of magnitude for the same type of land use, high uncertainties are linked to the FC loads and concentrations that were computed. Despite these uncertainties, main FC sources in the watershed could be identified, and the efficiency of various intervention measures could be compared. Installation of one or more additional monitoring stations in the river and at some stormwater outfalls would provide more accurate EMC and better estimates of the contribution of FC from stormwater runoff. The three step method proposed here could be applied with water quality components other than FC, provided that they are present in stormwater runoff and/or CSOs, and that the time of concentration of the watershed is significantly lower than their persistence in urban waters.

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Author Contributions

Sophie Duchesne and Amélie Thériault conceived and designed the methodology; Amélie Thériault performed the calculations and models simulations; Amélie Thériault and Sophie Duchesne analyzed the data; Amélie Thériault produced most of the figures and tables; Amélie Thériault and Sophie Duchesne wrote the paper together.

Conflicts of Interest

The authors declare no conflict of interest.

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Evaluating the Infiltration Performance of Eight Dutch Permeable Pavements Using a New Full-Scale Infiltration Testing Method

Floris Boogaard, Terry Lucke, Nick van de Giesen and Frans van de Ven

Abstract: Permeable pavements are a type of sustainable urban drainage system (SUDS) technique that are used around the world to infiltrate and treat urban stormwater runoff and to minimize runoff volumes. Urban stormwater runoff contains significant concentrations of suspended sediments that can cause clogging and reduce the infiltration capacity and effectiveness of permeable pavements. It is important for stormwater managers to be able to determine when the level of clogging has reached an unacceptable level, so that they can schedule maintenance or replacement activities as required. Newly-installed permeable pavements in the Netherlands must demonstrate a minimum infiltration capacity of 194 mm/h (540 l/s/ha). Other commonly used permeable pavement guidelines in the Netherlands recommend that maintenance is undertaken on permeable pavements when the infiltration falls below 0.50 m/d (20.8 mm/h). This study used a newly-developed, full-scale infiltration test procedure to evaluate the infiltration performance of eight permeable pavements in five municipalities that had been in service for over seven years in the Netherlands. The determined infiltration capacities vary between 29 and 342 mm/h. Two of the eight pavements show an infiltration capacity higher than 194 mm/h, and all infiltration capacities are higher than 20.8 mm/h. According to the guidelines, this suggests that none of the pavements tested in this study would require immediate maintenance.

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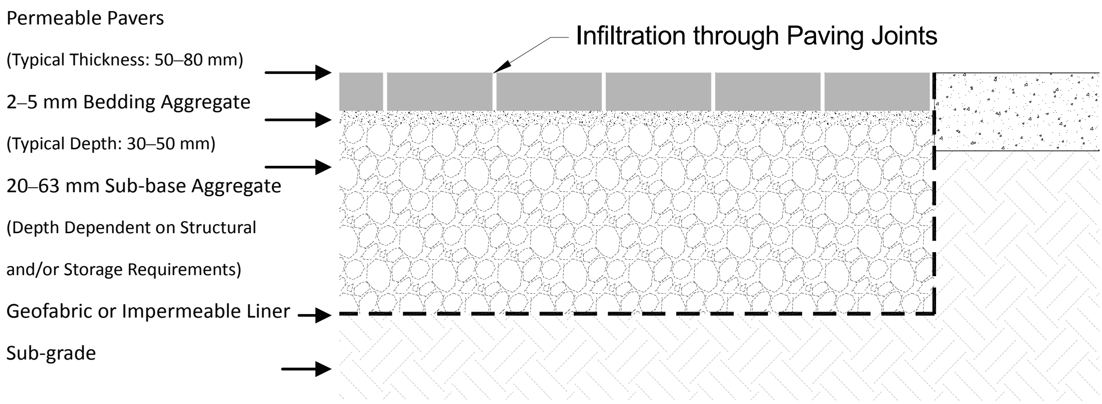
1. Introduction

Permeable (or porous) pavements are a type of sustainable urban drainage system (SUDS) technique that are used around the world to infiltrate and treat stormwater runoff. Permeable pavements are specifically designed to promote the infiltration of stormwater through the paving and basecourses, where it is filtered through the various layers (Figure 1). This can significantly reduce runoff volumes and discharge rates from paved surfaces [1–5] which can potentially minimise the risk of downstream flooding. Permeable pavements also provide considerable water quality improvements by treating and trapping stormwater pollutants [1,6–8].

There are several types of permeable pavements typically used in Europe, including concrete pavers with wide joints or apertures (Figure 2a) and porous concrete pavers, either with or without wide joints (Figure 2b). These are usually manufactured as blocks and are generally referred to as permeable concrete interlocking pavers (PCIP). Concrete and plastic grid pavers (CGP and PGP) are also often used in Europe. The design and function of CGPs and PGPs are similar to PCIP; however, the areas of the individual pavers are generally much larger than those used for PCIP systems. They also have more open void spaces to promote infiltration. Stormwater is able to infiltrate through the large gaps in these pavers, which are usually filled with gravel, or topsoil planted with grass (Figure 2c).

Figure 1. Typical permeable pavement structure.



Research has shown that urban stormwater runoff can contain significant concentrations of suspended sediments and gross pollutants [1,7,9]. Clogging is a result of fine, organic matter and traffic-caused abraded particles, blocking the gaps and surfaces of permeable pavement systems, due to physical, biological and chemical processes [8]. This clogging decreases the porosity/permeability of the paving surface and, hence, the infiltration rate of a system [9–11].

Figure 2. (**a**) Impermeable concrete PCIP (permeable concrete interlocking pavers); (**b**) porous concrete PCIPs; (**c**) grass-filled plastic grid pavers (PGPs).

It is important for stormwater managers to be able to determine when the level of clogging has reached an unacceptable level, so that they can schedule maintenance or replacement activities as required. In order to assess the reduction in infiltration capacity that occurs in permeable pavements over time due to clogging, a variety of infiltration test procedures have been utilised in the past. However, the results have generally been inconsistent and have shown a large variation in the range of infiltration rates measured [5,6,12–15]. As the number of global permeable pavement installations increases, a more reliable and more accurate method to measure surface infiltration rates is needed [16].

1.1. Infiltration Rate Testing

A number of previous permeable pavement infiltration studies [4,10,13,15] have been based on results using a modified version of either the single- or double-ring infiltrometer test (ASTM D3385-09) [17]. In these tests, rings are sealed to the pavement surface and filled with water. The time taken for the water to infiltrate through the permeable surface area is used to estimate an average infiltration rate (usually in mm/h) for the test location. Both the constant head and the falling head methods can be utilised in these testing procedures. Double-ring infiltrometer tests (DRIT) have generally been the preferred method in the past. This is because the outer ring is thought to reduce measurement errors and to prevent lateral flow from occurring beneath the rings. However, on pavements where the infiltration rate is so high that it is difficult to supply enough water to both rings, the single-ring surface infiltration test [4] has been used (Figure 3c).

Three variations of ring infiltrometers used in past permeable pavement studies are shown in Figure 3. Other permeable pavement infiltration research has been undertaken using specially fabricated rainfall simulation infiltrometers [6,9]. A new Standard Test Method for the Surface Infiltration Rate of Permeable Unit pavement Systems (ATSM C1781M-13) [18] has recently been published. However, to date, there have been no studies published using this method.

Figure 3. Modified ring infiltrometers used for permeable pavement testing: (**a**) double-ring infiltrometer tests (DRIT) [15]; (**b**) square, double-ring [13]; (**c**) single-ring surface inundation test [4].

The permeable pavement infiltration testing methods described above are based on the infiltration rate through a very small area of the pavement that is used to represent the total pavement area infiltration. For example, the area of the inner ring of the ASTM D3385-09 [17] DRIT test is 0.0707 m^2. The minimum area recommended by the Dutch guidelines [19] is even smaller, at only 0.01 m^2. Using such small areas for testing could potentially lead to erroneous results, as a number of studies have demonstrated a high degree of spatial variability between different infiltration measurements undertaken on the same pavement installation [4,9,13,20]. It was hypothesised that more accurate infiltration results may be produced by significantly increasing the area of the pavement surface being tested. By inundating a much larger area of pavement during testing, it was anticipated that any spatial variations in infiltration capacity would be averaged-out, and this would produce more reliable infiltration data.

In order to test this hypothesis, this study developed and trialled a new, full-scale infiltration testing method. Using the new method, it was possible to test the infiltration capacity of large sections of existing permeable pavements at one time. This paper describes the new experimental test procedure developed in the Netherlands to more accurately determine the surface infiltration rate of existing permeable pavement installations. The results from eight test locations in the Netherlands using the new infiltration testing method are presented and compared to national guideline requirements.

2. Methodology

In order to evaluate the performance of the new, full-scale infiltration testing method, the method was first trialled on an existing permeable pavement street installation that had been in service for over seven years in Utrecht in the Netherlands. The results of the initial testing were successful [21] and showed that the new method could be used to accurately measure infiltration rates of permeable pavements *in situ* after full-scale testing and tests with ring infiltrometers. The new testing method was therefore used on the eight existing pavements in five different municipalities evaluated in this study. The testing methodology for the eight test locations in the Netherlands is discussed in the following sections.

2.1. Test Area Selection

To enable an accurate estimation of the average surface infiltration rate using the new test method, a permeable pavement area of approximately 50 m^2 was recommended for all tests. This minimum area is recommended in order to obtain a good representation of the whole surface and to minimise any potential leakage problems. Roads in the Netherlands are typically five meters wide, which means the minimum length of the test pavements should ideally be at least 10 m (5 m × 10 m = 50 m^2). This area is over 700-times greater than the area of the inner ring used in typical infiltrometer tests. However, achieving this was dependent on site practicalities, such as pavement width, length, slope and cross-fall, the location of drainage gullies, parked cars and resident access requirements. It should be noted that in order to undertake the testing, it was necessary to close the section of pavement for a number of hours. It is therefore recommended that local council permission be obtained before any testing is conducted.

2.2. Water Containment

To accurately define the infiltration testing area and to contain the water used to infiltrate the pavement, it was necessary to construct small, temporary dams at the ends of the pavement test sections. The roadway kerb and gutter system retained the water on the sides of the pavement test sections. A number of dam variations were trialled at the eight different test locations (Figure 4). These included:

1. Soil core wrapped in plastic sheeting;
2. Sand core wrapped in geotextile;
3. Soil- or sand-filled plastic bags;

4. Impermeable barriers inserted into paving gaps; and
5. Use of existing traffic calming devices (speed-humps).

Figure 4. Various dam variations used at the different test locations; (**a**) impermeable barriers; (**b**) plastic wrapped soil core; (**c**) soil-filled plastic bags.

2.2.1. Recommendations

Where possible, one of the preferred methods of containing the water within the test site is to choose a section with an existing raised traffic calming device (speed hump) at one (or both) ends. This saves considerable setting-up time and also minimises leakage problems during testing. It is also advisable to select the section of pavement with the least number of existing drainage gullies within the pavement surface or gutter. Drainage gullies need to be properly sealed to prevent water from leaking from the test area and entering the underground stormwater drainage system. This can be both difficult to accomplish and time consuming. Of all the methods trialled to create temporary dams, the soil-filled were found to be the most effective. This was due to their ability to properly seal the test sections, the rapid filling and emptying characteristics of the bags, the ability to reuse the material and the ease of construction by hand without the need for heavy machinery.

2.3. Water Supply

The new infiltration test requires large volumes of water to be discharged onto the test paving section in order to inundate the pavement surface. Depending on the site location, a number of different water supply options were trialled in this study, including transporting water directly to the site with water trucks (Figure 5a) or water tanks (Figure 5b) and pumping water directly from nearby canals (Figure 5c).

Figure 5. (**a**) Water truck supply; (**b**) water tank supply; (**c**) pumping from canal.

After the pavement test area had been selected and sealed with temporary dams, the pavement area was inundated with water to the maximum allowable water level possible that would not cause overtopping of the roadway kerb and gutter system. The maximum inundation depth was dependent on the type of construction. However, this was generally between 50 and 90 mm from the lowest point in the pavement to the top of the gutter. Due to the different levels of the pavement surface, this meant that the depth of water in the inundated test section was dependent on the measurement location, with the lowest pavement elevation generally having the highest inundation water levels.

2.3.1. Recommendations

Of the three water supply methods trialled, it was found that pumping the water from a nearby canal was the easiest option, where this option was available. This method offered total flexibility with types of testing and also offered an unlimited availability of water. It is recommended to include a flowmeter in the water supply line to allow accurate monitoring of water inflow rates. Water trucks were the second easiest option. However, these had the disadvantages of being expensive and difficult to arrange, manoeuvre and park, and they generally had only limited water supply capacity. When a water truck must be used, it is advisable to ensure that the outlet is fitted with a flowmeter to measure flow rate into the test pavement area.

2.4. Determining Pavement Infiltration Rates

Pressure transducers were used in the study as the primary method of measuring and recording the reduction in water levels over time at various locations on the pavement surface. Two wireless, self-logging pressure transducers were installed at the lowest points on the left-hand and right-hand sides of each test pavement area (Figure 6a). The transducers continuously monitored the static water pressures at those locations and transmitted this information to a laptop computer. The static water pressure was then converted to an appropriate depth of water above the pavement. This process produced accurate and reliable data over the duration of the tests. It also enabled visual representation of the pavement infiltration process.

Three different measurement methods (Figure 6) were used in conjunction with the pressure transducers in order to calibrate and verify the transducer readings. The three methods were:

1. Hand measurements;
2. Calibrated underwater camera;
3. Time-lapse photography.

2.4.1. Hand Measurements

Water level measurements were taken using a simple 300-mm hand ruler (Figure 6b) at strategic locations on the pavement surface throughout the duration of the testing. These measurements were used to verify the functionality and accuracy of the self-logging pressure transducers, as described above. Photographs of each hand measurement were also taken for documentation and verification purposes.

Figure 6. (**a**) Minidiver installed at lowest point of pavement; (**b**) hand measurement point; (**c**) underwater camera set-up; (**d**) underwater camera view.

These three methods are explained in more detail below.

2.4.2. Calibrated Underwater Camera

A high-definition video camera was also used at a number of strategic locations to record the decrease in pavement water levels over the duration of the tests. The camera was placed inside a waterproof, calibrated, transparent box, so that it could capture the entire infiltration process (Figure 6c). This system allowed real-time monitoring of the entire infiltration process and also facilitated precise verification of the pressure transducer measurements.

2.4.3. Time-Lapse Photography

Time-lapse photography was used at each test location to record all research activities and to enable verification of the pressure transducer and hand measurements. The time-lapse photographs were also used to compile an accelerated video of the entire pavement testing.

2.4.4. Recommendations

While pressure transducers and loggers provide an abundance of data and allow informative and attractive graphs to be complied, much care needs to be taken to ensure that the pressure transducer readings are verified and accurate. Pressure transducers can be unreliable and inaccurate. They have also been shown to be sensitive to external influences, such as wind effects and changes in atmospheric pressures [21]. Therefore, the high frequency data from pressure transducers is useful for a detailed infiltration curve, but it is highly recommended that transducer readings are calibrated and verified using at least one of the other methods described above.

2.5. Study Test Locations

The infiltration rates of eight existing permeable pavements in the Netherlands were tested in the current study. The locations and details of the pavements are listed in Table 1. All test locations are located in residential areas (30 km/h zones). No maintenance other than street sweeping has taken place at the locations. All tests were carried out after an antecedent dry period of at least three days.

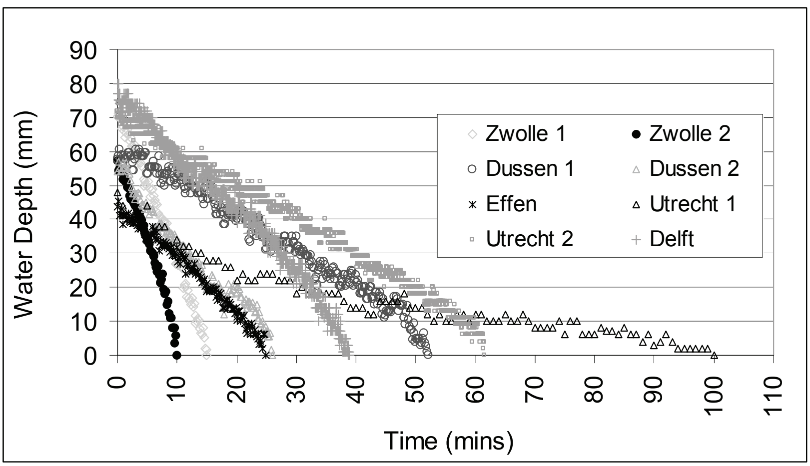
Table 1. Permeable pavement locations tested in the Netherlands.

Test location	Street name	Type of pavement	Year of construction	Test area (m^2)	Test date
Zwolle 1	Pieterzeemanlaan	Porous Concrete PCIP	2006	44.2	11/15/2013
Zwolle 2	Pieterzeemanlaan	Porous Concrete PCIP	2006	39.9	11/15/2013
Dussen 1	Groot Zuideveld	Impermeable Concrete PCIP	2006	59.5	10/23/2013
Dussen 2	Groot Zuideveld	Impermeable Concrete PCIP	2006	69.7	10/23/2013
Effen 1	Baanakker	Impermeable Concrete PCIP	2006	29.4	10/30/2013
Utrecht 1	Nijeveldsingel	Impermeable Concrete PCIP	2006	51.9	11/28/2012
Utrecht 2	Brasemstraat	Impermeable Concrete PCIP	2006	60.0	06/13/2013
Delft 1	Drukkerijlaan	Impermeable Concrete PCIP	2005	74.0	06/19/2013

2.6. Calculating Infiltration Rates

All eight test pavements (Table 1) were sealed, inundated and monitored as described above. The pressure transducer readings were then plotted against time to generate precise infiltration curves for each of the test sites (Figure 7). Simple linear regression analysis was used to generate lines of best fit for the transducer readings from each site. The equations of the linear regression lines were then used to calculate the average infiltration rate in mm/h for each test site (Table 1).

Figure 7. Infiltration curve results for the eight permeable pavements tested in the study.



3. Results

The surface infiltration rates recorded for each of eight test pavements using the new experimental test procedure are shown in Figure 7.

The linear regression analysis results for the eight test pavement measurements are listed in Table 2.

Table 2. Linear regression analysis results for the eight test pavements.

Test location	R^2	Equation	Max water level (mm)	Total time (mins)	Calculated infiltration (mm/h)	Percentage of recommended EU value (194 mm/h)
Zwolle 3	0.9844	$y = -5.211x + 58.935$	57	10	342	176%
Zwolle 1	0.9928	$y = -4.634x + 73.373$	71	15	284	146%
Dussen 2	0.9624	$y = -1.8498x + 52.742$	57	26	132	68%
Delft 1	0.9821	$y = -1.8195x + 77.848$	80	39	124	64%
Effen 1	0.9837	$y = -1.6099x + 44.451$	45	25	109	56%
Utrecht 2	0.9792	$y = -1{,}031x + 70.576$	72	61	71	36%
Dussen 1	0.979	$y = -1.0572x + 61.858$	60	52	69	35%
Utrecht 1	0.8826	$y = -0.3577x + 34.154$	48	100	29	15%

4. Discussion

Although the eight permeable pavements tested in this study were of a similar construction type and of similar age, Table 2 shows a large variation in the calculated infiltration rates between the eight study pavements. This variation in results is similar to the findings of a number of previous studies that have attempted to quantify the infiltration rates of permeable pavements [4,13,16,21–23]. The infiltration rates of the eight test pavements differed from between 29 and 342 mm/h.

There are a number of potential reasons for the observed variations in the surface infiltration rates between the test pavements, including:

- Age: although most of the pavements were generally of a similar age range, it would be reasonable to expect small variations in surface infiltration capacity in the older pavements.
- Construction: While the construction of the test pavements were generally similar to that shown in Figure 1, there were slight differences between the sites. These included the size of the paving joints, different types of bedding aggregates and different pavement laying processes.
- Maintenance: There were distinct variations in the pavement maintenance procedures between the different municipalities. Some municipalities conducted occasional street sweeping of their permeable pavements. However, as this was done to all pavements, this is generally not considered as targeted maintenance to improve the permeable pavement performance and to reduce clogging.
- Variations in hydraulic ground conditions: The water table was higher at some pavement test locations (particularly in the western areas of the Netherlands), while the permeability of soils in the eastern test locations were generally higher.
- Environmental site conditions: The type and amount of trees surrounding the pavements were not the same. Trees are known to affect the infiltration rate of permeable pavements [15]. Other test pavement locations may have been affected by the close proximity of industrial areas.
- Pavement usage: There were distinct variations observed between the type and number of vehicles using the different pavements on a daily basis.

4.1. Dutch Permeable Pavement Infiltration Guidelines

Guidelines for the construction and performance of permeable pavements are generally limited in the Netherlands. However, guidelines on acceptable infiltration rates for newly-installed permeable concrete pavement systems in the Netherlands have been developed by Kiwa Nederland [19] in 2014, and local government engineers and designers often refer to these guidelines when designing new permeable pavement systems. Recently published Kiwa permeable pavement infiltration testing guidelines [19] stipulate the following:

> "A minimum of three infiltration tests shall be performed. If all three tests demonstrate an average infiltration rate of equal to or greater than 194 mm/h (540 L/s/ha), the pavement is deemed to comply."

A number of other European countries also have construction and infiltration guidelines for concrete permeable pavements. Newly-installed permeable pavements systems in the Netherlands, Belgium and Germany all need to demonstrate an infiltration capacity of 194 mm/h [24–26]. Every test should demonstrate a minimum infiltration rate of 97 mm/h.

The overall infiltration rates calculated for six of the eight pavements tested in this study were below the Kiwa recommendation of 194 mm/h (Table 2). Other permeable pavement guidelines in the Netherlands [27] recommend that maintenance is undertaken on permeable pavements when the infiltration falls below 0.5 m/d (20.8 mm/h). According to these guideline values, none of the pavements in Table 2 would require immediate maintenance. Previous studies have demonstrated that infiltration rates that have diminished over time due to clogging can be restored by undertaking pavement maintenance, such as street sweeping and vacuum cleaning [4,6,28].

An interesting outcome from the study was the differences in perceptions between the various maintenance personnel regarding the measured infiltration rates of the test pavements within their municipalities. Interviews were conducted with a variety of maintenance personnel from the different municipalities where the full-scale tests were performed in order to ascertain their opinions on the infiltration performance of the pavements. For example, some of the people interviewed were satisfied with a low infiltration rate just above the 20.8 mm/h corresponding to the RIONED [27] recommendations. However, others were disappointed with the relatively high infiltration rate, as it was just above the KIWA [19] guideline of 194 mm/h, and they expressed concern that this value would reduce over time.

Infiltration rates of newly-installed permeable pavement systems have been shown to be very high. However, this has been shown to decrease significantly over time [9,12,13,23], and it is the long-term infiltration performance of a pavement that determines their ultimate success or failure [11]. Whether the surface infiltration rate obtained from testing is considered acceptable or not depends on a number of factors, including the location of the pavement, the intended purpose of the pavement and the stakeholder expectations. Most stakeholders in the Netherlands expect a life span of 20 to 60 years, comparable with the life span of conventional stormwater drainage infrastructure. Most roads in the Netherlands will be reconstructed within 20 years. From this data, it should be considered to test the pavement right after construction and every five years. Our suggestion is that municipalities should plan to undertake maintenance after about 10 years of continuous use.

5. Conclusions

This study used a newly-developed, full-scale infiltration test to evaluate the infiltration performance of eight permeable pavements in five municipalities that had been in service for over seven years in the Netherlands. Traditional permeable pavement infiltration testing methods generally base results on the infiltration rates obtained through a very small area of the pavement, which is then used to represent the total pavement area infiltration. This approach of using small areas for testing could potentially lead to erroneous results being obtained. This study tested the hypothesis that more accurate infiltration results may be produced by significantly increasing the area of the pavement surface being tested. An earlier study on one location in Holland demonstrated that the newly-developed, full-scale infiltration testing methodology was successful and produced reliable surface infiltration results [21]. Issues that need to be considered when using the new test method are also presented in the paper.

Infiltration rates of newly-installed permeable pavement systems are generally very high, although they have been shown to decrease significantly over time. Newly-installed permeable pavements in the Netherlands must demonstrate a minimum infiltration capacity of 194 mm/h. This study found that only two of the measured infiltration results of the eight tested pavements were above the 194 mm/h requirement. Other permeable pavement guidelines in the Netherlands recommend that maintenance should be undertaken on permeable pavements when the surface infiltration falls below 20.8 mm/h. According to these guideline values, none of the eight pavements tested in this study would require immediate maintenance.

While the results of the study may initially appear discouraging at first, the study found that whether the results were considered acceptable or not depended on a number of factors. These included the location of the pavement, the intended purpose of the pavement and the stakeholder expectations and perceptions. The authors advise testing the pavement right after construction and again after five years to estimate the clogging rate of the pavement. Municipalities should plan to undertake maintenance around 10 years of continuous use. The findings of this study will help planning the required maintenance of the pavements with more confidence so that they will continue to perform over their intended design life.

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Author Contributions

This study was undertaken as a collaborative research project by the Delft University of Technology, the University of the Sunshine Coast in Australia, Hanze University of Applied

Sciences and by TAUW in the Netherlands. The experimental design of the project was undertaken by Floris Boogaard, Terry Lucke, Frans van de Ven and Nick van de Giesen. The majority of the experimental field work was conducted by Floris Boogaard with assistance from Terry Lucke. The paper was written by all four authors equally.

Conflicts of Interest

The authors declare that they have no conflict of interest.

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Field Study of Infiltration Capacity Reduction of Porous Mixture Surfaces

Luis A. Sañudo-Fontaneda, Valerio C.A. Andrés-Valeri, Jorge Rodriguez-Hernandez and Daniel Castro-Fresno

Abstract: Porous surfaces have been used all over the world in source control techniques to minimize flooding problems in car parks. Several studies highlighted the reduction in the infiltration capacity of porous mixture surfaces after several years of use. Therefore, it is necessary to design and develop a new methodology to quantify this reduction and to identify the hypothetical differences in permeability between zones within the same car park bay due to the influence of static loads in the parked vehicles. With this aim, nine different zones were selected in order to check this hypothesis (four points under the wheels of a standard vehicle and five points between wheels). This article presents the infiltration capacity reduction results, using the LCS permeameter, of Polymer-Modified Porous Concrete (9 bays) and Porous Asphalt (9 bays) surfaces in the University of Cantabria Campus parking area (Spain) 5 years after their construction. Statistical analysis methodology was proposed for assessing the results. Significant differences were observed in permeability and reduction in infiltration capacity in the case of porous concrete surfaces, while no differences were found for porous asphalt depending on the measurement zone.

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1. Introduction

Intense urban growth during the last decades [1], together with large-scale waterproofing of the natural soil in cities [2] and changes in the rainfall intensity patterns in the world [3], have led to many problems regarding flooding. This is actually the most common and costly disaster in the world [4,5].

Porous surfaces are one of the main Sustainable Urban Drainage Systems (SUDS) for source control in car park areas [6–8]. Many different devices have been used to measure the infiltration capacity on-site. Some of the most widely applied field devices nowadays around the world are single-ring infiltrometers [9], double-ring infiltrometers [10], and the LCS ("Laboratorio de Caminos de Santander") permeameter [11,12].

Previous studies, [13], highlighted the importance of analyzing different zones within a car park bay in order to obtain a more comprehensive view of real infiltration behavior in a car park with porous surfaces. The static and dynamic loads produced by the vehicle wheels can produce permanent deformations in the pervious surface, which could affect both porosity and permeability. Moreover, the vehicle wheels are the main source of particulate matter that can clog the pervious surfaces, especially due to the compaction force produced by the vehicle loads, this effect being more important in the contact zone between wheel and surface [14,15].

A new methodology was created for this research. Firstly, a scheme of field tests was created by using the LCS permeameter to measure the permeability, and finally, a statistical scheme of several analyses was designed and developed specifically for this kind of on-site test.

The aim of the new methodology presented in this paper was twofold. Firstly, the analysis of the influence of the porous mixture surface type on the permeability and the reduction in the infiltration capacity after 5 years of use. Secondly, the analysis of the possible differences in the infiltration capacity in different zones within the pervious parking bays.

2. Experimental Methodology

The whole study was carried out in the "Las Llamas" parking area in the University of Cantabria campus in Santander (Spain) 5 years after this car park was opened for light traffic. No maintenance operations have been carried out during this period. This parking area registers intense traffic activity every day, being nearly 100% occupied. Eighteen car parking bays of 4.2 m long and 2.4 m wide were analyzed with nine bays of Polymer Modified Porous Concrete (PMPC) and nine of Porous Asphalt (PA) surfaces (Figure 1).

The specific characteristics of the two porous mixture surface materials used can be checked in [12] based on the dosage recommended by [16] for PMPC, and [17] for PA. The high percentage of voids is remarkable, 25%–30% in the case of PMPC and 23% in the case of PA [12], as was the thickness of both porous surfaces (80 mm).

Figure 1. (**A**) Scheme of the eighteen car parking bays analyzed; and (**B**) measurement zones selected within each car park bay and LCS on-site.

The infiltration capacity reduction was analyzed through the permeability results obtained now (after 5 years of operational life) in each test carried out using the LCS permeameter [18], comparing these values with those registered by [12] for the same porous surfaces when built (0.020 m/s for the case of the PMPC surfaces and 0.012 m/s for PA surfaces on average).

Nine different points were selected within each car parking bay in order to undertake the LCS tests. Each point represents a specific zone (Figure 1) which hypothetically could influence the infiltration capacity reduction. Points 1, 3, 7 and 9 (Figure 1) represent the zone of the car parking bays in static contact with wheels, the zones that directly support the weight of the vehicles when parked. In contrast, points 4 and 6 (Figure 1) represent the zones that were in dynamic contact with wheels while a vehicle is performing its parking maneuver, being part of the wheels path. Finally, points 2, 5 and 8 (Figure 1) represent the zones that almost never have been in contact with vehicles tires.

2.1. Descriptive Analysis

The permeability results in the tests were partially described based on permeability ranges defined by [19] for porous asphalt surfaces when using the LCS permeameter. Each measurement zone in every car park bay and all car park bays received a score based on the time taken by the LCS test, using the criteria in Table 1. Moreover, plots of the average values of the outcome variables (permeability and reduction of the infiltration capacity) were used to analyze descriptively the infiltration behavior of the whole car parking area studied.

Table 1. Criteria for defining the permeability of a porous mixture surface when using the LCS permeameter.

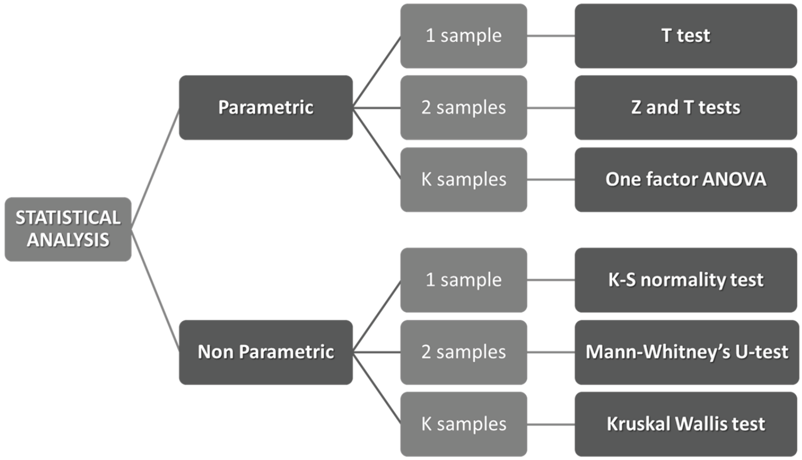
Time (s)	Permeability (cm/s)	Score
<50	>0.50	Newly built
50–100	0.25–0.50	High
100–200	0.13–0.25	Medium
>200	<0.13	Poor

2.2. Statistical Analysis

To achieve the objectives explained in the introduction, a statistical methodology was designed, as can be seen in Figure 2.

The statistical approach begins with the analysis of the normality distribution of the data in order to decide the path to follow in the statistical scheme in Figure 2: Parametric test for normally distributed data and non-parametric test for non-normally distributed parameters. Then, a more in-depth statistical analysis was done based on different significance tests (see Figure 2) with the aim of determining whether there are significant differences among the results obtained for the variables considered.

Figure 2. Scheme of the statistical methodology designed.



3. Results and Discussion

3.1. Descriptive Analysis

The distribution of the permeability values registered using the LCS permeameter at each measurement point of the analyzed parking bays of both types of pervious surfaces is in Figure 3.

It can be observed descriptively that there are differences in the infiltration capacity among the different measurement zones on both types of pervious surfaces, generally showing a reduction in infiltration capacity in some wheel-surface contact zones. Considering the average permeability values in each measurement zone of each pervious surface type, the average reductions of the infiltration capacity were calculated and the results are shown in Tables 2 and 3 for the PMPC and PA surfaces, respectively.

Although the average value of the PMPC surface infiltration capacity demonstrated a high decrease of 79.43% (Table 2), the average permeability value is still high (0.41 cm/s). This value can be considered "high" in the score classification based on the criteria shown in Table 1. A highly similar decrease in the average reduction of the infiltration capacity was found in Table 3 for the PA surface (82.04%). However, the average score was "medium" for PA surfaces. This indicated possible problems in the future with the permeability behavior of this surface.

As can be seen in the box-plots in Figure 4, average PMPC permeability was almost double that of PA (0.41 cm/s for PMPC and 0.22 cm/s for PA), while the reduction in the infiltration capacity on both porous mixture surfaces was quite similar (79.43% in the case of the PMPC surface and 82.04% in the case of the PA surface).

Figure 3. Box-plots of the average values of permeability in each measurement zone of parking bays.

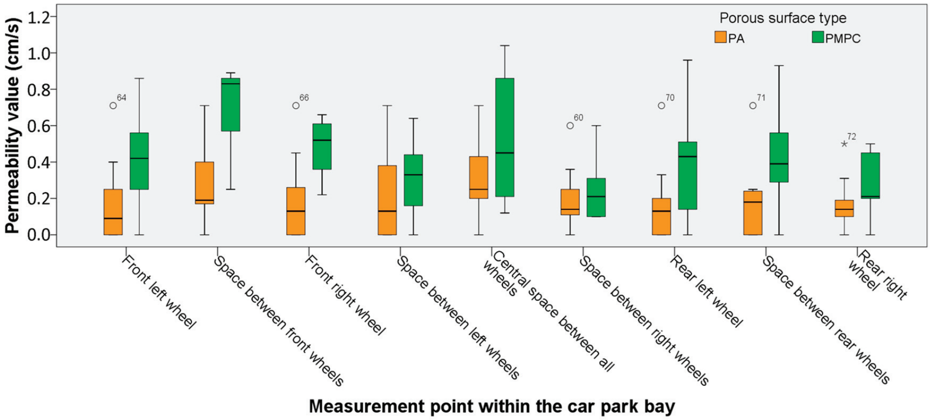


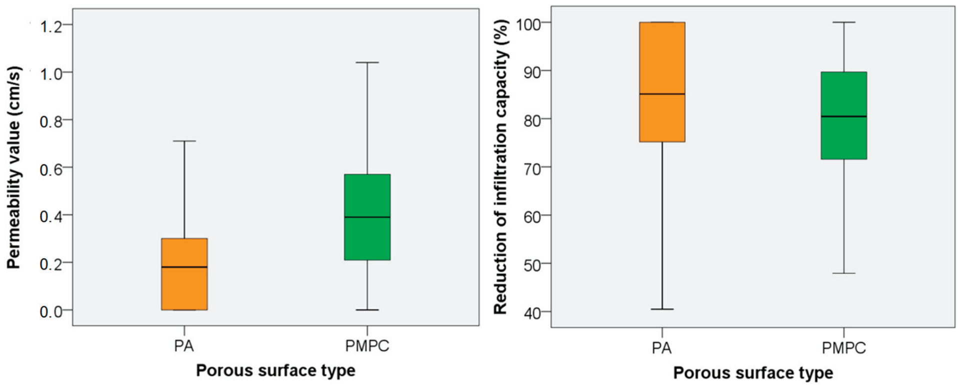
Table 2. Average permeability and reduction of the infiltration capacity values registered in each measurement zone within each Polymer Modified Porous Concrete (PMPC) surface car park and their corresponding score.

Measurement zone	Permeability (cm/s)	Score	Reduction of the infiltration capacity (%)
1	0.41	High	79..65
2	0.69	Newly built	65.62
3	0.47	High	76.45
4	0.31	High	84.47
5	0.54	Newly built	73.22
6	0.25	High	87.62
7	0.39	High	80.67
8	0.40	High	79.97
9	0.26	High	87.24
Mean value	0.41	High	79.43

Table 3. Average permeability and reduction of the infiltration capacity values registered in each measurement zone within each Porous Asphalt (PA) surface car park and their corresponding score.

Measurement zone	Permeability (cm/s)	Score	Reduction of the infiltration capacity (%)
1	0.20	Medium	83.52
2	0.27	High	77.46
3	0.21	Medium	82.40
4	0.22	Medium	81.70
5	0.30	High	74.85
6	0.21	Medium	82.57
7	0.17	Medium	85.61
8	0.18	Medium	85.05
9	0.18	Medium	85.23
Mean value	0.22	Medium	82.04

Figure 4. Box-plots of the average values of permeability (**A**) and the reduction of the infiltration capacity (**B**).



(**A**) (**B**)

3.2. Statistical Analysis

The first step was to check the normality of both outcome variables by using the Kolmogorov-Smirnov test. Neither variable had a normal distribution. Therefore, non-parametric significance analyses were carried out (Figure 2), by using a Mann-Whitney test for the type of porous mixture surface (two samples: PMPC and PA) and a Kruskal Wallis test for the measurement zone (9 samples: zones 1 up to 9) (Table 4).

Significance tests shown in Table 4 demonstrate that only the type of porous mixture surface significantly influenced permeability results, while neither the type of porous surface nor the measurement zone influenced the reduction in infiltration capacity.

Table 4. Mann-Whitney and Kruskal Wallis significance tests for the outcome variables.

Significance test	Parameter	Permeability	Reduction of the infiltration capacity
Mann-Whitney *	U de Mann-Whitney	1888.5	2716.5
	Asymptotic significance	0.000	0.058
Kruskal Wallis **	Square Chi	12.493	13.329
	Asymptotic significance (bilateral)	0.131	0.101

Notes: * Grouping variable: type of porous mixture surface; ** Grouping variable: measurement zone.

Once the influence of the porous mixture surfaces has been demonstrated in Table 4, it is only necessary to verify the real influence of the measurement zone on the outcome variables for each type of porous mixture surface. With this aim, the normality and homoscedasticity of both outcome variables was analyzed as an initial step. PMPC surface permeability and reduction in infiltration capacity results were distributed according to a normal and homoscedastic distribution, while in the case of the PA surface, these results were not normal. Thus, in order to use the same test for both types of pervious surfaces, a Kruskal Wallis test was done to analyze the influence of the measurement zone on the outcome variables (Table 5) based on the statistical scheme shown in Figure 2.

Table 5. Significance analyses for k-independent samples (measurement zones) by using Kruskal Wallis test.

Type of surface	Statistical Significance Test	Permeability	Reduction of the infiltration capacity
PMPC	Square ChiChi	17.752	17.742
	Significance (bilateral)	0.023	0.023
PA	Square Chi	4.397	4.522
	Significance (bilateral)	0.820	0.807

Note: Grouping variable: measurement zone.

The results shown in Table 5 demonstrate the influence of the measurement zone on permeability values and on the reduction in infiltration capacity obtained after 5 years of use in car parking bays made of PMPC. However, in the case of PA, no influence was identified.

Therefore, both the statistical methodology and the measurement zones shown in this article can be used for present and future research when using the LCS permeameter to study the infiltration behavior of porous mixture surfaces on-site during their operational life.

4. Conclusions

The statistical methodology described in this article has proven its efficiency in this particular scenario. Therefore, this methodology could be used in similar investigation in order to prove the general suitability of materials used in infiltration surfaces.

In this field study, permeability is significantly different for PMPC and PA surfaces after 5 years of use, as it was at the beginning of their operational life, the PMPC surfaces having higher permeability values.

No significant differences were found between PMPC and PA surfaces regarding their infiltration capacity reduction after 5 years of use.

The measurement zones proposed for this research for analyzing the infiltration capacity behavior of a porous surface car parking bay after 5 years have demonstrated a significance influence of the zone on permeability results for PMPC surfaces.

No significant differences were identified among all the measurement zones for PA surfaces, its infiltration behavior being quite uniform after 5 years of use.

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Author Contributions

The four co-authors designed the research methodology, specifically Luis A. Sañudo-Fontaneda and Valerio C.A. Andrés-Valeri carried out the test in the car park and the statistical analysis of the results, while Jorge Rodriguez-Hernandez and Daniel Castro-Fresno validated the analysis, lead the discussion and stated the final conclusions.

Conflicts of Interest

The authors declare no conflict of interest.

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Review and Research Needs of Bioretention Used for the Treatment of Urban Stormwater

Jia Liu, David J. Sample, Cameron Bell and Yuntao Guan

Abstract: The continued development of urban areas in recent decades has caused multiple issues affecting the sustainability of urban drainage systems. The increase of impervious surface areas in urban regions alters watershed hydrology and water quality. Typical impacts to downstream hydrologic regimes include higher peak flows and runoff volumes, shorter lag times, and reduced infiltration and base flow. Urban runoff increases the transport of pollutants and nutrients and thus degrades water bodies downstream from urban areas. One of the most frequently used practices to mitigate these impacts is bioretention. Despite its widespread use, research on bioretention systems remains active, particularly in terms of mix design and nitrogen treatment. Recent research focusing on bioretention is reviewed herein. The use of mesocosms provides the ability to isolate particular treatment processes and replicate variability. Computational models have been adapted and applied to simulate bioretention, offering potential improvements to their operation, maintenance, and design. Maintenance practices are important for sustained operation and have also been reviewed. Predicting maintenance is essential to assessing lifecycle costs. Within these research areas, gaps are explored, and recommendations made for future work.

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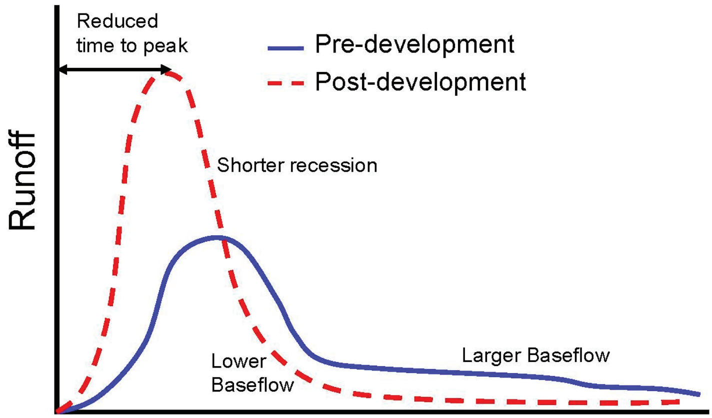
1. Introduction

The 20th century has witnessed the rapid transformation of rural lands to urban areas on a global scale. By 2050, it is projected that 64.1% developing and 85.9% of the developed world will be urbanized [1]. The growth in cities is caused mainly by rural migration to urban areas in the developing world and suburban development in the developed world [2]. Urban development causes a variety of impacts associated with serving the human population, including increased withdrawals of fresh water from surface and groundwater sources to meet demand, increased wastewater loading in separate and combined sewer areas, increased generation of solid wastes, and issues associated with human transportation [3]. The impervious surfaces created by buildings and pavement significantly alter the way water flows through and from watersheds, conveying additional pollutants with it [4]. Understanding and mitigating the consequences of urbanization on urban stormwater hydrology and quality is the key to addressing some of these issues.

1.1. Urban Stormwater Impacts

In urban areas, impervious surfaces include pavement and buildings, structures, and, in some cases, heavily compacted urban soils [5]. With the removal of vegetation and creation of hard surfaces, rainwater infiltration and natural groundwater recharge decrease. This results in increased runoff rates and volumes, reduced infiltration, groundwater recharge, and baseflow to urban streams [6,7]. The altered hydrology then causes environmental impacts [8], including downstream flooding [9], streambank erosion and stream downcutting [4,9,10]; declining water quality due to increases in sediment, nutrients, and heavy metals [11,12], and a decline in aquatic biota [13]. The hydrologic patterns before and after development are conceptually illustrated in Figure 1, adapted from [14].

Figure 1. Schematic illustration of the pertinent impacts of urbanization on hydrology at the catchment scale.



Note that post development runoff is greater in volume and peak with a lower baseflow, and reduced time to peak. A study of a 4047 m^2 (1-ac) paved parking lot found that it generates 16 times more runoff than a meadow of the same size [15]. In urban areas, various pollutants that accumulate on impervious surfaces during dry periods, are subsequently washed off during storm events and then discharged into receiving waters [16]. Changes in rainfall—runoff behavior and the generation of pollutants by urban land surfaces and activities result in the degradation of water quality and associated aquatic life in receiving waters. In general, this degradation is the result of two primary mechanisms (i) increased generation of pollutants, through changes in land use due to human activity [17], increased mobilization and transport as a result of increased surface runoff, and the hydraulic efficiency of the stormwater conveyance network [6]. Urban stormwater can contain numerous pollutants including suspended solids, nutrients, organic compounds, pathogenic bacteria, heavy metals, toxic pesticides or herbicides, trash, debris, and floatable materials [16]. Stormwater is highly variable [18], and with respect to nutrients like phosphorus, a portion is associated with hetero-disperse particulate matter [19]. Rainfall depth, catchment area, and the percentage of asphalt and natural surrounding land use have proven adequate predictors of nutrient concentrations and loads [20]. Other possible pollutants, such as heavy metals, pesticides, bacteria, hydrocarbons, and vehicle byproducts may also be conveyed by urban runoff from impervious surfaces to receiving waters, causing a wide variety of adverse (toxic, pathogenic, and sanitary) environmental issues in urban receiving waters [21]. For more information, the reader may consult data from the National Urban Runoff Program, or NURP [22] and the National Stormwater Quality Database, or NSQD [23] for further information.

1.2. Low Impact Development (LID)

Low impact development, or LID, is an ecological engineering practice that was introduced by Prince George's County, Maryland in the early 1990s as a means to holistically address the impacts of urban development. LID, also known as sustainable urban drainage, is a land planning and engineering design approach that implements small-scale hydrologic controls with integrated pollutant treatment to compensate for land development impacts on hydrology and water quality. The goal of LID is to maintain or replicate the predevelopment hydrologic regime using enhanced infiltration and evapotranspiration to reduce off-site runoff and ensure adequate groundwater recharge [24]. LID practices have multiple purposes, including: enhancing management of runoff, improving surface water quality, improving groundwater recharge, improving habitat, and enhancing the aesthetics of the community [25].

One of the most frequently used LID practices is bioretention. Despite its widespread use, research on bioretention systems is active, particularly in terms of mix design and treatment. The objective of this paper is to review recent research on bioretention systems, including field and mesocosm monitoring studies, the development of computational models, and the assessment of lifecycle costs. These areas are important for implementing the practice and improving the sustainability of urban drainage systems. Research gaps are identified and explored, and recommendations made for future work.

2. Bioretention and Its Applications

2.1. Definition and Function

A bioretention system is a landscaped depression that receives runoff from upgradient impervious surfaces, and consists of several layers of filter media, vegetation, an overflow weir, and an optional underdrain (see Figure 2 adapted from [26]). Bioretention cells are typically small and usually treat catchment areas less than 2 hectares [27]. Bioretention systems mimic the natural hydrologic cycle by retaining runoff to decrease flow rates and volumes [28]. Other benefits may also include an improvement in the aesthetics of the neighborhood, the enhancement of habitat for wildlife, a reduction in soil erosion, and the recharge of groundwater [29] and thus enhance base flows to local streams. Incoming runoff infiltrates through the media layers and is discharged through underdrain pipes. Internal water can also be lost through exfiltration and evapotranspiration. Exfiltration refers to a loss of water from a drainage system as the result of percolation or absorption into the in situ soil. Vegetation within the bioretention cell uptakes water and nutrients from the media. Overflow may occur if the media is saturated, and the small storage area then ponds until reaching a control elevation, upon which it begin to discharge. Bioretention normally consists of a layer comprised of media (sand/soil/organic mixture) for treating runoff, a surface mulch layer, various forms of vegetation, a storage pool of between 15 and 30 cm of depth and associated hydraulic control appurtenances for inlet, outlet, and overflow conveyance [30]. An underdrain is a preferable option when underlying soils are low in permeability [31] (<13 mm/h), effectively reducing the bioretention toa filter system [26]. Figure 2 demonstrates the profile of a typical bioretention facility with an underdrain. Runoff is filtered sequentially through each layer; however, the main filtration action is performed in the media layer [32]. Debris, particles, sediments, and other pollutants from runoff are filtered and treated before draining into a stormwater conveyance system or directly into receiving waters. The vegetated surface layer slows the runoff velocity and traps sediment [33]. Within a bioretention cell, treatment is performed by a variety of unit processes making use of the chemical, biological, and physical properties of plants, microbes, and soils to remove pollutants from urban runoff. Bioretention reduces peak flows [34], runoff volume [35], and pollutant loads [36,37]; increases evapotranspiration by vegetation uptake [38], and increases lag time [34]. An example of a field-scale bioretention cell is shown in Figure 3.

2.2. Media Specification and Amendment

Media is a key factor in bioretention design. Selection criteria are intended to improve runoff reduction and pollutant removal performance of bioretention and address local conditions. Examples of selected specifications from Virginia Department of Conservation; Maryland Department of the Environment; and Delaware Department of Natural Resources and Environmental Control and Recreation are compared for hydrologic management effectiveness, pollutant removal efficiency, construction and maintenance costs, and constructability. In general, a typical bioretention ideally contains approximately 50%–60% sand and 40%–50% mix of loam/sandy loam/loamy sand on a per volume basis. Clay content should be minimized to maintain proper cell hydrology, ideally in the range of 5%–8% [39]. A media with too much clay may reduce infiltration into the media. There are a wide variety of bioretention blends.

Figure 2. Schematic diagram of a bioretention facility and its hydraulic pattern.

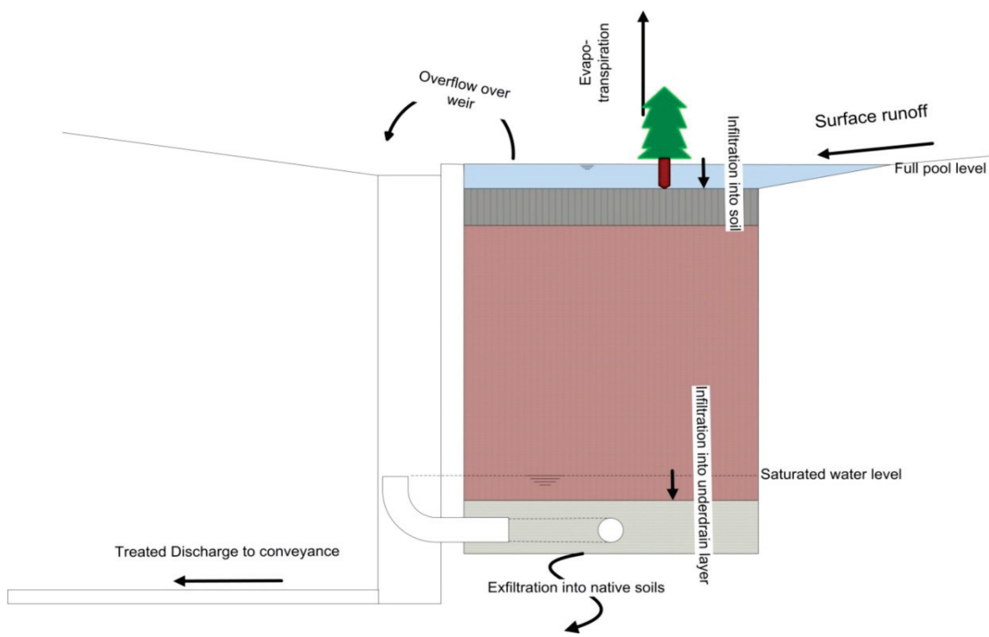


Figure 3. A bioretention facility at the Science Museum of Virginia in Richmond, Virginia.

Another key aspect of media specification is the P content. Soil P should be balanced between the growth needs of the plant for nutrients and to reduce the potential to leach nutrients in the long term. A media specification developed by the Virginia Tech Crop, Soils, and Environmental Sciences Department recommends that soil P the within the range of 5–15 mg/kg under the Mehlich I extraction procedure or 18–40 mg/kg Mehlich III extraction. There is a conversion table between these two methods [40]. According to Beck et el, keeping soil P within these ranges helps to minimize leaching [41].

The depth of the media layer is one of the primary design features controlling hydrologic performance of bioretention systems. A monitoring study was conducted that compared six bioretention cells in Maryland and North Carolina that differed by media depth, two were 1.2 m, and the rest were 0.5–0.6 m in depth. The larger media depths met their water quality volume capture target 80% the time; for the smaller, it was 44%, suggesting media depth may be the primary parameter influencing hydrologic performance [42]. A long-term observation from 2004 to 2006 of a bioretention cell in Charlotte, NC demonstrated that the peak outflow for 16 storms with less than 42 mm of rainfall was at least 96.5% less than the peak inflow, with the mean peak flow reduction being 99% [27]. From this study, it can be concluded that in an urban environment, bioretention can effectively reduce peak runoff from small to midsize storm events. This finding suggested that deeper media depths could improve hydrologic performance of bioretention systems. The depth of the layer should also consider construction cost and the local groundwater level. In general, a media layer of 0.7–1.0 m thickness is recommended in bioretention design [43]. It may be advantageous to use two media layers with the top designed to support vegetation and the bottom optimized for filtration. According to soil column studies by Hsieh and Davis, [44] a layer having a greater sand content optimized for pollutant removal media could be used below a media optimized for plant health to achieve increased pollutant removal.

2.3. Hydrologic Restoration

For this review, field monitoring studies that were published in peer-reviewed journals were evaluated. Design details, watershed characteristics and available hydrologic performance data for the reviewed studies are provided in Table 1. Only studies with an underdrain were included. One key feature of bioretention is its ability to mimic the pre-development hydroperiod of an undeveloped watershed and thus help to maintain a natural water cycle in urban areas. A study was conducted that compared underdrain flow from four bioretention cells in North Carolina within comparably sized, undeveloped watersheds draining to small streams, normalized by drainage area. The results indicated no statistical difference between flow rates from the undeveloped watersheds and bioretention outflow rates for the two days following the commencement of flow [45]. This study confirmed that bioretention outflow can mimic non-urban, shallow interflow to streams, and thus help restore the natural hydroperiod.

The use of bioretention facilities can also increase runoff time of concentration [34]. A typical time of concentration value would be in the range of 5–10 min for a parking lot 0.2–0.4 ha in size draining directly to a storm drain. In contrast, the placement of a bioretention facility in front of the drainage outlet will increase the time of concentration, or time for the runoff to discharge, from a quarter hour to several hours [34], depending on the flow rates through the treatment media. Up to 31% of runoff entering the bioretention cells was lost through these exfiltration, and up to 19% was lost to evapotranspiration [42].

Table 1. Summary of bioretention field studies, hydrologic performance.

Source	Location	Bioretention Characteristics						Watershed Characteristics		Hydrologic Performance		
		Description	Media Composition	Media Depth (cm)	Bioretention Surface Area (m^2)	Ponding Depth (cm)	IWS depth (cm)	Impervious %	Drainage Area Surface Area (ha)	Delay $T_{p(out)}/T_{p(in)}$	Peak Flow Reduction %	Runoff Volume Reduction %
Brown and Hunt, 2008 [46]	Rocky Mount, NC	Different vegetation	98% sand, 2% fines	90	140	**	30	76	0.22	**	**	90 [2]
					150	**	60	72	0.24	**	**	98 [2]
	Nashville, NC	Media depth change	87% sand, 13% fines	60	425	23		79	0.65	**	**	75 [3]
				90	300	**		94	0.43	**	**	50 [3]
Brown and Hunt, 2011 [47]	Rocky Mount, NC	Depth change of IWS	96% sand, 4% fines	110	146	16	88	76	0.22	**	**	89
							58			**	**	93
				96	142	13	72	72	0.25	**	**	98
							42			**	**	100
Brown and Hunt, 2012 [48]	Nashville, NC	Pre-Repair	87% sand, 13% fines	60	290	13		83	0.68	**	84 [2]	63
		Post			322	20				**	92 [2]	88
		Pre-Repair		90	206	15		97	0.43	**	92 [3]	65
		Post			226	27				**	95 [3]	89
Davis, 2008 [34]	College Park, MD	Lined	50% sand, 30% topsoil, 20% OM	90	28	**		100	0.24	7.2	44	52
				120		**	90			5.8	63	65
Debusk and Wynn, 2011 [49]	Blacksburg, VA		Washed sand with fines and leaf compost, 88% sand, 8% fines, 4% OM	180	35	10	150	96	0.17	**	99	97
Hunt *et al.*, 2006 [35]	Greensboro, NC	15 cm IWS	Organic Sand	120	10	**	75	**	0.2	**	**	**
		High P index	Organic Sand	120	10	**		**	0.2	**	**	78
	Chapel Hill, NC	Low P index	Quarried Sand	120	9	**		**	0.06	**	**	**
Hunt *et al.*, 2008 [27]	Charlotte, NC		Loamy Sand, 6% fines	120	229	**		>99	0.37	**	97	**

Table 1. *Cont.*

Source	Location	Bioretention Characteristics						Watershed Characteristics		Hydrologic Performance		
		Description	Media Composition	Media Depth (cm)	Bioretention Surface Area (m^2)	Ponding Depth (cm)	IWS depth (cm)	Impervious %	Drainage Area Surface Area (ha)	Delay $T_{p(out)}/T_{p(in)}$	Peak Flow Reduction %	Runoff Volume Reduction %
Li *et al.*, 2009 [42]	College Park, MD		Sandy Loam, 12% OM	50–80	156	10–34		**	0.26	22	14 [1]	60 [1]
	Silver Spring, MD		Sandy Clay Loam, 5.7% OM	90	90	30		**	0.45	200	2 [1]	10 [1]
	Greensboro, NC		Loamy Sand, 3% OM	120	250	23	60	**	0.5	200	0 [1]	0 [1]
				120	240	23		**	0.48	13	0 [1]	10 [1]
	Louisburg, NC		Sandy Loam, 5% OM	50–60	162	15		**	0.36	4	4 [1]	36 [1]
				50–60	99	15		**	0.22	3	10 [1]	60 [1]
Olszewski and Davis, 2013 [50]	Silver Spring, MD	-	54% sand 46% fines	50–80	102	**		**	0.37	**	83	79

Notes: ** Not available; [1] Expressed as fraction, *i.e.*, $Q_{p,out}/Q_{p,in}$, rather than a% reduction. Values are in %; [2] Measured when there was outflow; [3] Value at 50% exceedence probability.

To enhance the reduction of outflow volume and facilitate denitrification, a modified design was introduced, known as Internal Water Storage (IWS). IWS is an optional subsurface portion of the media to provide storage volume in the bioretention cell [51]. The IWS layer is often created by installing an elbow at the end of the drain so that an IWS zone was produced between the bottom of the cell and the top of elbow [46]. Introducing an IWS layer tends to increase runoff reduction. The effects on hydrology caused by IWS within bioretention cells were investigated in North Carolina. IWS cells experienced pronounced reductions on stormwater volume (99.6% and 100%), while conventional cell reduced 78% volume under the same hydrologic condition. A study of bioretention performance in North Carolina showed that among 63 events monitored, the bioretention cell with IWS had outflow on 18 occasions, while the bioretention with a conventional underdrain design had 40 [46]. The effect of IWS depth is currently being explored. Two bioretention cells were constructed with equal drainage conditions with 30 and 60 cm IWS zones. In 40 precipitation events of, the two cells generated outflow in 34 and 22 times, respectively. The deeper IWS resulted in more retainage of storm runoff and alleviated hydrologic impacts to the surrounding environment. Evapotranspiration and exfiltration play major roles in volume reduction in a bioretention cell and its IWS layer [46]. Including an IWS layer may assist in nitrate (NO_3) removal through denitrification process by providing an anoxic zone in the bottom media layer of bioretention [51]. Studies of pilot-scale bioretention with IWS layers had positive results of 80% NO_3 mass removal [52]. Passeport *et al.* conducted a field study comparing two grassed bioretention cells including IWS zones for 16 months. Significant load reductions were observed for NO_3 and nitrite (NO_2) that varied from 47% to 88% in the growing season [53].

A critical concern that negatively impacts bioretention functions is surface clogging caused by fine silts and sediments in urban runoff. Hydrologic performance of bioretention can significantly degrade if impacted by large quantities of sediment, leading to less-than-adequate water storage volume and surface infiltration rates [48]. A study on urban particle capture in bioretention media showed clay-sized components of incoming TSS clogged the media [54]. In a survey of 43 bioretention cells across North Carolina, Wardynski *et al.* [55] found that 65% of the cells were undersized. Despite 71% not meeting particle size distribution specifications, most were found to adequately drain and still meet hydrologic goals by treating the water quality storm.

A key feature of hydrologic restoration is exfiltration of water to surrounding soils. Eventually this water migrates to the groundwater table. This has raised some concerns in some regions and has resulted in suggested buffers from building foundations. In a modeling study in Syracuse, NY, Endreny and Collins [56] estimate a 1.1 m rise in the water table after bioretention implementation. The mass load reductions associated with the loss of water due to exfiltration may simply be transferred to groundwater, with a lag time for nutrients of 4–5 years [57].

2.4. Pollutant Treatment

For this review, field monitoring studies that were published in peer-reviewed journals were evaluated, and are listed with design details, watershed characteristics in Table 2. As with Table 1, only studies with an underdrain were included. Performance results on pollutant removal of bioretention systems from both laboratory and field studies suggest that bioretention practices have the potential to be one of the most effective BMPs in pollutant removal [30]. The water quality improvement by bioretention has been extensively observed and reported through field experiment or management. Note, all references to pollutant removals are referring to mass load reduction, unless specified otherwise.

2.4.1. Nitrogen

The treatment of N species includes ammonification, volatilization, nitrification, denitrification, and vegetative uptake. Ammonification is the process to breaking organic N chemicals into ammonium. Volatilization processes are mainly responsible for the loss of ammonia in bioretention systems. Ammonium ions can be transferred to ammonia gas with a pH above 7.5 or 8 [58], however, media are typically below these values. Nitrification is a microbial process by which reduced N compounds (primarily NH_4) are sequentially oxidized to NO_2 and NO_3 [35]. The process of nitrification, which is controlled by autotrophic microbes, is dependent on pH and dissolved oxygen content. Nitrification occurs in waterlogged soils in the thin aerobic zone created around the roots of plants [59], and in other aerobic zones. Denitrification is the process through which NO_3 is converted to gaseous N by microorganisms under anaerobic conditions. It is the only point in the N cycle at which fixed N reenters the atmosphere as N_2. The complete denitrification process can be expressed as a redox reaction [60]. NH_4 and NO_3 in soil are assimilated by plants through their root systems for physiological activities. The N uptake rate is influenced by plant growth rate, and concentrations of inorganic N forms [61]. Field sampling and analysis on 3 bioretention sites found that high annual NO_3 mass removal rates varied between 13% and 75% [35].

2.4.2. Phosphorus

Phosphorus can serve as a throttle to the productivity of most freshwater systems and can lead to eutrophication under high inputs [59]. The main treatment processes for P removal within bioretention are precipitation, adsorption, filtration, and vegetation uptake. Precipitation of P occurs when the critical concentration for nucleation of seed crystals is exceeded and two or more substances combine to form a solid phase [58]. Precipitation can be an important removal process in stormwater high in metal ion content. P ions can be adsorbed readily by many soils through the process of ion exchange or ligand exchange [62]. Adsorption is considered a necessary process to remove P within bioretention, and can be modeled using isotherm equations including linear, Freundlich, and Langmuir among others. In bioretention systems, particulate phosphorus (P) can be retained in soils through filtration, and become part of the soil-water system of bioretention [63]. The soluble PO_4 is the most readily available form of P species for vegetative uptake [64]. Factors that influence the rate of P uptake in plants include the proportion of plant roots that are exposed to P, plant and root age, as well as environmental conditions including temperature and soil pH [65]. Long-term PO_4 removal in a field-scale bioretention system was observed. The study found that the median PO_4 concentration decreased by 0.21–0.25 mg/L in the ponded water and down to 0.03 mg/L in the pore water at the bottom of the infiltration bed. The removal performance did not decrease during 9 years of monitoring [66].

Table 2. Summary of bioretention field studies, water quality performance.

Source	Location	Description	Media Composition	Bioretention Characteristics: Media Depth (cm)	Bioretention Surface Area (m^2)	Ponding Depth (cm)	IWS depth (cm)	Watershed Characteristics: Impervious %	Drainage Area Surface Area (ha)	Water Quality Performance: TSS %	TN %	TP %	Coliform %	Zn %	Cu %	Pb %
Brown and Hunt, 2008 [46]	Rocky Mount, NC		98% sand, 2% fines	90	146	**	30–60	76	0.22	92	80	72	**	**	**	**
Brown and Hunt, 2011 [47]	Rocky Mount, NC	Depth change of IWS	Sandy Clay Loam, 96% sand, 4% fines	110	146	16	88	72	0.22	58	58	−10	**	**	**	**
Brown and Hunt, 2012 [48]	Nashville, NC	Pre-Repair	87% sand, 13% fines	60	290	13		83	0.68	71	12	5.3	**	**	**	**
		Post			322	20				79	35	12	**	**	**	**
		Pre-Repair		90	206	15		97	0.43	84	13	44	**	**	**	**
		Post			226	27				89	32	19	**	**	**	**
Chen, 2013 [67]	Lenexa, KS		Sand with hardwood mulch and sandy loam planting soil, 20% OM	71	200	23		40	0.25	**	56	**	**	**	**	**
Debusk and Wynn, 2011 [49]	Blacksburg, VA		Washed sand with fines and leaf compost, 88% sand, 8% fines, 4% OM	180	35	10	150	96	0.17	99	99	99	**	**	**	**
Hathaway and Hunt, 2011 [68]	Wilmington, NC	Different soil depths	Baymead fine sand, 87%–88% sand, 12%–13% fines	60	55	28		100	0.1	100	**	**	70	**	**	**

Table 2. *Cont.*

Source	Location	Description	Media Composition	Bioretention Characteristics: Media Depth (cm)	Bioretention Surface Area (m^2)	Ponding Depth (cm)	IWS depth (cm)	Watershed Characteristics: Impervious %	Drainage Area Surface Area (ha)	Water Quality Performance: TSS %	TN %	TP %	Coliform %	Zn %	Cu %	Pb %
Hunt *et al.*, 2006 [35]	Greensboro, NC	IWS added	Organic Sand	120	10	**	75	**	0.2	**	68	**	**	>98	>98	>80
		High P index		**	10	**		**	0.2	−170	40	−240	**	**	**	**
	Chapel Hill, NC	Low P index	Sand	**	9	**		**	0.06	**	40	65	**	**	**	**
Hunt *et al.*, 2008 [27]	Charlotte, NC		Loamy Sand, 6% fines	120	229	**		98	0.37	60	32	31	71	60	77	32
Li and Davis, 2008 [54]	Washington DC		50% sand, 30% topsoil, 20% mulch	110	17	**		100	0.077	55–99	**	**	**	**	**	**
Li and Davis, 2009 [37]	College Park, MD		Sandy Loam, 80% sand, 20% fines, 6% OM	50–80	181	15		90	0.28	96	−3	−36	95	92	65	83
	Silver Spring, MD		Sandy Clay Loam, 54% sand, 46% fines, 12% OM	90	102	30		90	0.45	99	97	100	100	99	96	100
Passeport and Hunt, 2009 [53]	Graham, NC	North cell	Expanded slate, 15% sand, 80% fines, 5% OM	60	102	23	45	40	0.69	**	54	63	95	**	**	**
		South cell		90			75	**		**	54	58	85	**	**	**

Note: ** Not available.

2.4.3. Metals

Metals are of particular concern due to their ecotoxicity accumulation potential [69]. It has been observed that the surface layer of bioretention systems performs a significant role in retaining metals [70]. Field studies suggest that bioretention appears to be an efficient facility to remove heavy metallic elements from runoff. A bioretention cell in an urban setting in North Carolina was studied from 2004 to 2006. Water quality samples were collected for 23 events and analyzed for some typical heavy metals including Cu, Zn, and Pb. There were significant reductions in the concentrations of Cu, Zn, and Pb. Efficiency ratios for Cu, Zn, and Pb were 0.54, 0.77, and 0.31, respectively [35]. Another bioretention cell in the District of Columbia accumulated Zn, Pb, and Cu with total metal concentrations of 532, 660, and 75 mg/kg, respectively [70].

2.4.4. Solids

Total suspended solids (TSS) can be effectively removed through bioretention layers, typically through sedimentation in the basin and filtration in the media. A bioretention system in North Carolina under study with 23 rainfall events showed a removal ratio as 0.60 for TSS [35]. A Maryland field study of two cells has documented 54% and 59% mass removals of TSS [71]. Mature systems demonstrate enhanced filtration and sedimentation of TSS with improved TSS removal efficacy. Care must be taken to avoid the use of bioretention as a sediment trap. Despite their efficient sediment removal, clogging may occur.

2.4.5. Pathogens

Bacteria that can cause infection are known as pathogenic bacteria, and are a major water quality concern that can be treated by bioretention. A significant reduction of pathogenic bacteria was observed in an urban bioretention from 19 storms for fecal coliform and 14 events for *E. coli*. The efficiency ratios for fecal coliform and *E. coli* are 0.69 and 0.70 respectively [27].

These results of pollutant treatment indicate that in an urban environment bioretention systems can reduce concentrations of most target pollutants, including pathogenic bacteria indicator species. It also reduces mass loading because of runoff reduction through exfiltration to surrounding soils [34]. One study examined water quality improvements of numerous pollutant parameters including total arsenic, total cadmium, chloride, total chromium, total and dissolved copper, *Escherichia coli* (*E. coli*), fecal coliform, lead, mercury, N species, oil and grease, P species, total organic carbon, TSS, and Zn via monitoring for a 15-month period at 2 bioretention cells in Maryland. The monitoring results showed both bioretention cells effectively removed suspended solids, lead, and zinc from runoff and the effluent EMCs met local water quality criteria [37]. The variability in bioretention treatment performance may be influenced by the site's environment, including the climate, the groundwater, the surrounding watershed characteristics, and background pollutant levels. The following section describes current research on bioretention performance.

2.5. Temperature Reduction

Thermal impacts have been demonstrated to result in a decline in coldwater fisheries [72] of the salmonid family. Rainfall falling on hot pavement in the summer will increase in temperature significantly by the time it is discharged. Control practices differ in how they improve or exacerbate these thermal impacts. In a four-year study, Roseen *et al.* [73] evaluated thermal impacts from a retention pond and a gravel wetland, and found that the retention pond was more susceptible to thermal variability. The gravel wetland was found to have a greater capacity for thermal buffering of discharges. Bioretention has been found to also provide thermal buffering by both runoff reduction and attenuation [74]. Another study evaluated the size of bioretention and its thermal buffering capacity, and found smaller bioretention cells may be more effective at reducing thermal impacts.

2.6. Biological Diversity

Bioretention systems in urban Australia have shown to support greater diversity and species richness than both lawn and garden bed-type green spaces in the same area [75,76]. These studies found bioretention had a significant increase in plant and invertebrates taxa, both of which are used as indicators of aquatic ecosystem health. While microbial action and plant uptake play a role in the treatment processes involved in bioretention, little is known how these mechanisms can be augmented by system design. Variation among plant species has been shown to affect bioretention performance [77], which was one of the factors identified by Zhang *et al.* [78], in which more diverse plant species resulted in reductions in nutrient loading. Whether ecosystems facilitated by different plants and invertebrates foster pollutant removal in bioretention remains an open research area.

3. Current Research

3.1. Aspects of Bioretention Research

Major aspects of bioretention research have focused upon hydrologic mitigation and runoff treatment. A common means to investigate these features is through direct observation on field-scale bioretention facilities. Another method employed in research is to simulate a bioretention system within an artificial container, called a mesocosm. Mesocosms clarify the roles of media, plants, and microbes in this complicated and interrelated ecosystem. Computational models may extend the reach of our ability to simulate complex bioretention processes based upon physical laws and mathematical equations. Modeling simplifies the bioretention system, helps characterize its internal water flow, pollutant mass fluxes and hydrology, and assists in evaluating pollutant treatment performance. Since the mechanisms and maintenance practices of bioretention systems are still evolving, long-term performance and life-cycle cost [30] relationships are still being documented. As these relationships become better understood, simulations can better predict lifecycle costs and maintenance intervals. These areas of research are detailed in the following sections.

3.2. Column and Mesocosm Bioretention Studies

A mesocosm is an experimental tool for small-scale laboratory study of bioretention [79]. The merit of mesocosm studies is their simplification of a complex full scale system, and ability to separate individual factors for evaluation through replication. Mesocosm experiments can be used to determine optimal designs with specific combinations of media selection and layer setting. They can also circumvent some impediments in large scale implementation of bioretention practices, including uncertainties related to performance and cost, insufficient standards and technical regulations, institutional and legislative gaps, insufficient funding, and effective market incentives [79]. Although a mesocosm is an artificial system with limited space, and less realistic than field scale studies, they can be used as a tool to reveal the internal mechanisms and fluxes within bioretention cells.

Mesocosm experiments have been extensively conducted to evaluate bioretention performance and understand internal treatment processes [80–82]. An early example of this research is Hsieh and Davis [44], who performed two experiments with 18 bioretention mesocosms using synthetic runoff. The experiment compared pollutant removals between two designs to show that a uniform profile was a more cost-effective alternative than multilayer media. Another mesocosm experiment was conducted [83] to evaluate bioretention media characteristics. Results showed media with excess clay could clog and increase TSS discharge.

Amendment of media to improve bioretention performance is an active area of research. Water treatment residuals (WTRs), containing alum, are used as an admixture within bioretention media to enhance P removal. A specific media using WTR as an admixture can provide effective initial total P retention >94% [81]. Other research on WTR P removal demonstrated that Al oxides in WTR could adsorb P, and increasing WTR content in the media resulted in greater P adsorption [81,84,85]. Another mesocosm study [86] examined the capability of a bioretention soil mixtures with 60% sand, 15% compost derived from yard, garden, wood, and food wastes, 15% shredded cedar bark, and 10% water treatment residuals containing alum to reduce nutrients from storm runoff. Results showed that a saturation zone could reduce NO_3 significantly in the effluent (71%), however PO_4 removal was higher without it (80% compared to 67% with IWS). Vegetation did not make a difference in this study. A higher P removal of >94% removal was achieved using a specific media with WTR as an admixture with coir peat to reduce nutrient leaching losses [81]. The presence of vegetation was a significantly correlated with improved P retention [81,82,87]. Carbon-enriched media was hypothesized to enhance N removal, with carbon serving as an electron donor to facilitate the denitrification process. Modifying bioretention media with newspaper and wood chips provided N removal above 90% [52].

Some mesocosm research has shown that nutrient removal from stormwater can be enhanced by promoting plant growth and microbial activity. Retention and removal of nutrients in vegetated and unvegetated bioretention mesocosms were investigated with 30 well-established 240-L "wheelie-bin" containers to evaluate the effects of plants [82]. The experiment demonstrated that the vegetated sandy loam mesocosms retained higher amounts of nutrients, suggesting that this combination of media type and vegetation may promote pollutant removal in bioretention cells.

The improvement in N removal indicates that denitrification is being facilitated by plants and associated microbes in the root zone.

Mesocosm experiments can also help determine hydraulic retention time (HRT) for optimizing treatment. Lucas and Greenway conducted a series of bioretention mesocosm experiments with planted vegetation to compare hydraulic response and N retention with free discharge and regulated outlets to increase the HRT by up to 8 times. At a hydraulic loading rate (HLR) of 60 cm over 90 minutes, the regulated outlet retained 68% of $NO_{2\text{-}3}$ and 60% of total N; while the corresponding free-draining treatment retained 25% of $NO_{2\text{-}3}$ and 27% of total N [80]. At half this HLR, TN removal was as high as 78%, and $NO_{2\text{-}3}$ removal was over 90% [80]. Outlet control and lower HLRs provided longer HRTs and thus improves N removal. However, runoff capture is compromised with longer HRTs [80,82]. This result indicates that HRT should be a significant point of consideration in design for nutrient and metal removal, especially for those pollutants that require redox or biological conversion.

3.3. Field-Scale Bioretention Monitoring

Studies of a field-scale bioretention cells have been conducted to provide design factors that are important to meet hydrologic and water quality goals [35,42,48,66,88,89]. To evaluate the hydrologic impacts of bioretention within an urban environment, Davis [34] monitored the performance of two bioretention cells receiving runoff from adjacent parking lots for approximately two years, covering 49 rainfall events. Results indicated that discharge flow peaks were reduced by over 50%, and were lagged in time by a factor of 2 or more. Another study on six cells in Maryland and North Carolina showed that bioretention could achieve substantial hydrologic benefits by delaying and reducing peak flows and decreasing runoff volume. Performance diminished as rainfall depths increased and rainfall durations became longer. The authors found a large cell media volume to drainage area ratio and drainage configurations were the most dominant factors that improved performance. Annual water budget analysis suggested that approximately 19% of runoff entering the bioretention cells was lost to evapotranspiration, and 8% was loss to exfiltration [38]. The sites in Louisburg, North Carolina monitored the infiltration rate, and it was found to be in the range of 2.5–3.8 cm/h.

Li and Davis [37] evaluated water quality improvements of two bioretention cells for a 15-month period in Maryland. The authors found that bioretention performed effectively in removing TSS, Pb, and Zn from runoff. They found runoff volume reduction promoted pollutant mass removal and linked outflow quality benefits with hydrologic performance. Lloyd and Wong [90] found that a landscaped bioretention reduced suspended sediments by 68% and total P and N by 60% and 57%, respectively. In some cases, effluent from bioretention areas might have higher pollutant concentrations than those of the influent. A monitoring experiment in North Carolina indicated mean pollutant reduction efficiencies for the bioretention cells of 79% reduction for TSS with an increase in NO_3 and NO_2, resulting from a combination of N additions within the cell and conversion [91]. This is consistent with other observations as bioretention typically reduces TSS, oil and grease, heavy metals and P, but have been less effective for N [44]. Yang *et al.* [92] evaluated a biphasic bioretention cell; which uses sequencing batch reactor processes including alternating aerobic and anaerobic sounds in a longer HRT to facilitate denitrification. Approximately 91% of introduced NO_3 was removed.

3.4. Development of Computational Models

Numerical modeling of bioretention systems is an area of active research. Computational models can provide assistance in characterizing the multiple physiochemical and biological processes occurring within a bioretention cell. Coupling these processes with the hydrology of a site can provide a means of predicting treatment performance of a given design. Before models can be used to predict behavior, however, they must reliably replicate observed data and/or be calibrated. Most studies focus on model calibration and performance of the model.

A variety of models have been applied to LID practices, including bioretention, and a selection of these is listed in Table 3. A review of these and other similar models can be found in [93,94].The US EPA's SWMM, or Storm Water Management Model [95,96] uses a rainfall–runoff model to estimate runoff volumes, peak flows, and with continuous simulation, flow duration. Bosley modeled multiple bioretention cells within a watershed using SWMM to evaluate their hydrologic performance [97]. Bioretention is one of the LID options within SWMM. As currently configured, a bioretention cell must be contained within a sub catchment, effectively limiting its use to upper portions of a watershed [98], which is the norm. Most of the components of a typical cell can be input by the user, including underdrains. Water quality treatment is limited to mass load reduction. A computational model of a bioinfiltration cell (similar to a bioretention cell, no underdrain) [99] in a traffic island was developed using the Hydrologic Modeling System, or HEC-HMS [100]. The authors were able to separately simulate many key hydrologic elements, such as infiltration using the Green-Ampt infiltration submodel. However, key control and routing elements needed for design were beyond the capability of the model, which is limited primarily to simulating storage, *i.e.*, detention. A new feature of the model was added in version 4.0, in which the nutrients N and P concentrations are simulated, incorporating overland flow and within stream processes.

Lucas conducted a design of integrated bioretention urban retrofits with storm event simulations by HydroCAD [101]. The author found that, excluding reverse flows, HydroCAD simulated the hydraulics of the cell in a manner virtually identical to SWMM, however the latter model can provide continuous simulation [31], whereas the former cannot. DRAINMOD [102], originally developed for the purpose of simulating agricultural fields, was recently adapted to simulate bioretention [103], due to the similarity between an underdrain and subsurface drain tiles. The authors found that the model was able to simulate IWS and replicate soil water characteristic curves, a unique capability. Model validation was performed on two bioretention cells in Rocky Mount and Nashville, North Carolina. It became evident after beginning the study that there were problems in measuring soil moisture at Rocky Mount because drainage removed water too quickly. Issues also arose at Nashville in terms of overflows. It was found that the model performed reasonably well after adjusting for a design modification that added surface storage at Nashville and an IWS at Rocky Mount. While these applications do not include water quality simulation, additional modules of DRAINMOD are available to simulate the N cycle [104,105]. P currently is not simulated.

Table 3. Descriptions and uses of major computational models to simulate bioretention systems.

Model	Brief Description	Capabilities	References
SWMM	Hydrologic, hydraulic and water quality model with optional continuous simulation	Detailed analysis of watershed with storage-focused LID	Documentation: [95,96] Applications: [31,109–112] Download: [113]
Hydro-CAD *	Hydrologic model that uses a design storm methodology to calculate runoff and detention pond routing with exfiltration option	Analysis of storage and infiltration based LID within a watershed	Documentation: [101] Applications: [31,114] Download: [101]
HEC-HMS	Model to develop standard hydrograph based on precipitation input	Obtaining standard, non-adjusted hydrographs. Not recommended for modeling integrated practices	Documentation: [115] Applications: [99,116,117] Download: [100]
RECARGA	Hydraulic model for optional event and continuous simulation or design purpose	Detailed analysis for bioretention hydraulics and runoff retention	Documentation: [118–120] Applications: [119,121,122,] Download: [120]
DRAINMOD	Hydrologic model based upon agricultural field drainage, and treatment, a similar process to bioretention	Simulates water table and soil-moisture profile.	Documentation: [102] Applications: [103–105] Download: [123]
WinSLAMM **	Hydrologic model that uses a derived distribution based upon small storm hydrology to simulate performance of controls	Pollutant washoff calculated based upon land characteristics. Model traces pollutants from sources and predicts effects of controls	Documentation: [124] Applications: [106,110,125] Download: [126]
IDEAL *	Hydrologic model that uses a derived distribution to simulate performance of controls, for both quality and quantity	Process-based pollutant loading and treatment model, includes decay, settling, and infiltration, focused upon evaluation of a site before and after development.	Documentation: [127] Applications: [107] Download: [128]
WWHM	Hydrologic model based upon HSPF adapted for control practice design using continuous simulation	Calibrated regional parameters for the 19 counties of Western Washington, Version 2012 includes modeling elements to more accurately model bioretention and other LID practices.	Documentation: [129] Applications: [108] Download:[130]

Notes: * Proprietary; and ** Licensed, the remaining are public domain.

Two derived distribution models have been applied to LID simulation, WinSLAMM, and IDEAL. WinSLAMM uses a small storm methodology for hydrology [106], coupled with characterization of land use to develop predictions of water quality input to a control. Pollutant treatment through a variety of processes is also simulated within the control such as bioretention. WinSLAMM is usually applied at a watershed scale, in contrast with IDEAL, which is usually applied at the site scale. IDEAL also provides process-based estimates of pollutant removal for each control, including a very detailed sediment submodel [107]. The Western Washington Hydrologic Model, or WWHM is an adaptation of HSPF, or Hydrological Simulation Program—Fortran. Like its parent model, WWHM uses continuous simulation of most hydraulic processes to model LID [108], and must be calibrated to specific watersheds.

Another model, RECARGA [120] was specifically developed to simulate an individual bioretention cell to assist in design. RECARGA uses a physically-based approach to simulate the water balance for runoff inputs, surface ponding, infiltration, evapotranspiration, overflows, underdrain outflows, and exfiltration or groundwater recharge [118]. RECARGA was applied to the Sugar River watershed in Verona, WI to develop site-specific hydrologic criteria [131]. These RECARGA simulations illustrate trade-offs in design; *i.e.*, maintaining a predevelopment recharge rate while minimizing increases in runoff. RECARGA replicated site hydrology well. It does not, however, simulate constituents nor estimate water quality treatment at present.

Roy-Poirier *et al.* developed a numerical model to calculate unit processes of bioretention that reduce P in both soluble and particulate forms [63]. The authors presented simplified reaction equations to describe the processes of precipitation, dilution, vegetative uptake, isotherm sorption, and settlement. This model does not consider vegetation uptake and defoliation and thus cannot complete the full cycle of P transformation within a bioretention system. A sophisticated model of nutrient flux was developed by Kadlec and Hammer that describes the dynamic changes of P, N, and carbon within wetlands [132]. These processes included mineralization, plant uptake, nitrification, denitrification, and volatilization using coupled differential equations [133]. Event-based simulations are typically used to define limits of nutrient retention under standard conditions for regulatory compliance, and can be informative in comparing performance of design alternatives. A review of similar models for nutrient simulation is provided by Langergraber *et al.* [134]. Most of these models incorporate vegetation, but assume biomass content remains constant, *i.e.*, no growth, and no seasonal defoliation.

A computational model of bioretention can be a useful tool to provide a means to estimate output metrics such as runoff peak, runoff volume and nutrient removal for the purpose of guiding design and enhancing performance. In effect, this allows the user to try multiple designs. Models simplify the complicated processes within bioretention using mathematical constructs and equations. The initial models of bioretention systems suffered from lack of data and inappropriate assumptions. Improving computational programs for bioretention modeling is an ongoing research need.

3.4. Maintenance, Costs, and Life Cycle Analysis

3.4.1. Maintenance

Recent studies have focused upon the management and maintenance of bioretention in order to enhance performance and reduce lifecycle costs. In a recent study, 2 sets of bioretention cells were repaired by excavating the top 75 mm of fill media to remove accumulated fine sediments. This increased the surface storage volume by nearly 90% and the infiltration rate by up to a factor of 10. Overflow volume decreased from 35%–37% to 11%–12% respectively. Nearly all effluent pollutant loads exiting the post-repair cells were lower than their pre-repair conditions [48]. This outcome showed that clogging was limited to the surficial media layer, and maintenance was critical to performance. In another study, 43 bioretention cells were evaluated across North Carolina to assess if they were constructed in compliance with their design [55]. In addition to discrepancies between their design and practice, media specification also changed in 2005. Despite more than 65% of the cells being undersized, most were meeting their infiltration drawdown goal after a storm event. In addition to the visual drawdown inspection, infiltrometer tests can be performed, allowing calculation of the saturated hydraulic conductivity across the bioretention cell [135]. In a study of two bioretention cells receiving bridge deck runoff in North Carolina [136], the units were sized for the 25 (standard design) and 8 mm (undersized) rain events and had similar depth, and water storage characteristics. Despite its size, outflow pollutant loads between the two cells were not significantly different. Because smaller systems are likely less expensive, this suggests that undersized systems may perform better in terms of cost per unit of drainage area.

3.4.2. Costs

Costs of bioretention have been found to be highly variable, and depend greatly upon design objectives and the characteristics of a given site [30]. The U.S. EPA model SUSTAIN [137] (System for Urban Stormwater Treatment and Analysis Integration) contains links to various cost databases that assist in general and specific cost estimates of bioretention [138]. Generalized relationships have been developed for construction and operation and maintenance costs in North Carolina [139], using regression analysis to develop a power equation between costs and drainage area. An alternative approach is a spreadsheet cost model developed by the Water Environment Research Foundation (WERF). Because of the relative newness and uniqueness of the different bioretention designs, the WERF cost model [140] had few experiential examples to base its calculations upon; instead estimates are developed based upon unit costs from national sources such as RS Means [141]. Since maintenance requirements for bioretention practices are still being established [30], costs will then very substantially based upon what activities are conducted. In a recent study, Houle *et al.* [142] provided insight into maintenance activities by tracking costs and labor demands for bioretention practices over a period of 2–4 years. The authors found that despite conventional wisdom, LID practices such as bioretention, which typically require proactive rather than reactive maintenance, experience lower marginal costs than conventional practices. In addition to maintenance, an often overlooked but substantial component of costs is the opportunity costs of the space or land occupied by the bioretention practice [143,144]. Roy *et al.* [145] pointed out that performance enhancements from bioretention are very difficult to measure unless implementation is targeted on a small watershed scale. Because of limited resources, implementation is usually spread out and not focused a single watershed where impacts can be focused and measured. Roy *et al.* contended that costs and performance are inseparable and future research should target both of these metrics through implementation at a watershed scale where improvements can be measured and assessed.

3.4.3. Lifecycle Analysis

Flynn and Traver [146] trace the life cycle of a bio-infiltration cell and assess its performance using metrics such as carbon footprint, acidification potential, human life cycle economic costs and *etc.* to evaluate its benefits and impacts. Results showed that the construction phase is the main contributing life cycle phase for all adverse environmental impacts, as well as total life cycle cost and labor impacts [146]. The assessment provided guidance towards refined design and possible sustainable management of bioretention practices. Taylor and Fletcher [147] describe a new costing module that is part of MUSIC (Model for Urban Stormwater Improvement Conceptualization). A key benefit of using a module such as this is the potential collection of additional data sets to improve the accuracy of cost estimates.

3.5. Implications for Design

Design practices, including that of the media blend and hydraulics of bioretention cells are evolving. Due to the propensity of bioretention to collect sediment and potentially clog, pretreatment removal of sediment prior to treatment in a bioretention cell is essential. Current practice in media design is to use blends heavy in sand content, to eliminate clogging and provide rapid infiltration into the bioretention cell, an example of one of these is the Commonwealth of Virginia recommendation [148]. Plants should be selected carefully considering the anticipated cycles of wet and dry soil moisture conditions, with sandy mixes typically resulting in drier conditions. Multiple layers may be with dual purposes [44], may be an option. Underdrainage is usually needed for soils with slow infiltration rates, *i.e.*, less than 13 mm/h. A range of compost materials have been used in media blends, as organic carbon can provide an electron source and facilitate denitrification. However, recent Washington State monitoring data [149] indicates that compost with sources other than yard waste may contain loosely bound heavy metals and nutrients which may result in an increase in these compounds in discharges, at least initially. Admixtures such as water treatment residuals (WTRs) containing alum have been demonstrated to increase P removal. However, performance varies substantially depending upon the specific blend of WTRs. Inclusion of a means of retaining water for longer periods, such as an IWS may increase N removal. Models may be able to facilitate hydraulic design of bioretention, future models should be able to assist in customized treatment processes.

5. Summary and Research Needs

Bioretention is one of the most recognized LID practices for mitigating the hydrologic impacts of urbanization development and improving water quality in urban areas. Extensive research work has been conducted on bioretention to understand its function, improve its performance, and lengthen and predict its lifecycle. After compiling this review, the authors make the following recommendations for further research:

- Direct monitoring experiments of field-scale bioretention provides a means to evaluate hydrologic and treatment performance. Much work has been conducted in terms of field-scale bioretention monitoring. Several interesting studies have been conducted on undersized systems. A continuing study of the operation of undersized systems (currently underway through the Washington State TAPE program) until a substantial decline in performance can be observed may provide insight into the life cycle of bioretention. . This would require continued collection of performance monitoring, maintenance activities, and costs. Sufficient numbers of these studies need to be performed in various locations so the observations can be generalized. Groundwater data should be collected, where appropriate, at any field study location. This is to address potential mounding issues and to evaluate eventual fate and transport. Evaluating the thermal impacts of stormwater, and the benefits of bioretention remains a research need. Evaluating the biodiversity of existing bioretention systems, comparing them with forested ecosystems, and assessing that the effect on performance is also a research need.
- Mesocosms may provide a cost-effective alternative to field scale studies, and are similar in cost to column studies. They are less realistic than field scale studies. However, because of the ease of replication, use of mesocosms enable studies to focus on optimization of differing media blends and other factors such as HRT. Research is needed to better optimize mix design and provide better guidance to designers. Media amendments such as WTRs should be further evaluated. To maximize the utility of both field studies and mesocosm studies, results of both should be compared to assess whether generalizations can be made.
- Construction costs for bioretention vary widely. Part of this is due to the novelty of bioretention to the design community, but often there are unique factors at each site that influence costs. While municipalities are the main implementers of bioretention, there is presently little incentive to collect cost data or share it. While there are a few studies on maintenance activities and associated costs with bioretention, much more needs to be done over long durations. Research is needed to identify cost drivers, account for variability, and develop better tools for predicting costs. These activities will lead to a better understanding of lifecycle costs for bioretention.

- While there are a wide variety of computational models available for bioretention, there are still shortcomings of each. There is a need for a model that can estimate the hydrologic performance and nutrient removal of bioretention for design. Computation of biomass change, plant uptake, and defoliation are important processes which should be included to complete N and P cycles within bioretention systems, and complete the lifecycle of the practice. Computational models may provide a means to identify what is being transferred to groundwater.

Bioretention systems are small but highly complex. The physical and biological processes that occur within bioretention mimic ecological processes similar to those that occur in nature. These systems are perhaps the best effort so far at providing hydrologic ecological restoration of urban areas. To the extent that these systems can be installed cost-effectively and operated reliably for water quality treatment of runoff, they may represent a truly sustainable treatment practice. Improved estimates of performance will help meet downstream water quality goals. Continued research should lead to refinement of bioretention design and improved performance and help provide sustainable solutions to our urban drainage problems.

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Conflicts of Interest

The authors declare no conflicts of interest.

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A Review of Sustainable Urban Drainage Systems Considering the Climate Change and Urbanization Impacts

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Abstract: Climate change and urbanization are converging to challenge city drainage infrastructure due to their adverse impacts on precipitation extremes and the environment of urban areas. Sustainable drainage systems have gained growing public interest in recent years, as a result of its positive effects on water quality and quantity issues and additional recreational amenities perceived in the urban landscape. This paper reviews recent progress in sustainable drainage development based on literature across different disciplinary fields. After presenting the key elements and criteria of sustainable drainage design, various devices and examples of sustainable drainage systems are introduced. The state-of-the-art model approaches and decision-aid tools for assessing the sustainable alternatives are discussed and compared. The paper further explores some limitations and difficulties in the application of the innovative solutions and suggests an integrated and trans-disciplinary approach for sustainable drainage design.

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1. Introduction

For a long time, urban drainage systems have existed as a vital city infrastructure to collect and convey stormwater and wastewater away from urban areas [1,2]. Despite development over the years, it remains a significant challenge to design an effective functioning drainage system. In particular, impacts due to climate change and urbanization have been widely acknowledged, which could entail a substantial increase in the frequency and magnitude of urban flooding in many regions of the world [3–6]. At the same time, water quality problems also emerge as a result of urbanization that increases the variety and amount of pollutants and nutrients in receiving water bodies [7–9].

The conventional drainage system is mainly a single-objective oriented design with its focus on water quantity control. Today's drainage solutions also highlight the need to embrace more deliberately the other important aspects in urban water management, such as runoff quality, visual amenity, recreational value, ecological protection and multiple water uses [1,10–13]. Water quality has become increasingly significant in the design of urban drainage as a result of a wider political recognition of sustainability. A good example is the EU Water Framework Directive (WFD) that sets targets for good ecological status of all watercourses. This illustrates the current problem of aquatic environment protection and the urgent demand of developing strategies to cope with pollutants to receiving water bodies [14,15]. Besides environmental concerns, there also has been increasing criticism on the limited capacity and flexibility of conventional sewer systems to adapt to future climatic variability and urbanization [16,17].

On the other hand, since the Brundtland Report, the Rio declaration and Agenda 21, sustainable drainage systems have been highly promoted as an alternative and/or complement to the traditional approach to address long-term sustainability in the design of the system [2,11,18]. There is a growing trend towards managing water in a more sustainable way by activating its natural behaviors and process in the urban environment [19–21]. Unlike conventional drainage focusing on the "end-of-pipe" or "at the point of the problem" solutions [22], with small and decentralized techniques, sustainable drainage systems can largely alleviate the adverse impacts of non-point source pollution to urban water bodies [10,23,24]. Such solutions rely on local treatment, retention, re-use, infiltration and conveyance of water runoff in urban areas and thus are in better agreement with sustainable principles [19,25]. At the same time, there is rising acknowledgement of the potential of such systems with respect to their positive effects on urban landscape [11,26]. It is suggested to treat water as a positive source in sustainable drainage design to create new recreational sites in the urban landscape [12,13]. By doing so, the urban water is no longer hidden from the public, but used as an asset to increase user satisfaction and perceived values [5,27].

Sustainable drainage design is a multi-disciplinary research field that requires knowledge from specialists with different backgrounds; this paper therefore aims to give an overview of the status and emerging studies of sustainable drainage for researchers that are interested in participating in its development.

1.1. Drivers

1.1.1. Impacts of Climate Change and Urbanization

Climate change has been widely acknowledged as a global issue due to its anticipated impacts on urban water systems in terms of changes in water runoff and urban flooding [28–31]. Many scholars have reported in their studies that the expected increase in design intensities due to climate change can reach 20%–80%, depending on the region [32–34]. This has posed a huge challenge to the current drainage system that was designed based on a certain return period. The system is therefore faced with severe capacity problems in coping with the increasing amount of water due to climate change impacts. More importantly, future drainage design needs to take the increased frequency and intensity of precipitation into account in order to maintain an acceptable frequency of system overloading [35,36].

Urbanization represents another essential factor influencing the quantity and quality of urban water in cities. The process of city development can not only cause a significant change in runoff patterns in terms of both peak flow volumes and speed of runoff due to its impacts on impervious surfaces [3,4], but also vulnerabilities to flood hazards due to the change in urban intensity and distribution [5,37,38]. Meanwhile, land cover modifications generally associated with the economic explosion, such as removal of vegetative surface, replacement of raw land with impervious pavements, clearance and filling of natural ponds and streams, could induce increased amount of pollutants and harm the quality of urban water systems [7,39–42].

1.1.2. Challenges to Conventional Drainage Systems

Conventional drainage systems are designed to collect and transport water runoff from urban areas as quickly as possible via sewer networks and water treatment facilities to nearby receiving water bodies [1,11]. The main goal is to manage water volume in order to avoid urban flooding in city areas. The water is treated as a nuisance in the landscape and thus transported in a manner of "out of sight and out of mind" [43,44]. That is to say in the design of conventional drainage system there is limited concern for water quality issues and even less for its amenity and recreational values.

Many researchers have raised their concerns for the long-term sustainability of traditional drainage solutions by exploring their negative impacts on urban environments [25,45,46]. Stewart and Hytiris [47] talked about the pollution to receiving watercourses through combined sewer overflows (CSOs); the strong environmental interference of conventional drainage has also been criticized [48,49]. More notably, the traditional system is comprised of a large number of structural measures, such as concrete pipes and underground basins. The costs and time needed for restoration and installation of drainage network are tremendous [37]. This means the conventional system in many cases has to be expanded by bits and pieces and therefore lacks sufficient flexibility to adapt to critical circumstances [16,50]. In facing climate change and urbanization, expanding the conventional underground pipe system may not meet the general criteria of sustainability [43,48].

2. Terms and Cases of Sustainable Drainage Systems

The techniques of sustainable drainage systems are widely recommended and applied in many parts of the world, whereas the terminology varies in different regions, but with similar design philosophies. In Europe, Sustainable Urban Drainage System (SUDS) is used with its main focus on maintaining good public health, protecting valuable water resources from pollution and preserving biological diversity and natural resources for future needs [29,48,51]. In Australia, the term Water Sensitive Urban Design (WSUD) was proposed as a catchment-wide approach of which SUDS is a part and mainly refers to a planning and engineering approach to sustainably integrate urban water management into city landscape to minimize environmental degradation and achieve harmony between water and the urban environment [25,52,53]. SUDS is known as Low-Impact Development (LID) in the United States and Canada, which describes an approach promoting the interaction of natural processes with the urban environment to preserve and recreate ecosystems for water management [54]. LID puts the emphasis on conserving and using natural features in combination with small-scale hydrological controls to mitigate adverse impacts of urbanization [42,55,56]. Examples of similar approaches are Best Management Practices (BMP) in the United States and LIUDD (low impact urban design and development) in New Zealand.

As a result of the promotion of sustainability, several major research projects have been initiated worldwide. In Denmark, large national research programs include the "Water in urban areas" project working on transformation of the city water infrastructure to climatically robust systems [57], and the 2BG "Black, Blue & Green" project committed to integrated infrastructure planning for sustainable urban water systems [58]. The working papers from 2BG further expound their main goals and include case studies on sustainable urban drainage design implemented in Denmark and the Netherlands [58]. In the United Kingdom, the Construction Industry Research and Information Association (CIRIA) promotes sustainable drainage systems and also published a series of documents on design practices and applied projects [19]. In Ireland, Dublin's strategic drainage study involves several local authorities to perform an in-depth drainage assessment of integrated constructed wetlands [59]. In Sweden, a large six-year research project entitled "Sustainable Urban Water Management" was initiated by the Swedish Foundation for Strategic Research Programme with its focus on protecting valuable water resources in urban areas [48,60]. In Australia, one of the largest research activities on sustainable drainage solutions is the Cooperative Research Centre (CRC) for Water Sensitive Cities, which brings together over 70 inter-disciplinary partners to deliver sustainable water strategies facilitating transformation of the city into a more livable and resilient environment [61–63].

3. Previous Reviews

Wilderer [37] discussed how to apply the concepts of sustainable water management in rural and urban areas via diverse means. The paper addressed the necessity of taking into account multiple aspects (e.g., engineering, economical, administrative and cultural) in research to allow efficient application. Ashley, Garvin, Pasche, Vassilopoulos and Zevenbergen [19] present an overview of the prevalent SUDS components nowadays and showed the potential of integrating SUDS with traditional conveyance systems to satisfy both quality and quantity needs of flood management. Charlesworth [20] showed a review with more specific focus on vegetated and hard-engineering SUDS devices applied for climate change adaptation and mitigation in multi-site case studies. The paper emphasized the need of developing retrofiring technologies to existing buildings and built-up areas in SUDS design. From a more technical point of view, Elliott and Trowsdale [42] assessed 10 models with regard to their capability and relevance to sustainable drainage systems. The paper provides insights into the pros and cons of the reviewed models in response to different requirements of the various SUDS devices. From a management and governing perspective, Brown and Farrelly [64] explored transition bundles from conventional drainage approaches to sustainable solutions and revealed that the barriers are largely socio-institutional rather than technical. Butler and Parkinson [51] addressed new elements of sustainable drainage design and strategies to facilitate the transition from current practices to the new paradigm. To facilitate decisions on SUDS, Lai *et al.* reviewed a multi-criteria decision aid for integrated sustainability assessment, where three other popular decision-making support tools were also analyzed and compared [65]. All of these previous reviews provide valuable background on the concepts, features, objectives, techniques and tools for sustainable drainage design, with a specific focus on one of the components.

4. Sustainable Perspectives and Criteria

Over time, urban drainage has played different roles in cities. Earlier objectives of urban drainage include provision of a convenient cleaning mechanism of wastes for public hygiene and an efficient conveyance facility for flood protection. In recent years, additional focus has been on environmental protection and the recreational benefits of urban drainage [43]. Despite the various objectives and criteria of drainage systems' indifferent time periods, nowadays there is general agreement that sustainable drainage should integrate water quantity, water quality, and biodiversity and amenity aspects into design, namely the SUDS triangle [11,20,66]. In addition, several researchers called forth a renewed focus on public health and hygiene in SUDS design [48,67,68]. On the basis of these fundamental elements, Ellis *et al.* [69], Berke [70] and Makropoulos *et al.* [71] further explored four primary potential sustainability criteria: technical, environmental, social and economic factors. Each criterion contains sub-indicators enabling an assessment of drainage systems with regard to its economic evaluation, functional performance, resource utilization, environmental impacts, *etc.*

The sustainable criteria for urban drainage has become a great challenge, as this new paradigm needs to employ various disciplines of engineering and sciences to take into account all parts of the urban water cycle in management to ensure economic, social, ecological and environmental sustainability [63,72]. This requires a rather complex approach beyond the traditional one with its narrow focus mainly on the physical performance of the system [73,74]. Many researchers have promoted an integrated trans-disciplinary approach in an attempt to embrace and accommodate different key criteria for future drainage systems [46,72,75,76]. In this way, the design of urban drainage will no longer be formulated only based on single technical standards; more attentions will be paid to solutions with benefits for flooding management, spatial design and nature conservation. One good example is the "three-point" approach introduced by e.g., Geldof and Kluck [77] and Fratini, Geldof, Kluck and Mikkelsen [27]. This approach seeks to tackle the conflicts between the three typical design domains (daily amenity, technical optimization and extreme climatic conditions, respectively) of urban drainage and results in an integrated regime where different groups of values and professionals collaborate in the drainage design.

5. Techniques

SUDS are a range of drainage techniques and devices allowing for runoff attenuation and mitigation, pollutants reduction and amenity construction [19]. The most popular SUDS techniques applied nowadays include filter and infiltration trenches, permeable surfaces, water storage, swales, water harvesting, detention basins, wetlands and ponds [42,49]. The devices can be structural by employing mainly fixed physical constructions, such as wetlands, ponds and swales. Non-structural devices involve small scale decentralized facilities such as vegetation and also soft measures using knowledge and practice to influence the behavior and attitude of stakeholders, e.g., training and education programs, policies and laws [20,78–80]. In practice, SUDS is often a mix of both types of measures to make the best use of both their functions. Furthermore, SUDS techniques can be centralized measures targeting point source of pollution and/or decentralized small-scale solutions combating diffuse pollution [23]. All the mentioned SUDS devices can be used individually or combined in series to provide services at different temporal and spatial scales.

From a hydrological point of view, SUDS measures can be classified into three groups based on their impacts on the water runoff and routing process [49,81]. The first group refers to source control measures aimed at detaining and attenuating excess water runoff upstream, such as local infiltration, impervious pavements and green roofs. On-site control measures focus on preventing and reducing flood hazard impacts on recipient susceptibility, such as individual assets protection and topographic modification. The third group includes downstream measures concerning the conveyance capacity of the system [16,82].

One successful application of permeable pavements to mitigate stormwater runoff was presented in Jayasuriya *et al.*, which shows the potential of pervious pavement to reduce peak flow and to improve water quality in extreme rainfall [41]. Stewart and Hytiris demonstrated a case study using SUDS techniques (mainly swales, filter drains and an infiltration basin) to mitigate the risk of flooding in a new development area [47]. The results showed a promising performance of the SUDS to provide storage capacity for extreme rainfall and water quality control to meet the good status required by the WFD. Another good example of the utilization of SUDS techniques is introduced by Holman-Dodds *et al.* showing the effects on water runoff by means of manipulating the layout of the urbanized landscape [83]. Nascimento *et al.* [84] presented a case study using a detention basin in combination with upstream infiltration and a grass swale system for local flood management.

Despite the many benefits of SUDS for water quantity and quality management, there have also been questions and skepticism regarding their performance and feasibility. For example, Bergman *et al.* [85] examined the performance of two stormwater infiltration trenches installed in late 90s in central Copenhagen, Denmark, and revealed that the life-span of the infiltration trenches was much shorter than expected due to sand clogging effects. Similar concerns were shared by Achleitner *et al.* [86] on the hydraulic permeability of an infiltration and swale system. Their results show that the measured chemical conditions of the soil material are strongly influenced by the initial background concentrations. Zhou *et al.* described a case study using infiltration trenches and detention ponds to mitigate flood risk under climate change impacts [26]. The paper showed the great potential of detention basins in attenuating water runoff in extreme events and providing additional recreational amenities in the urban landscape. Nevertheless, concerns of the practical operation and maintenance of the ponds were also raised, due to e.g. geological and spatial limitations, problems associated with urban erosion, water pollution and the lack of regulation measures. Furthermore, several studies discussed the limitations of the SUDS techniques in response to the increasing hydrological and hydraulic loading under climate change impacts [19,83,84]. It was found that the SUDS techniques impact water flows; however, the reduction of water volume is very limited in extreme events and sensitive to local conditions, such as size and duration of rainfall event, soil material and texture. Therefore, it is wise to appropriately integrate SUDS and traditional drainage solutions to enhance their synergy for drainage design.

6. Sustainable Assessment Tools

6.1. Models

Nowadays, there are dozens of commercial and open-source software packages available for modelling of sustainable drainage techniques and devices in terms of both water quantity and quality simulations [42,87]. Although methods of drainage modelling have taken a leap forward due to advances in measurement and computational techniques, they are still an approximation of a practical complex phenomenon. Nevertheless, the modelling approaches contribute to an improved process understanding of the SUDS practices (e.g., flow mechanism, sources of pollutants, cause of flooding) and facilitate the application of SUDS in the field [88,89].

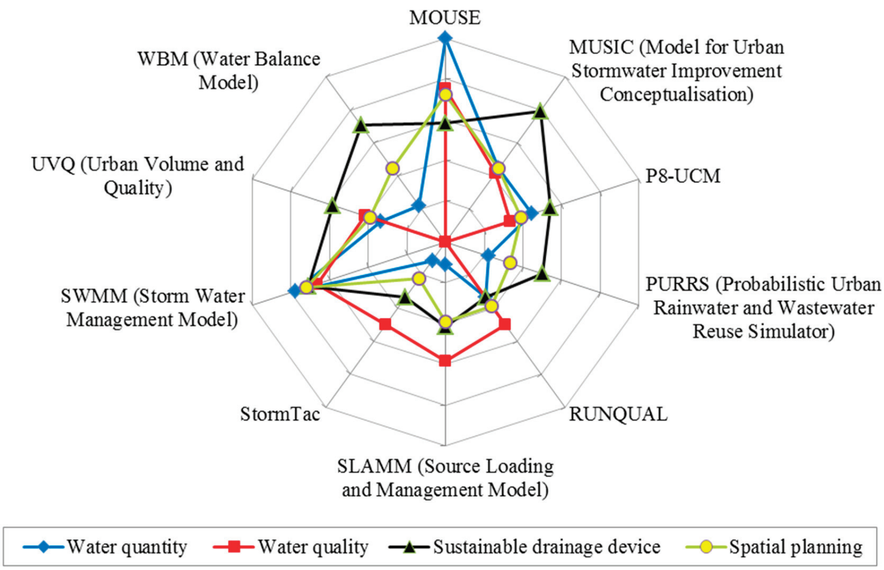
The literature contains details of many modelling approaches employed for SUDS evaluation in different case studies. Elliott and Trowsdale [42] examined 10 modelling methods for SUDS according to their capacity with respect to water quantity and quality simulations, sustainable drainage device modelling and spatial planning, see Figure 1. The paper shows that most of the reviewed models contain functions on hydrological simulation in terms of rainfall generation and runoff routing and only a few are capable of simulating the drainage network hydraulics, such as SWMM and MOUSE (the old version of Mike Urban). Besides PURRS and WBM, all the remaining models include modelling of sediments, nutrients and heavy metals. Regarding the ability to incorporate sustainable devices, most models can be used to investigate reduced imperviousness, ponds and wetlands, infiltration trenches and swales, even though some models do not present the device explicitly. MUSIC and WBM further include bio-retention devices and rain gardens, and only WBM deals with green roofs specifically. Sharma *et al.* present a case study carried out in Canberra, where three modelling tools (Aquacycle, MUSIC and PURRS) are used to predict the effects of alternative scenarios for integrated water management [53]. The study provides more insights into the performance of the modelling tools with regards to their simulation of rainwater tanks, greywater use, swales and ponds, and on-site detention tanks. Mitchell *et al.* [90] reviewed six integrated urban water models (UVQ, Hydro Planner, Krakatoa, Mike Urban, UrbanCycle and WaterCress) based on a quick screen of 65 models in the literature. These models are compared from several aspects, e.g., spatial and temporal representation, climatic inputs, water quality, stormwater, groundwater and wastewater treatment. It was found that all six models cover a good range of spatial scales from lot to region. Only two of the models (Mike Urban and UrbanCycle) are able to model at sub-hourly temporal resolution, whereas the rest of models mainly use a daily time step, which is a very strong limitation for urban drainage applications. Among all the reviewed models, Mike Urban has more advanced water quality algorithms in comparison to Krakatoa, UVQ and Hydro Planner. UrbanCycle and WaterCress barely cover water quality simulation in their methods.

Additionally, there are a few reviews with more specific focus on one or more aspects of SUDS simulation. For example, Knapp, Durgunoglu and Ortel [89] reviewed current rainfall-runoff modelling methods for stormwater management based on e.g., model inputs, applications and modelling procedures. Obropta and Kardos [91] assessed three approaches (deterministic, stochastic and hybrid) to stormwater quality modelling and showed that the hybrid approaches are more promising to reduce model prediction error and uncertainty. Zoppou [87] reviewed 12 models (DR3 M-QUAL, HSPF, MIKE-SWMM, QQS, STORM, SWMM, SWMM Level 1, the Wallingford Model, BRASS, HEC-5Q, QUAL2E-UNCAS and WQRRS) for stormwater quantity and quality simulation and summarized their strengths and limitations in terms of their functionality, accessibility and modelling approaches for water quantity and quality components, and spatial and temporal scales.

Even with the diverse models for SUDS, many of them are still claimed to be non-user friendly because of their technical complexity [92]. Open-source models require a nominal cost; however, they provide very little technical support for users. In contrast, commercial models support the beginners well, but their costs are often too high for widespread use [87]. Concerns are also expressed due to the lack of a shared interface/platform for the different models. Most models have specialized use for only one or a few aspects of SUDS and therefore the simulation is often performed in isolation and only partially reveals the SUDS's effects. It is difficult for users to know and choose which models to apply and how to extend/integrate them for a more comprehensive

SUDS analysis. Even though some models can be poorly integrated, it is tedious and time-consuming to obtain the huge amount of input data for each sub-model [93]. Model integration also faces problems associated with heterogeneous spatial and temporal scales [90,94]. This makes it difficult to transfer and use the data among integrated models and thus demands additional work for data preparation and processing. In particular, commercial models run based on executable files and are difficult to modify to interface with other software [87].

Figure 1. Capacity of various models in terms of water quantity simulation, quality simulation, sustainable drainage device modelling and spatial planning, adapted from Elliott and Trowsdale [42].



The use of geographical information/display (e.g., GIS) in SUDS modeling has also been limited. Most SUDS systems are geographically referenced; integrating SUDS models with GIS system could reduce a huge amount of work on data formatting and process, allowing easy interpretation of model inputs and outputs with a more user-friendly map representation [95]. Certainly, it is also notable that the use of GIS will require large spatial and temporal databases, which are challenging to integrate into existing SUDS models.

6.2. Decision-Aid Tools

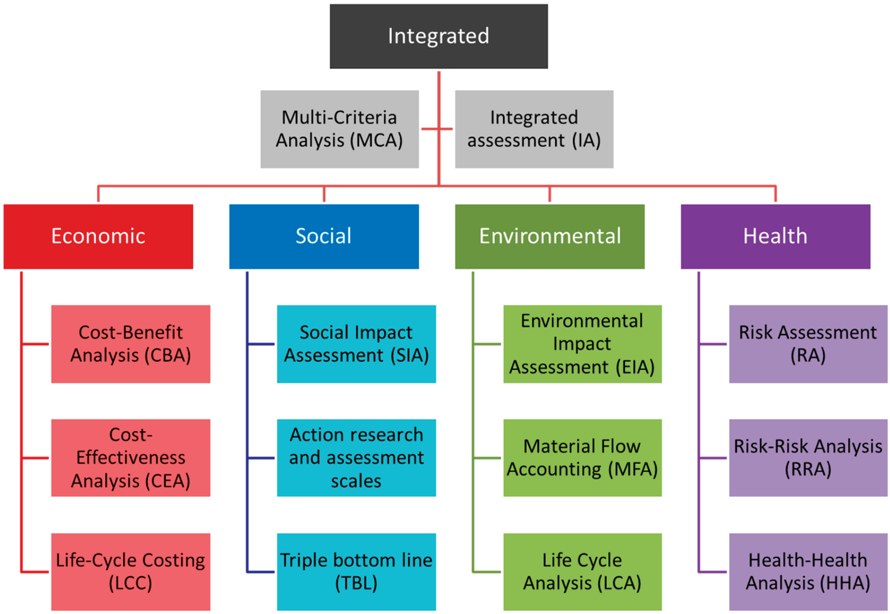
With models of SUDS devices, decision aid tools are further necessary to incorporate the model results and findings in an assessment procedure to facilitate the ranking and selection of drainage alternatives based on the sustainability criteria mentioned previously. Over the years, numerous decision-aid tools emerged in the field to improve the efficiency of decisions; the current review does not attempt to list and discuss all the tools available, just those used commonly for SUDS assessment. A more comprehensive discussion of the various tools can be found in [48,65,96,97].

Figure 2 sorts the reviewed tools into four categories. Economic assessment tools include, but are not limited to Cost-Benefit Analysis (CBA), Cost-Effectiveness Analysis (CEA) and Life-Cycle Costing (LCC). These tools deal with identification and quantification of costs and benefits incorporated in a project/policy [26,48,97]. Social aspects are reflected in tools such as the Social Impact Assessment (SIA), Action research and assessment scales and Triple Bottom Line (TBL) [48,65]. Popular environmental assessment tools for collecting and measuring environmental impacts of projects include Environmental Impact Assessment (EIA), Material Flow Accounting (MFA) and Life-Cycle Assessment (LCA) [96,98]. Health assessment tools are mainly used to evaluate and compare changes in health risks associated with a project. Examples are Risk Assessment (RA), Risk-Risk Analysis (RRA) and Health-Health Analysis (HHA) [97]. Common tools for integrated assessment are Multi-Criteria Analysis (MCA) and Integrated Assessment (IA) [48,65]. These tools use multiple factors to assess the potential of alternative solutions in regard to different criteria. Despite the differences in focus and scope, the tools listed above are not limited to analysis within its category. Depending on the context and the framing of the problem, each tool can be used as an integrated approach for sustainable assessment.

Many recent applications of these decision-aid tools for sustainable drainage assessment can be found in the literature. For instance, Ellis *et al.* employed a multi-criteria analysis to facilitate the evaluation and assessment of SUDS structures for treatment of highway and urban runoff [69]. Carter and Keeler and Zhou *et al.* investigated the performance of vegetated roof systems and open urban drainage systems using the cost-benefit approach and revealed the positive socio-economic benefits of the applied SUDS means [26,98]. Linkov *et al.* reviewed current developments and applications of the comparative risk assessment approach and multi-criteria analysis applied to environmental restoration projects in the United States and Europe [99]. Life cycle cost analysis is applied in Wong *et al.* to assess the economic benefits of rooftop gardens/green roofs in comparison with regular flat roofs [100]. Lai *et al.* examined CBA, TBL, IA and MCA tools to address the important role of integrative approaches in sustainable urban water management [65].

Assessment of SUDS solutions, in many cases, requires discussions and reflections of people's preferences, either expressed in monetary terms or in dimensionless weighting/scoring systems [97,101–103]. There are two general ways to capture people's preference in monetary terms: revealed preference techniques which assess preferences by capturing the behavior of customers and stated preference techniques utilizing survey/interview based techniques to uncover underlying preferences [101]. There are plenty of examples using these techniques: studies by Botzen *et al.* measured people's willingness to pay to avoid negative effects caused by flooding using surveys [104]; Lo and Jim investigated the willingness to pay of residents for conservation of urban green spaces in the city of Hong Kong, based on questionnaires using the contingent valuation method [105]. Similar findings were found by Zhou *et al.*, which revealed additional recreational benefits from urban blue-green features by means of the hedonic pricing method [26]. Kenyon [106] employed workshops to reveal participants' thinking and behavior behind decisions using the multi-criteria approach.

Figure 2. Classification of commonly used decision-aid tools in sustainable drainage assessment [48,65,96,97].



7. Conclusions

Sustainable drainage systems are gaining greater importance as a result of increased acknowledgement of the positive effects of such a system on nature and the environment. This paper performs a literature review of recent developments and applications of sustainable drainage systems around the world. It presents the design criteria and techniques of SUDS and various model approaches and decision-aid tools for simulating and assessing sustainable alternatives for drainage design.

Despite the enrichment of the techniques and tools for SUDS, application of sustainable drainage remains a very challenging task in reality. Although available modelling approaches for SUDS have evolved over many years, they are still limited in their mimicking of the natural response of the devices from both a quantity and quality point of view. Many practical implementations of SUDS tend to underestimate their complexity and therefore the resulting performance is often not satisfactory, due to e.g. a lack of experience of SUDS operation and maintenance, ignorance of interaction with other water bodies, and institutional impediments and barriers towards SUDS practices.

The design of SUDS involves many different disciplines and multidimensional criteria [21]. Nevertheless, most specialists and professionals tend to focus on and prioritize their own fields in the decision making process [64]. As a result, subject-specific techniques/solutions are often applied, which fail to account for important impacts from other fields. An integrated and trans-disciplinary approach will be necessary to incorporate the many disciplines in a common platform to facilitate innovative and sustainable solutions. It is essential for stakeholders to comprehend the broad scope of sustainable design and consider the urban water cycle as a whole planning unit. Meanwhile, climate change and urbanization changes need to be incorporated into the design in order for SUDS to adapt to future changing conditions [72]. In such a context, the future of sustainable drainage design is most likely a mix of both high and low tech solutions to seek a balance between investment cost and performance efficiency. A combination of centralized and decentralized systems will also be necessary to merge the best of the systems and enhance their synergy for sustainable design. To achieve these goals, a design framework integrating technical, social, environmental, economic, legal and institutional aspects will be crucial.

Conflicts of Interest

The authors declare no conflict of interest.

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Selecting Sustainable Drainage Structures Based on Ecosystem Service Variables Estimated by Different Stakeholder Groups

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Abstract: In times of recession, expert systems supporting environmental managers undergo a revival. However, the retrofitting of sustainable water structures is currently undertaken *ad hoc* using engineering experience supported by minimal formal guidance. There is a lack of practical decision tools that can be used by different professions for the rapid assessment of ecosystem services that can be created when retrofitting water structures. Thus the aim was to develop an innovative decision support tool based on the rapid estimation of novel ecosystem service variables at low cost and acceptable uncertainty. The tool proposes the retrofitting of those sustainable drainage systems that obtained the highest ecosystem services score for a specific urban site subject to professional bias. The estimation of variables was undertaken with high confidence and manageable error at low cost. In comparison to common public opinion, statistically significant differences between social scientists and the general public for the estimation of *land costs* using the non-parametric Mann-Whitney U-test were found. It was also surprising to find no significant differences in the estimation of *habitat for species* by civil engineers and ecologists. The new methodology may lead to an improvement of the existing urban landscape by promoting ecosystem services.

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1. Introduction

Traditional drainage often creates flooding and pollution problems in the lower catchment. The implementation of sustainable drainage systems (SuDS; UK) [1], which has similar characteristics to best management practices (USA) and water-sensitive urban design (Australia) [2], can help to solve these problems. The philosophy of SuDS is to promote infiltration of (partially) treated runoff into the ground [1]. Most SuDS techniques support attenuation of runoff before entering the watercourse, storage of water in natural contours, infiltration of partially treated runoff into the ground and evapotranspiration of surface water by vegetation [3–5].

The traditional objective of SuDS is to reduce the negative impact of urbanization on the quantity and quality of surface runoff, while simultaneously increasing amenity and biodiversity opportunities, where possible. SuDS are capable of managing and controlling surface runoff through techniques such as infiltration, detention/attenuation, conveyance and/or rain harvesting [1,6]. Potential improvement opportunities in terms of ecosystem services including aesthetics, amenity and biodiversity by introducing SuDS are often neglected by engineers and planners in practice [5].

Ecosystem services can be integrated within water-sensitive urban design [2] and multi-functional land use planning to maximize wider value opportunities for the benefit of humans and the environment.

The benefits human beings may obtain from the semi-natural (managed) environment can be referred to as ecosystem services [7–9]. Ecosystem services are often defined as the benefits individuals gain from the goods and services produced by nature and its natural systems [10]. The natural resources such as food, timber and water, and functioning natural systems such as healthy fertile soils, clean water [11] and air, and a regulated climate are essential for human wellbeing, security and economic prosperity [7]. A high biodiversity helps to sustain the natural environment and is thus an important factor for ecosystem service provision.

A list of 17 ecosystem service variables and their respective categories is provided in Table 1. The listed ecosystem services have been reinterpreted to make them relevant to SuDS retrofitted in urban areas and are categorized in broad agreement with other guidelines [9,12].

The aim of this article is to outline an innovative decision support tool based on the rapid estimation of novel ecosystem service variables at low cost and acceptable uncertainty. The key objectives to achieve this aim are: (1) to assess the uncertainties of the rapidly estimated SuDS variables based on drainage engineering expert opinion; (2) to evaluate the variability of estimated example variables and the learning process of estimation by different stakeholder groups; and (3) to support the development of a decision support tool for SuDS retrofitting taking into account the perspectives of drainage engineers, developers, ecologists, planners, social scientists and the general public.

The introduction of a transparent weighting system as a function of different professional bias allows for the investigations of "what if" scenarios giving decision-makers more flexibility to test the likely acceptance of various SuDS treatment trains. The tool will improve the urban landscape for the benefit of humans and nature.

Table 1. Ecosystem service variables.

Services	Number	Variable	Abbreviation
Supporting	1	*Habitat for species*	*HS*
	2	*Maintenance of genetic diversity*	*MGD*
Regulating	3	*Local climate and air quality regulation*	*LCAR*
	4	*Carbon sequestration and storage*	*CSS*
	5	*Moderation of extreme events*	*MEE*
	6	*Storm runoff treatment*	*SRT*
	7	*Erosion prevention and soil fertility*	*EPSF*
	8	*Pollination*	*P*
	9	*Biological control*	*BC*
Provisioning	10	*Food*	*F*
	11	*Raw materials*	*RM*
	12	*Fresh water*	*FW*
	13	*Medicinal resources*	*MR*
Cultural	14	*Recreation, and mental and physical health*	*RMPH*
	15	*Tourism and area value*	*TAV*
	16	*Aesthetics, education, culture and art*	*AECA*
	17	*Spiritual experience and sense of place*	*SESP*

2. Methodology

2.1. Site Assessment

A total of 100 sites and corresponding catchment areas that were large enough for the retrofitting of SuDS to have a positive urban drainage impact were identified by studying Ordnance Survey and Google maps of Greater Manchester. Moreover, discussions with local authorities, United Utilities (water authority) and major private land owners regarding suitable SuDS sites were held. The main areas targeted within Greater Manchester were Salford and Manchester.

The standard site assessment template was based on a combination of the frameworks developed by Scholz and his team for retrofitting of SuDS techniques in Glasgow, Edinburgh and elsewhere [4,6], and the Construction Industry Research and Information Association guidelines [1,13]. Each potential SuDS site was assessed during a site visit by a group of experts (2 to 5 team members) to reduce subjectivity [14]. A desk study subsequently supplemented the site visit. The following key information was collected:

1. General site information such as site number and name, postcode, grid reference numbers, location name, names of the inspection team members, site acceptability for SuDS and presence of existing SuDS. Photos of the key site features were taken for each potential SuDS site and its catchment;
2. Land ownership information such as number of owners, ownership type (private or public) and estimated site value (£);
3. Proportions (%) of site classification categories including development, regeneration, retrofitting and recreation;
4. Surrounding area characteristics such as descriptions of the neighborhood to the North, South, East and West, current and future site use, total area of the catchment (m^2), and catchment shape;
5. Location description and distance (m) to the nearest sewer, storm pipe, stream, river, canal, pond, lake and sea, if located within a reasonable distance within or at the border of the catchment;
6. Estimated current and future surface permeability (%) for the land categories grass, trees, shrubs and impermeability of the proposed SuDS site and its catchment;
7. Estimated proportions (%) of current and future roof runoff for the categories institutional, commercial, industrial, high density housing, medium density housing, low density housing and other;
8. Estimated proportions (%) of current and future road runoff for the categories car park, motorway, primary road (or dual carriageway), A road, B road, tertiary road and other.
9. For each sub-catchment, area (m^2) and gradient in the two main directions having an angle of 90° to each other in the horizontal plain;
10. Hydro-geological information such as contaminated land (present or absent), soil infiltration (low, medium or high) and groundwater level (below or above 2 m depth);

11. Additional remarks regarding current drainage techniques and potential problems regarding the implementation of future SuDS techniques.

The information collected with the standard site assessment template supports the assessment team in determining the variables required for the ecosystem services approach.

2.2. Ecosystem Service Variable Assessments

Table 2 shows an overview of the new ecosystem services assessment approach. The potentials of new quantitative and qualitative approaches to assessing ecosystem services have been explored by others [8]. Table 1 shows an overview of the proposed 17 new ecosystem service variables that were also determined for the 100 potential SuDS sites. These variables belong to the established four ecosystem service categories of supporting, regulating, provisioning and cultural (Table 1).

2.3. Uncertainties of the Rapidly Estimated Variables

A relative measure of certainty expressed in percentage points was given to each variable to indicate the reliability of the assessment; the higher the value given, the more certain was the group of assessors. Only values greater than 50% were considered to be acceptable to progress to the next estimation without conducting further studies. Inconsistencies were removed after discussion within the assessment team.

2.4. Variability of Estimated Variables and Learning Process

The approach for evaluating the variability of the randomly selected estimated example variables *aesthetics*, *land cost*, *land size*, *habitat for species* (Figure 1) and *safety* is outlined in this section. Furthermore, the learning process of estimation undertaken by a relevant civil engineering student cohort example is explained with the help of a three-stage questionnaire survey based on a PowerPoint presentation.

Table 2. Overview of the new ecosystem services assessment approach.

Step	Step Description	Comment
1	Select potential sustainable drainage system (SuDS) sites in a case study area	Essential
2	Undertake site visits and note general variables	Essential
3	Desk study for each potential SuDS site	Essential
4	Determine all ecosystem service variables (Table 1) and associated confidence values	Essential
5	Decide on application of a weighting system (if appropriate) for a specific profession (Table 3)	Recommended
6	Decide on dropping variables where the confidence values are too low or undertake further field and/or desk studies	Optional
7	Assess the feasibility of at least the top three proposed SuDS techniques	Recommended

For each variable tested, six corresponding relevant pictures representing virtually the whole numerical spectrum (*i.e.*, very low to very high values; e.g., Figure 1) of possible answers were selected for the questionnaire. The pictures were taken from actual case study sites in Greater

Manchester, and did not contain any misleading or irrelevant information such as distracting objects of random occurrence (e.g., an ice cream van or a pedestrian) in the foreground.

A mixture of 51 full-time BSc, BEng and MEng civil engineering students, who were broadly familiar with the overall case study area and studying water resources technology in their third year at The University of Salford, were asked on 19 March 2013 to assign values to each picture associated with a particular variable.

The questionnaire was split into three different stages to test progressive learning. For each stage, the same pictures had to be assessed. However, the order was changed at random. Approximately 15 seconds were allocated for each picture. At Stage 1, students had to assign values that they had to benchmark against their personal perception. They had to make reasonable assumptions about what is a low or high value for a particular variable. In comparison, at Stage 2, students were aware of the range of possible scenarios for each variable, and had the opportunity to refine their first choices purely based on their memory. In the third and final stage, all pictures associated with a particular variable were shown at the same time. Direct picture comparisons and value readjustments were possible.

Each mean score per picture provided by the student cohort was compared to a target score, which was determined by the research team based on professional drainage engineering perception (e.g., Figure 1). The target score is also subjective (expert opinion) and should therefore only be seen as a guideline to the reader.

2.5. Comparison of Variability with Other Cohorts

The variables *aesthetics*, *land cost*, *habitat for species* and *safety*, which were estimated in Section 2.4 by civil engineers, were also approximated by ecologists and social scientists for comparison. On 3 May 2013, 42 undergraduate students studying ecology at The University of Salford were tested. Furthermore, 31 undergraduate social science students were questioned at the same university on 1 May 2013. The same methodology as presented in Section 2.4 was applied. However, Stage 2 of the learning process was omitted.

The variables *aesthetics*, *land cost*, *habitat for species* and *safety* were also estimated by 49 randomly chosen members of the general public between 26 June and 25 July 2013. However, only Stage 3 (see Section 2.4) was applied; *i.e.*, all subjects were only presented with six pictures per variable in random order on a single sheet. The questionnaire survey can be found on the web [15]. The questionnaire will remain live at least until 25 December 2013, and further participation is still welcome.

The general public sample comprised subjects with the following backgrounds or professions: unidentified students (10%), civil engineering students (10%), engineers (33%), ecology students (0%), ecologists (12%), social science students (0%); developers (2%), planners (2%) and others (31%). Engineers and students are overrepresented in this sample. In contrast, members of the public with a below-average education are underrepresented.

Figure 1. Relative ranking values for the variable *habitat for species* (%). Ascending order (*i.e.*, from highly inadequate to highly adequate habitat) based on the authors' expertise: (**a**) 9%; (**b**) 23%; (**c**) 45%; (**d**) 62%; (**e**) 70%; and (**f**) 82%. All photographs were taken by the authors and Nathan Somerset in 2012 and 2013 (The University of Salford).

(**a**) (**b**)

(**c**) (**d**)

(**e**) (**f**)

2.6. Decision Support Tool for Different Professions

This section outlines the methodology for the development of a decision support tool for SuDS retrofitting taking into account the perspectives of drainage engineers, developers, ecologists, planners, social scientists and the general public as defined elsewhere [16]. A weighting system specific to the needs of a particular stakeholder group was introduced by providing weights for individual variables (Table 3) after consultation with different teams of academics representing different professions within The University of Salford.

Table 3. Weights for ecosystem service variables (Table 1).

Variable	Weights subject to bias					
	Drainage Engineer	Developer	Ecologist	Planner	Social Scientist	General Public
1	1	1	3	2	2	1
2	1	1	3	1	1	1
3	1	1	3	2	3	2
4	1	1	3	1	1	1
5	3	3	2	3	2	3
6	3	2	2	2	2	2
7	2	2	2	2	2	2
8	1	1	3	1	1	1
9	1	1	3	2	2	2
10	1	1	1	1	2	1
11	1	1	1	1	2	1
12	3	1	2	2	2	2
13	1	1	1	1	2	1
14	2	2	1	2	3	2
15	1	3	1	2	3	3
16	1	2	1	2	3	1
17	1	2	1	2	3	2

Variables of low relevance for a drainage engineer such as *MR* (see Table 1) in Greater Manchester were assigned with a low weight, while variables with a medium (e.g., *RMPH*) or high (e.g., *MEE*) relevance were assigned with a medium or high weight, respectively. Table 3 proposes weights from the viewpoint of different professionals (drainage engineer, developer, ecologist, planner, social scientist and the general public). A simple weighting system with only three categories (1, low; 2, normal; 3, high) has been proposed to keep the case study example simple. A maximum weight of 3 signifies that one variable is three times more important than a variable scoring only 1. However, the reader may wish to replace the proposed system by a more differentiated weighting system based on, for example, ten categories. Depending on the case study location and associated boundary conditions, end-users of the proposed tool may wish to select different weights, which will subsequently impact on the results. It is up the group of experts to decide if a weighting scale should be used and what weights may be appropriate for a particular case study. However, transparency in decision-making is essential.

2.7. Data Analysis

Microsoft Excel [17] was used for data storage and the general data analysis. The non-parametric Mann-Whitney U-test was computed using IBM SPSS Statistics Version 20 [18] and used to compare the medians of two (unmatched) independent samples. This was required because virtually all sample data (even after data transformation) were not normally distributed, so that an analysis of variance could not be applied.

3. Results and Discussion

3.1. Findings of the Assessment Method

Table 2 summarizes the new ecosystem services assessment approach applied to 100 potential example SuDS sites in Greater Manchester. Most ecosystem service variables did relate well to the natural environment such as biologically diverse parks (41% of all sites) and not to the built environment like impermeable car parks (33% of all sites). This relationship reduces the number of sites suitable for retrofitting of most SuDS, as car parks usually only perform well with respect to three ecosystem service variables [*moderation of extreme events* (*MEE*), *storm runoff treatment* (*SRT*) and *fresh water* (*FW*); Table 1]. The presence of public parks did not pull up the overall suitability of retrofitting sites, because they were usually small in size (30% of sites were <25,000 m^2), low in tree coverage (7%) and the presence of surface water [stream (0%), river (11%), canal (21%) and standing water (8%)] of the associated catchment was limited. However, the introduction of a weighting system (Table 3) that puts bias towards what a drainage engineer would perceive as more important variables for SuDS (e.g., flood control as part of *MEE* and water quality control considered by SRT) could increase the suitability of sites for retrofitting.

Table 4 shows the assessment approach in terms of proposed SuDS techniques for Greater Manchester. The relative proportions for each SuDS technique have been expressed in percentage points for all selected professions. Note that there were many occasions where more than one SuDS technique had the same order of preference.

Table 5 shows a comparison of the inter-site variability for a given sustainable drainage technique for Greater Manchester, and helps to interpret the preference distributions in Table 4. The relatively high variability for most variables such as ponds and constructed wetlands cannot be explained by factors relating to specific planning policies for Greater Manchester. Ponds are associated with the greatest inter-site variability because of their potentially relatively small size and great popularity [5,6,19].

Table 4. Drainage system preferences*.

Profession	Sustainable Drainage System	First	Second	Third
Drainage engineer	Permeable pavement	43	9	4
	Filter strip	2	7	12
	Swale	0	2	12
	Green roof	0	0	3
	Pond	33	11	4
	Constructed wetland	11	1	2
	Infiltration trench	5	9	44
	Soakaway	0	4	15
	Infiltration basin	1	4	8
	Belowground storage	5	44	13
	Water playground	3	17	9
Developer	Permeable pavement	42	13	12
	Filter strip	11	23	14
	Swale	1	13	11
	Green roof	0	0	1
	Pond	36	9	1
	Constructed wetland	8	6	1
	Infiltration trench	2	32	23
	Soakaway	3	1	34
	Infiltration basin	1	1	8
	Belowground storage	0	11	23
	Water playground	1	2	6
Ecologist	Permeable pavement	39	7	12
	Filter strip	13	22	22
	Swale	2	13	22
	Green roof	0	1	2
	Pond	30	13	5
	Constructed wetland	10	1	3
	Infiltration trench	8	33	26
	Soakaway	1	8	17
	Infiltration basin	2	8	12
	Belowground storage	1	13	32
	Water playground	5	19	8
Planner	Permeable pavement	39	8	6
	Filter strip	8	11	29
	Swale	1	6	17
	Green roof	0	1	1
	Pond	31	12	1
	Constructed wetland	10	1	1
	Infiltration trench	0	6	25
	Soakaway	0	3	16
	Infiltration basin	0	2	9
	Belowground storage	5	42	14
	Water playground	5	19	7

Table 4. *Cont.*

Profession	Sustainable Drainage System	First	Second	Third
Social scientist	Permeable pavement	39	7	6
	Filter strip	12	24	19
	Swale	0	1	11
	Green roof	0	1	0
	Pond	33	10	0
	Constructed wetland	10	0	1
	Infiltration trench	0	9	31
	Soakaway	0	2	20
	Infiltration basin	0	2	3
	Belowground storage	2	33	18
	Water playground	5	20	5

Note: * Proportion (%) of sites at which sustainable drainage system techniques are given first, second or third order of preference based on different professional perspectives (weights in Table 3). Note that numbers not necessarily add-up to 100, because some techniques received the same preferences.

Table 5. Inter-site variability* comparison for a given sustainable drainage technique.

Sustainable Drainage System	Drainage engineer	Developer	Ecologist	Planner	Social Scientist
Permeable pavement	21	17	16	19	16
Filter strip	16	18	19	19	18
Swale	15	17	17	17	13
Green roof	5	0	6	5	5
Pond	31	36	33	32	31
Constructed wetland	21	25	23	21	19
Infiltration trench	13	9	13	12	11
Soakaway	7	5	9	6	5
Infiltration basin	13	16	12	12	11
Belowground storage	17	15	13	15	13
Water playground	18	17	17	19	20

Note: * indicated by the standard deviation based on relative percentage points awarded.

It may come as a surprise that permeable pavements scored relatively highly on ecosystem service variables (Table 4), which contradicts the common belief among some engineers that there has to be a strong bias towards natural and soft techniques when using ecosystem service assessment techniques [5,20]. However, permeable pavements are likely to attract high values for variables such as *SRT* and *MEE*, respectively, if properly designed and managed.

3.2. Expert Judgment

The estimation of certainties associated with expert judgment needs to be undertaken consistently to be informative. Human judgment may vary considerably, and involves an appreciation of reality and what is a realistic solution to a given problem and an understanding of the importance of making the right choice about what action to take [21]. Confidence estimations are affected by ones familiarity of a topic, experience with probabilistic assessments, the level of difficulty of a task, and the environmental context in which the task is performed [22].

Research has proven that a group's level of judgment usually outperforms that of an average individual due to the sharing of responsibility between the group members. This sharing, in turn, leads to an increase in their confidence to communicate judgments [23].

Knowledge used by engineers to make judgments is not entirely of scientific nature, although a substantial part is derived by science, but is based on experimental evidence and on empirical observations of materials and systems. Understanding is built-up over time as a result of continuous unquantifiable but improving judgments and choices [24,25]. The introduction of a weighting system can address differences between assessor groups with different scientific backgrounds.

Previous studies indicate that good expert judgment performance can be observed when both the scientific validity of an estimated observation and the learnability of the estimation by the assessor are high. Poor expert opinion may occur if at least one of these factors is low [26]. Most variables (Table 1) to be estimated in the proposed SuDS retrofitting tool are strongly scientifically valid, and their estimation is uncontroversial and easy to learn (e.g., *SRT* and *FW*). Therefore, this paper focuses on the estimation of some of those more controversial variables that are highly subject to personal opinion and taste (*aesthetics* and *safety*), difficult to learn due to their highly dynamic nature in terms of time and space (*land cost*), and scientific complexity (*habitat for species*).

For example, the indirect assessment of biodiversity predominantly through the supporting ecosystem service variables *habitat for species* and *maintenance of genetic diversity* is difficult due to its scientific complexity in terms of sustainability assessment and ecosystem valuation. Any rapid and cost-effective screening method should preferably be undertaken by experts in order to avoid obtaining poor results based on guesses. In comparison, traditional biodiversity assessments are time-consuming and costly. Therefore, this paper assesses this challenge by researching to what degree users with different experience and scientific background (see Section 3.4) come up with similar findings.

3.3. Variability and Learning Process

An estimation tool has to be relatively simple to learn and apply [26], and should be based more on intuition than on expert understanding to limit the variability associated with estimations for the same variable by different assessors with potentially diverse backgrounds. Table 6 shows the findings of the questionnaire analysis. Figure 1 shows the relative ranking values for the variable *habitat for species* (%) in ascending order (*i.e.*, from highly inadequate to highly adequate habitat).

The example variables *aesthetics* and *land costs* were determined relatively well (Table 6). In comparison, *habitat for species* (Figure 1 and Table 1) and *safety* were associated with higher but still acceptable estimated errors. This can be explained by the high complexity of these variables (see Section 3.2). The cohort had serious difficulties in estimating *land size*. Nevertheless, this is not considered to be a problem, because *land size* can be easily measured in the field or estimated using maps.

Considering that the concept of "estimation" was new to the students, and they were neither briefed nor trained in advance of the questionnaire, someone might expect considerable progressive learning from stage to stage. However, learning only improved clearly for *land size* estimation between all stages (Table 6). Moreover, the authors expected to identify a clear reduction in variability (indicated by the standard deviation) as learning progressed. Nevertheless, this was not the case (Table 6).

Table 6. Summary of the questionnaire analysis* for the civil engineering student cohort.

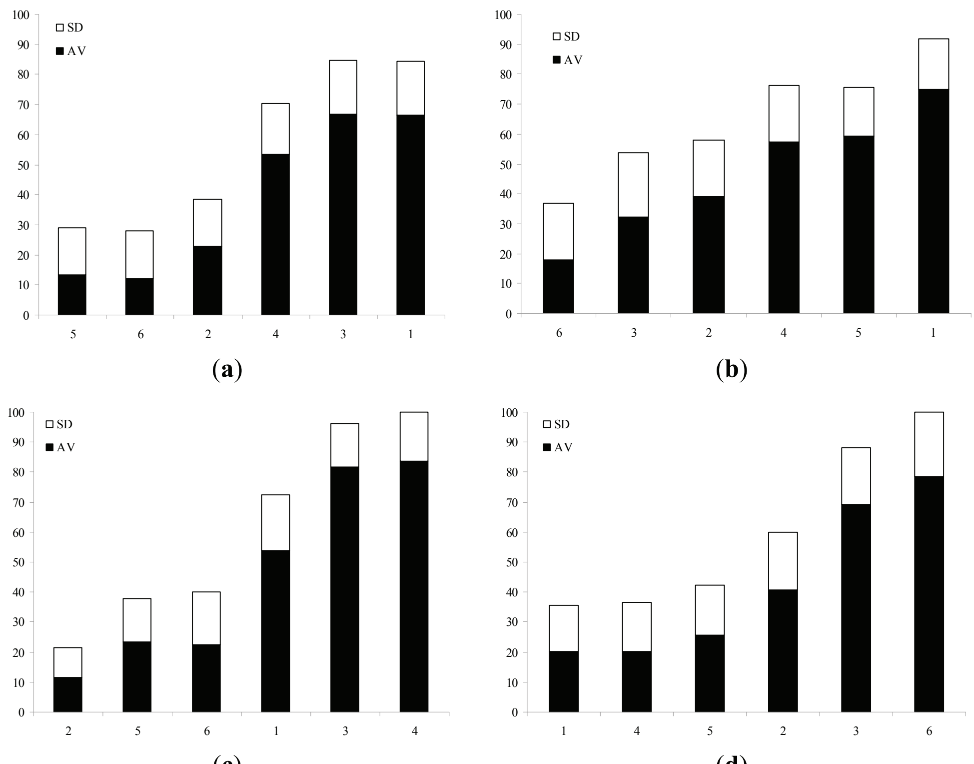
Picture number	Target score	Stage 1		Stage 2		Stage 3	
		Mean	STDEV[a]	Mean	STDEV[a]	Mean	STDEV[a]
Aesthetics (%), which is part of variable 16 (*Aesthetics*, *education*, *culture and art*; Table 1)							
1	30	36	20.9	29	22.0	31	24.4
2	43	35	18.3	36	18.8	40	17.8
3	49	48	22.4	41	27.2	39	24.2
4	62	55	10.6	57	15.5	63	14.8
5	74	58	21.1	65	19.4	69	22.2
6	82	64	23.9	61	22.0	69	20.5
Land size (m^2), which influences all variables (Table 1)							
1	3240	6370	11,613	8510	19,523	8400	14,302
2	4600	8540	11,621	14,630	25,144	10,990	18,423
3	8200	11,560	23,187	10,790	23,532	21,100	59,486
4	9440	57,010	216,610	16,040	35,940	21,690	48,024
5	10,350	49,520	69,104	63,160	149,055	56,650	91,580
6	70,000	123,470	436,125	84,940	159,947	70,790	101,090
Land cost (%), which is part of variable 15 (*Tourism and area value*; Table 1)							
1	27	27	24.9	25	20.0	25	21.9
2	35	42	15.0	45	17.7	44	17.4
3	54	53	22.4	58	21.6	59	22.4
4	60	58	19.3	62	17.1	60	20.3
5	69	65	19.7	63	19.0	64	18.9
6	78	71	17.9	68	18.5	70	20.2
Habitat for species (%), which is variable 1 (Table 1)							
1	9	10	13.2	16	21.5	16	20.6
2	23	30	17.5	29	18.9	28	20.4
3	45	35	22.0	38	20.3	40	19.5
4	62	52	24.4	53	16.7	56	17.5
5	70	67	19.4	62	21.3	64	20.0
6	82	69	23.2	68	23.8	74	23.3
Safety (%); which is part of variable 14 (*Recreation, and mental and physical health*; Table 1)							
1	20	21	20.7	22	20.0	26	32.2
2	29	24	22.6	27	21.6	27	21.2
3	34	33	20.4	32	20.6	31	22.9
4	40	46	24.3	45	22.8	47	32.3
5	62	46	23.9	45	25.2	53	22.5
6	74	59	35.7	61	30.4	64	32.7

Notes: * indicating the variability for example variables and progressive learning; [a] standard deviation.

Figures 2–4 show the findings for the ecology students, social science students and the general public, respectively. The standard deviations associated with variable estimations were usually lower for the ecology compared to the civil engineering students. In comparison, the same was the case for social science students (except for *aesthetics* and *habitat for species*). The standard deviations for ecology and social science students and the general public were rather similar.

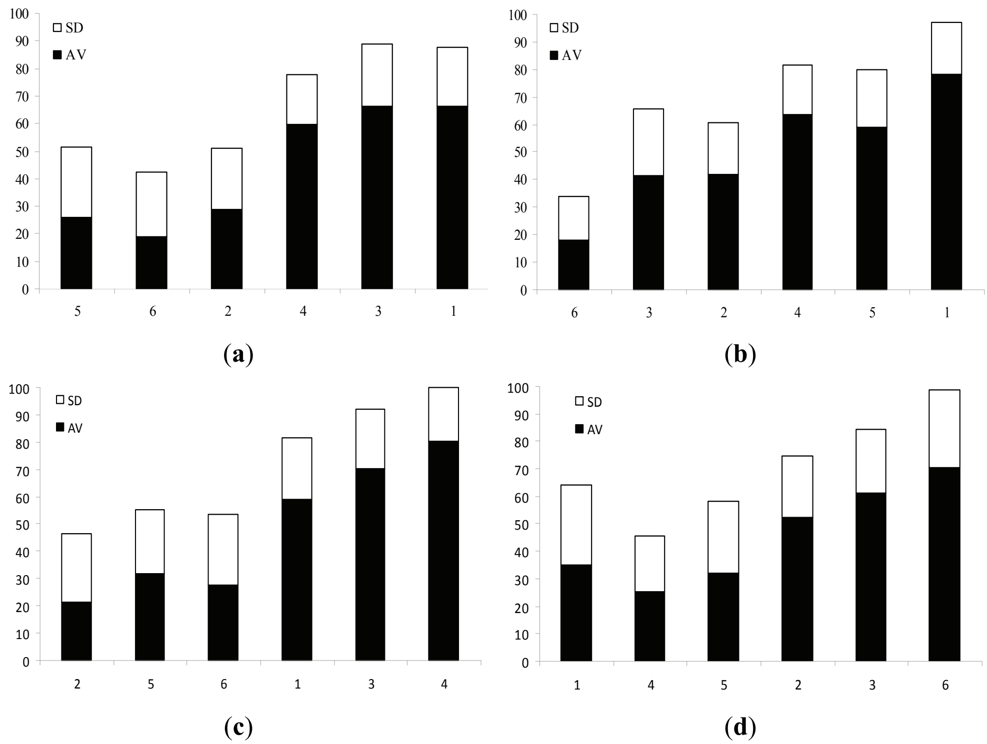
Table 7 shows an assessment of the statistically significant differences between different cohorts of estimators for selected SuDS characterization variables using the non-parametric Mann-Whitney U-test. There were five relationships that could be considered as unexpected with respect to commonly hold public opinions. Civil engineering compared to ecology students had similar views regarding *habitat for species* (P = 0.994; Table 7) and *safety* (P = 0.494; Table 7). However, one might assume that *habitat for species* would be much more important to ecologists than engineers. On the other hand, engineers are usually more aware of health and *safety* matters than ecologists.

Figure 2. Stage 3 estimations (%) by ecology students for the variables (**a**) *aesthetics*; (**b**) *land cost*; (**c**) *habitat for species*; and (**d**) *safety* based on different pictures represented by numbers on the x-axis. SD, standard deviation; AV, average.



Someone might expect that civil engineering and social science students might have different views regarding *habitat for species*. However, the study showed that the data were rather similar (P = 0.379; Table 7). It could be expected that ecology students would have a different opinion regarding *habitat for species* compared to the general public. However, their assessments were rather similar (P = 0.072; Table 7), which is surprising considering that ecologist should have a better understanding of the associated science and might therefore have different assessment criteria. Finally, social scientists and the general public might be expected to have similar opinions with respect to the estimation of *land costs*. However, their estimations were significantly different (P = 0.006; Table 7), which could be explained by the dominance of engineers in the general public sample.

Figure 3. Stage 3 estimations (%) by social science students for the variables (**a**) *aesthetics*; (**b**) *land cost*; (**c**) *habitat for species*; and (**d**) *safety*. based on different pictures represented by numbers on the x-axis. SD, standard deviation; AV, average.

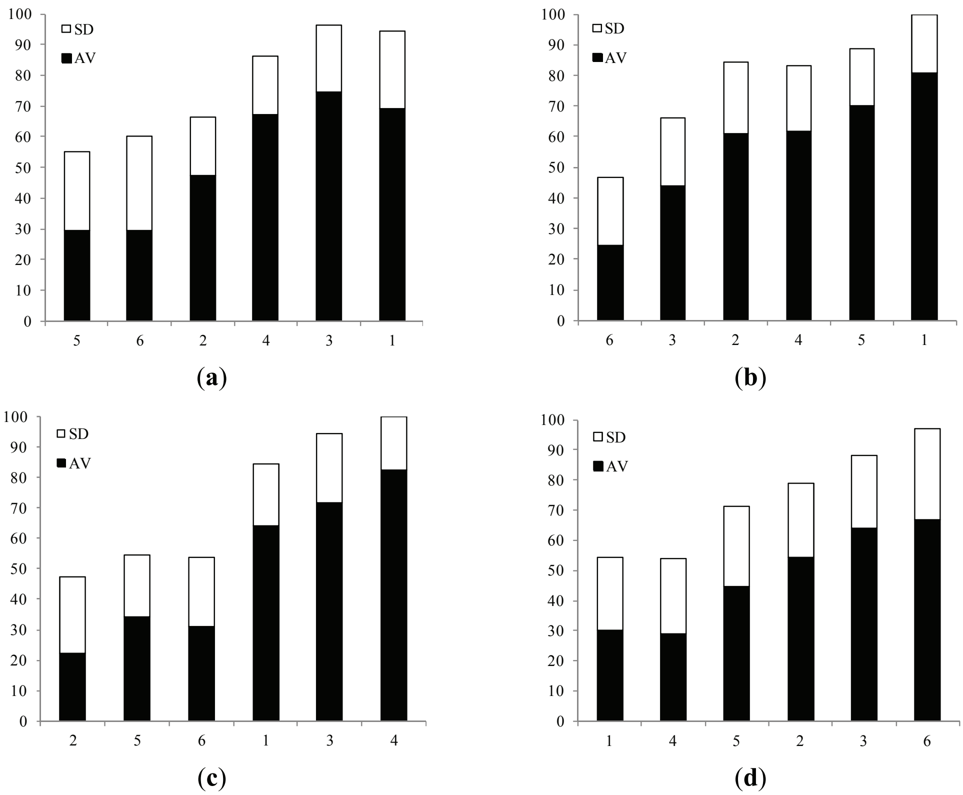


3.4. Different Professional Perspectives

Different professions will want to assign a higher importance to those variables that are of greater relevance to their interests (Table 4). Therefore, the new tool takes into account the diversity of professional opinions by giving any user the opportunity to select a weighting system (Table 3) of greatest relevance to his or her line of thought. However, the introduction of associated bias can be avoided by not selecting any weighting system.

In case a result that is free of any bias and error associated with the estimation by a specific cohort is preferable, the findings in Section 3.3 can be used to adjust the estimation results. For example, if an estimation is made by cohort A for a variable x, and it is known that A consistently overestimates x by 10% compared to all other relevant cohorts, x could be reduced by 10%, which would result in an estimation more acceptable by the majority of stakeholders. With respect to this study, the general public sample is dominated by engineers (at least 43%; Section 2.5). Considering that engineers consistently overestimate *aesthetics* for less beautiful (<50% for aesthetics) SuDS sites in comparison to, for example, ecologists and social scientists (Table 6; Figures 2 and 3), their estimations could be reduced by at least 15% and 5%, respectively, to bring them in line with those made by ecologists and social scientists. Such relationships can be formalized in numerical models based on uncertainty estimations associated with different cohorts and variables [27].

Figure 4. Stage 3 estimations (%) by the general public for the variables (**a**) *aesthetics*; (**b**) *land cost*; (**c**) *habitat for species*; and (**d**) *safety.* based on different pictures represented by numbers on the x-axis. SD, standard deviation; AV, average.



3.5. Strengths and Limitations

The strengths of the new ecosystem services approach to SuDS retrofitting, particularly in comparison to the community and environment methodology adopted by others [13,28], are as follows:

- Generic retrofitting approach based on universal ecosystem service variables;
- Recognition that various professions have different priority variables;

- Expert judgment may be more accurate than prediction models if the science base is strong, the learnability high and sufficient information is available [21,26];
- Inexpensive, user-friendly and easy-to-understand evaluation; and
- Overall ecosystem service potential of a site expressed through an individual value.

The potential weaknesses of the ecosystem services assessment approach are:

- Subjectivity and aggregation are generic limitations of an expert-based system, which can be addressed by involving expert groups and determination of uncertainty values for all estimations [14,29,30];
- Some ecosystem service variables are not always applicable;
- Strong perceived (often falsely; see below) bias towards natural sites and "soft" SuDS (e.g., ponds and wetlands) in contrast to urban sites and "hard" SuDS (e.g., permeable pavements and belowground storage systems); and
- Possibility of multicollinearity among variables due to potential dependencies between some of them [31].

Table 7. Assessment of the statistically significant differences between different cohorts of estimators (civil engineering, ecology *and* social science students, and the general public) for selected SuDS characterization variables (*aesthetics*, *land cost*, *habitat for species* and *safety*) using the non-parametric Mann-Whitney U-test (see also Section 2.7).

Cohort comparisons	**Statistic**	***Aesthetics***	***Land cost***	***Habitat for species***	***Safety***
Civil engineers and ecologists	*P*	0.000	0.004	0.994	0.494
	H	1	1	0	0
Civil engineers and social scientists	*P*	0.004	0.157	0.379	0.027
	H	1	0	0	1
Civil engineers and the general public	*P*	0.396	0.094	0.050	0.002
	H	0	0	0	1
Ecologists and social scientists	*P*	0.070	0.183	0.500	0.175
	H	0	0	0	0
Ecologists and the general public	*P*	0.000	0.000	0.072	0.018
	H	1	1	0	1
Social scientists and the general public	*P*	0.002	0.006	0.311	0.453
	H	1	1	0	0

Notes: *P* value, probability of obtaining a test statistic at least as extreme as the one that was actually observed, assuming that the null hypothesis is true; H, response indicator; if H = 1, filters are statistically significantly different ($P < 0.05$) for the corresponding water quality parameter; if H = 0, the difference is not significant.

Some of the above limitations such as subjectivity are also inherent in traditional assessment approaches [1,13]. However, multicollinearity might be a more relevant problem with the proposed ecosystem services approach due to the use of a high number of variables. In order to avoid artificial dependencies between some variables that could be considered as similar by the inexperienced assessor, all assessors need to be clear about their differences, which require training by more experiences evaluators. Considering that any tests for multicollinearity is case study-dependant, the inevitable bias associated with a case study does not allow for objective testing unless the number of case studies is very high and there is an adequate geographical spread to reduce bias. Nevertheless, a principal component analysis was carried out to identify redundant variables in order to reduce the risk of multicollinearity [31]. Findings indicate that all ecosystem services variables (Table 1) were considered to be necessary for the proposed expert system.

4. Conclusions and Recommendation for Further Research

A rapid estimation-based assessment methodology for retrofitting of SuDS was successfully introduced. This tool can be used together with water-sensitive urban design, multi-functional land use planning and regeneration strategies to prioritize sites for SuDS retrofitting, which is particularly important during difficult financial times.

The variable estimations and the assignment of associated confidence figures were based on expert judgment. However, findings show that estimation errors and variability are relatively low even for virtually untrained example cohorts. The introduction of a transparent and justified weighting system as a function of different professional bias leads to the preferred selection of some SuDS techniques by several professions. This methodology allows for the investigations of various "what if" scenarios giving decision-makers more flexibility to test the likely acceptance of various SuDS treatment trains.

Statistically significant differences between different cohorts of estimators for selected SuDS characterization variables using the non-parametric Mann-Whitney U-test were not found for about half of the possible combinations of cohorts. However, there were four of these relationships that could be considered as unexpected with respect to commonly hold public opinions. Civil engineering compared to ecology students had similar views regarding *habitat for species* and *safety.* Someone might also expect that civil engineering and social science students might have different views regarding *habitat for species*. However, the study showed that the data were rather similar. It could also be expected that ecology students would have a different opinion regarding *habitat for species* compared to the general public. However, their assessments were rather similar.

In comparison, statistically significant differences between cohorts for SuDS characterization variables using the non-parametric test that were surprising, were only found for social scientists compared to the general public, where someone might expect similar opinions concerning the estimation of *land costs*. However, corresponding estimations were significantly different.

More research on estimation adjustments to eliminate cohort bias, variability and errors would be welcome. Moreover, larger data sets would be beneficial in making judgments with higher confidence. It is therefore recommended to test the tool in different towns and cities to prove its validity for other case study scenarios.

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Conflicts of Interest

The authors declare no conflict of interest.

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