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**Key Points:**
- A framework is presented for planning cluster scale systems incorporating IUWM
- Rain water use is not Pareto-optimal; in general, wastewater reuse is better
- Nontraditional water sources are competitive in cost and energy requirements

**Supporting Information:**
- Readme
- 3-D rotating Pareto front
- Additional figures
- IUWM problem: cost energy model supplement
- IUWM Problem: sizing supplement

**Correspondence to:**
J. P. Newman,
jeffrey.newman@adelaide.edu.au

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# Multiobjective optimization of cluster-scale urban water systems investigating alternative water sources and level of decentralization

**J. P. Newman[1], G. C. Dandy[1], and H. R. Maier[1]**

[1]School of Civil, Environmental, and Mining Engineering, University of Adelaide, Adelaide, South Australia, Australia

**Abstract** In many regions, conventional water supplies are unable to meet projected consumer demand. Consequently, interest has arisen in integrated urban water systems, which involve the reclamation or harvesting of alternative, localized water sources. However, this makes the planning and design of water infrastructure more difficult, as multiple objectives need to be considered, water sources need to be selected from a number of alternatives, and end uses of these sources need to be specified. In addition, the scale at which each treatment, collection, and distribution network should operate needs to be investigated. In order to deal with this complexity, a framework for planning and designing water infrastructure taking into account integrated urban water management principles is presented in this paper and applied to a rural greenfield development. Various options for water supply, and the scale at which they operate were investigated in order to determine the life-cycle trade-offs between water savings, cost, and GHG emissions as calculated from models calibrated using Australian data. The decision space includes the choice of water sources, storage tanks, treatment facilities, and pipes for water conveyance. For each water system analyzed, infrastructure components were sized using multiobjective genetic algorithms. The results indicate that local water sources are competitive in terms of cost and GHG emissions, and can reduce demand on the potable system by as much as 54%. Economies of scale in treatment dominated the diseconomies of scale in collection and distribution of water. Therefore, water systems that connect large clusters of households tend to be more cost efficient and have lower GHG emissions. In addition, water systems that recycle wastewater tended to perform better than systems that captured roof-runoff. Through these results, the framework was shown to be effective at identifying near optimal trade-offs between competing objectives, thereby enabling informed decisions to be made when planning water systems for greenfield developments.

## 1. Introduction

Water supply security and environmental criteria are increasingly important aspects of water system design. Water supply security is particularly important in water-stressed regions, and will become increasingly important due to (1) population growth, (2) increases in per capita demand, (3) increased demand for environmental flows, and (4) climate change [*Gleick*, 2000, 2003; *Vörösmarty et al*., 2000]. In regard to environmental criteria, water system planners are increasingly challenged to restore natural hydrologic regimes, reduce pollution, and reduce greenhouse gas emissions arising from water infrastructure.

To address these criteria, a new urban water paradigm, called integrated urban water management (IUWM), has emerged in recent decades. When applied during the planning and design of cluster-scale water infrastructure, IUWM addresses some of the issues outlined above by (1) incorporating a number of alternative water sources; (2) integrating water supply, sewage disposal and stormwater as components of one system, allowing the capture and reuse of rainwater, stormwater, and wastewater; (3) distributing satellite treatment plants across urban areas, thereby decentralizing service provision; (4) emphasizing demand management approaches; and (5) considering multiple sustainability indicators of system performance [*Mitchell*, 2006; *Newman and Mouritz*, 1996; *Radcliffe*, 2004; *Niemczynowicz*, 1999].

As a result of these features of IUWM, the planning and design of water systems that apply IUWM principles tend to be more complex than those of conventional systems, because (1) multiple alternative combinations of water sources may be suitable for each household end use of water; (2) different aspects of the urban water cycle need to be considered, due to the integrated approach to rainwater, wastewater, stormwater and water supply services; (3) there is a requirement to select the number and location of decentralized treatment plants (that is to say, the level of decentralization); (4) multiple collection and distribution networks may need to be designed; and (5) there is a need to assess multiple sustainability indicators relating to the economic, environmental, social, and technical performance of the system. Furthermore, as multiple decisions need to be made regarding water sources, treatment trains, and scale, in addition to choices regarding distribution and collection networks, the decision space is larger. Consequently, identifying the optimal components of the conveyance, treatment, and storage infrastructure for water supply, as well as their sizes, is more challenging.

Given the discussion above, three issues are critical when planning and designing integrated urban water systems, including (i) the selection of alternative water sources, (ii) the identification of optimal water systems, which is difficult due to the complexity of the planning problem, and (iii) the determination of the level of decentralization for collection, treatment, storage, and distribution infrastructure.

In regard to the first issue of selecting alternative water sources, a number of studies have compared the potential water savings achievable by using different sources [*Alegre et al*., 2004], while others have made this comparison also in terms of pathogen risk, water quality aspects, and costs [*Gray and Booker*, 2003; *Sharma et al*., 2008, 2009; *Zhang et al*., 2010; *Makropoulos et al*., 2008; *Rozos et al*., 2010; *Ghisi and de Oliveira*, 2007]. However, only *Kang and Lansey* [2012] and *Penn et al*. [2013] compared the design of infrastructure systems across alternative water sources, although these studies only considered the distribution network and collection network, respectively. Therefore, previous research has often neglected important performance indicators, has not always considered some of the infrastructure aspects that could have a significant impact on the economic and environmental performance of urban water systems, and has not investigated the marginal benefits of different levels of decentralization.

In regard to the second issue of selecting alternative water sources, it might be counterintuitive that the use of multiple collection, treatment, storage, and distribution network infrastructure for supply of alternative water streams could be optimal for the provision of services, given the increased cost of additional infrastructure. However, this is not necessarily the case, as highlighted by *Mitchell* [2006]. This is because alternative water sources increase supply security in arid regions, and because treatment requirements may be lower and conveyance distances are shorter for locally sourced water streams. However, in order to address the potential benefits of such approaches in an holistic manner, the interactions between choices of water sources and infrastructure design need to be taken into account, requiring larger search spaces to be explored. Therefore, the application of formal optimization techniques is likely to be beneficial, as this potentially opens the door to unforeseen results and new and innovative solutions that perform better in terms of the selected objectives, leading to the most efficient use of resources, especially when a very large number of infrastructure and policy options is available. Some studies have utilized formal optimization for the planning and design of systems that incorporate IUWM principles, although often the formulation of the problem could have been improved. For example, while *Makropoulos et al*. [2008] considered multiple objectives for the optimal selection of in-house water use devices, they combined these into a single objective via an appropriate weighting method. However, the use of multiobjective optimization is preferred, as its use enables better understanding of the trade-offs in performance objectives in a less biased way. Other formal optimization studies have only considered certain infrastructure aspects of integrating urban water systems [*Kang and Lansey*, 2012], which is a limitation due to the interactions that occur between collection, distribution, and treatment components. Furthermore, some studies have applied optimization to decision spaces that were arguably too small to warrant the use of optimization [*Rozos et al*., 2010]. However, given the objectives of IUWM, it would be beneficial to formulate the optimization problem in a way that: (1) has broadened scope to include high-level decisions regarding choices of water source, and the design of storage, treatment, and conveyance infrastructure, so that systems representing the optimal trade-offs between competing sustainability objectives can be identified, and (2) uses computationally efficient evolutionary *multiobjective* techniques so that optimal designs can be identified by only evaluating a small proportion of the large search space.

In regard to the third issue (the determination of the level of decentralization for collection, treatment, storage, and distribution infrastructure), a number of diseconomies and economies of scale exist in water systems. For example, diseconomies of scale are present in pipe networks, while economies of scale exist in treatment [*Kim and Clark*, 1988; *Abbott and Cohen*, 2009]. While there have been some studies that have considered scale [*Booker*, 1999; *Clark*, 1997; *Jeffrey et al*., 1999; *Fagan et al*., 2010], there have been no works quantifying how the level of decentralization, which has an impact on the size of infrastructure components and networks, affects the marginal costs, carbon footprint, and water cycle in small water systems that incorporate IUWM principles. While the diseconomies of scale associated with pipe systems are weak in small systems as a result of minimum pipe diameter contraints, some IUWM technologies tend to be implemented separately at each household (such as roof-collected rainwater). Therefore, given the scales of economy present in water treatment, it is important to quantify what benefit arises from implementing these technologies in larger clusters.

Consequently, there is a need for an holistic, integrated framework for the planning and design of cluster-scale water systems that incorporate IUWM principles. Such a framework needs to consider multiple sustainability objectives and evaluate these through modeling the interactions between the various sources and infrastructure components. In addition, it needs to consider a wide range of planning options, including different water sources and the level of decentralization. The framework should incorporate multiobjective optimization, to ensure efficient use of natural resources, which will become increasingly important as resources become more limited, and to ensure unbiased comparison is made between alternative systems. Although some systematic frameworks have been described in the literature for the planning of water systems that incorporate IUWM principles [for example, see *Sharma et al*., 2009], these frameworks neither consider the extent of decentralization nor do they incorporate formal optimization techniques.

In order to address the shortcomings outlined above, this paper has the following two objectives.

1. To develop a framework for the initial planning and design of the water supply systems for greenfield developments that incorporates:
   - 1.1. the selection of water sources,
   - 1.2. the mapping of water sources to end uses forming multiple configurations,
   - 1.3. the level of decentralization, and
   - 1.4. the evaluation of options in terms of sustainability criteria, and the identification of optimal solutions, through the use of formal multiobjective optimization approaches.
2. To apply this framework to a case study, demonstrating its use for:
   - 2.1. investigating the optimal trade-offs between water savings, cost, and GHG emissions for residential water systems of different spatial scales and with different sources of water,
   - 2.2. comparing how alternative water sources perform, when evaluating these objectives across the entire water system, and
   - 2.3. investigating the marginal benefits of differing levels of decentralization.

The remainder of this paper is organized as follows. Section 2 provides an overview of the case study. Section 3 describes the framework and shows how it was applied to the case study. The results and discussion arising from the case study are subsequently presented in section 4. In section 5, the benefits of the framework, as illustrated by examples from the case study, are discussed. Finally, conclusions are presented in section 6.

## 2. Overview of the Streaky Bay Case Study

To meet the objectives of this paper, a case study was developed that involved the planning and conceptual design of a realistic greenfield development located within a semiarid region of Australia. The case study was located at the outskirts of the remote rural township of Streaky Bay, South Australia (Figure 1). The region has a long-term average rainfall of 379.5 mm per annum with an average of 62 days per year experiencing rainfall exceeding 1 mm. Most of this rainfall occurs in winter, as shown in Figure 1.

The case study was based on an allotment layout of an actual greenfield development. This development consisted of 285 allotments, although for the purposes of this study, the maximum number of allotments was increased to 1140 to enable exploration of the impact of the level of decentralization, as shown in Figure 2.

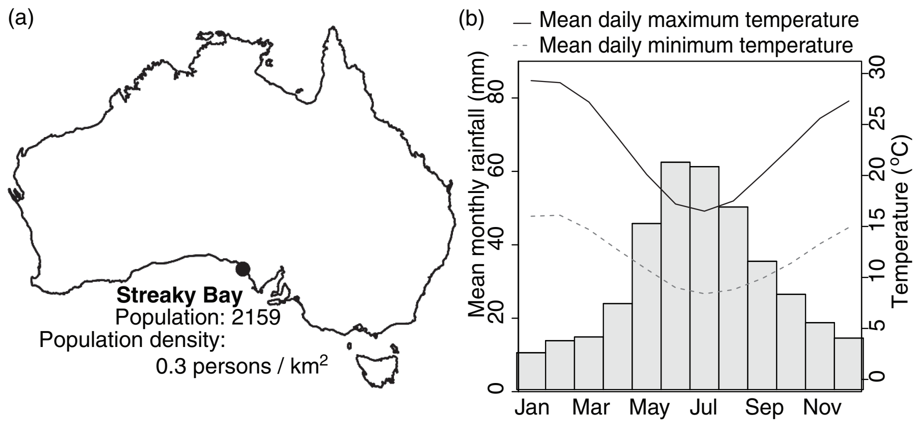


**Figure 1.** Location, population, and climate statistics for the Streaky Bay region.

## 3. Methodology

In the subsequent sections, details of the proposed framework (Figure 3) and how it was applied to the Streaky Bay case study are given.

### 3.1. Determining Performance Objectives for the Water System

First, it is appropriate to specify the objectives to be optimized during the planning and design process, as shown Figure 3a, which should be those that are most important for the case study under consideration. It should be noted that at this stage of the process, it is not necessary to formulate how these objectives will be quantified using criteria functions.

As discussed in the Introduction, multiple objectives should generally be considered, and would usually cover economic (e.g., incorporating costs and benefits criteria), environmental (e.g., incorporating

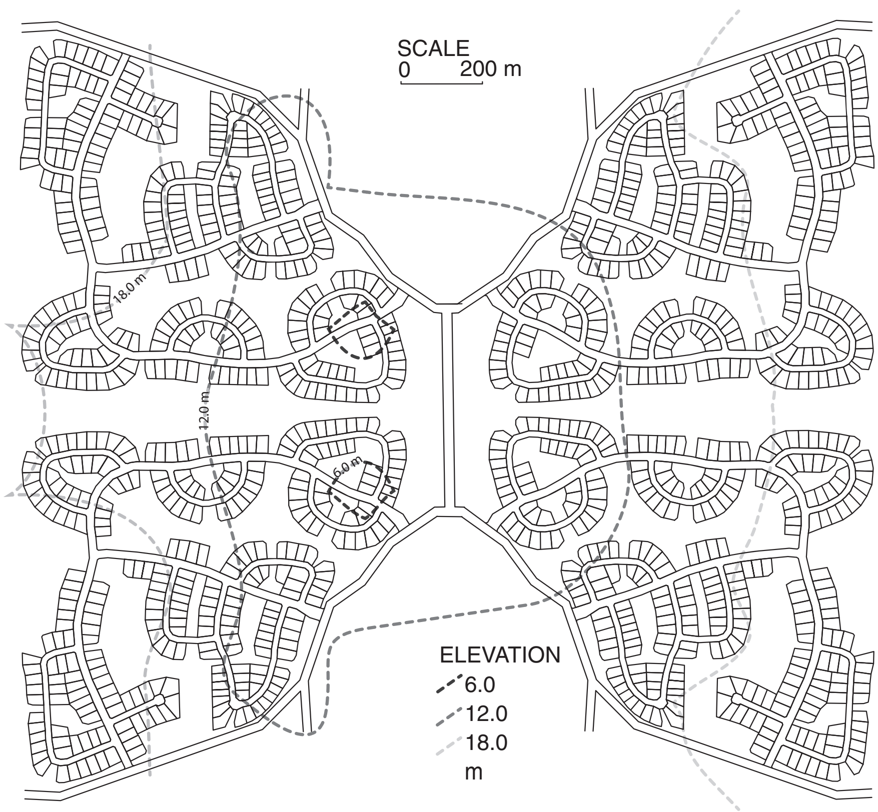


**Figure 2.** The greenfield development case study, showing the layout of the 1140 allotments and road network.

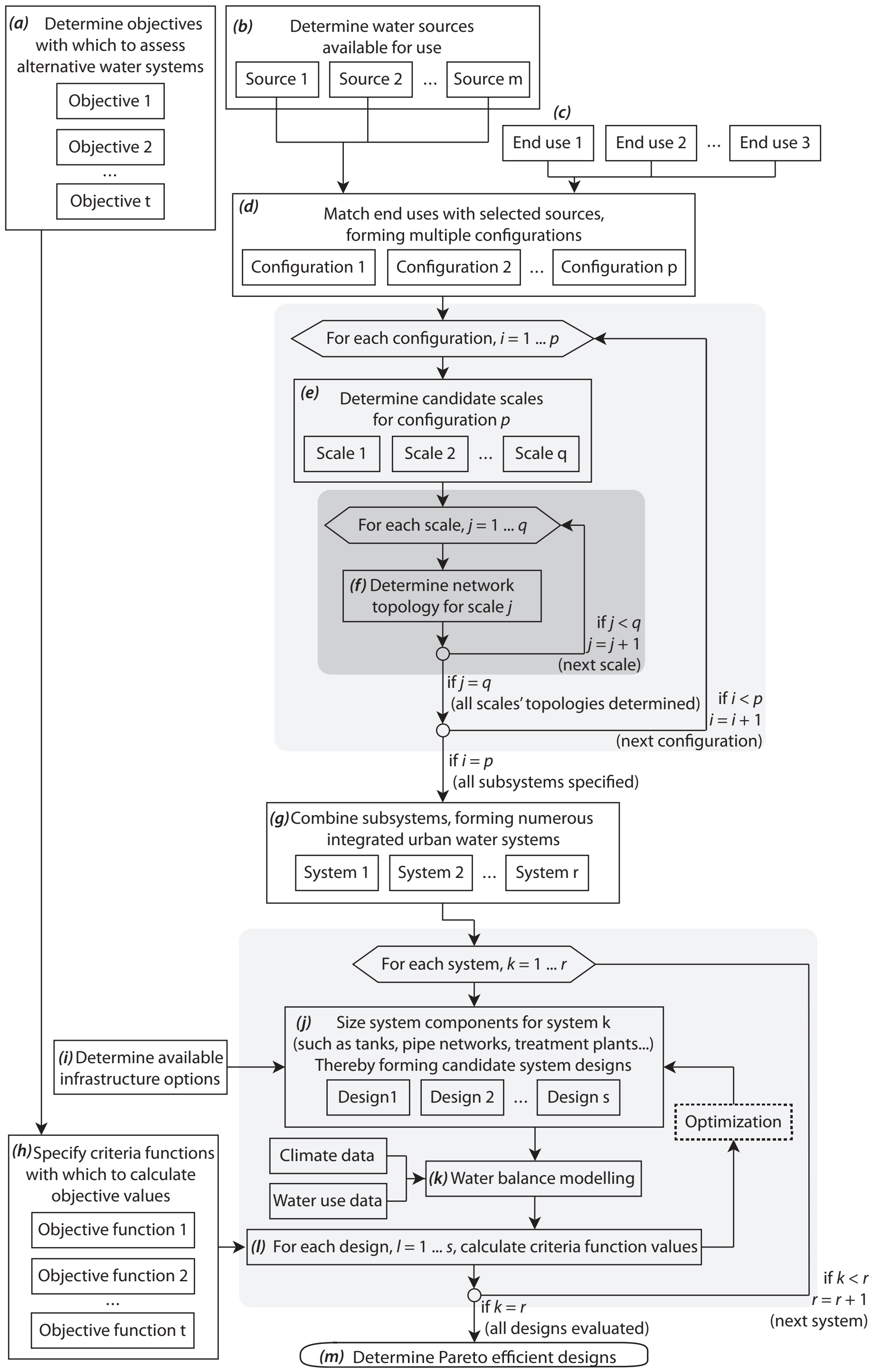


**Figure 3.** Proposed decision making framework for planning integrated urban water systems at the greenfield development scale.

hydrological change and pollution criteria), and social aspects (e.g., incorporating reliability and resilience in supply). Frameworks are available for selecting appropriate objectives, such as those introduced by *Hellstrom et al.* [2000], *Balkema et al.* [2002], and *Foxon et al.* [2002], which also give guidance on the quantification of the corresponding criteria. However, care should be taken not to increase the size of the search space unnecessarily by including superfluous objectives [*Efstratiadis and Koutsoyiannis*, 2010].

For the case study, the primary purpose of using an IUWM approach is for increasing water supply security, not for pollution control. Therefore, the selected objective functions include: (1) the minimization of the present value of costs, as in conventional economic analysis, (2) the minimization of supply from centralized potable supplies, given the level of water scarcity in the Streaky Bay region, and (3) the minimization of the present value of greenhouse gas emissions. The reduction of GHG emissions from water supply systems is becoming an increasingly important consideration in the literature, as water authorities strive to reduce their carbon footprint in an attempt to meet GHG emission reduction targets [*Stokes et al*., 2014b; *Paton et al*., 2014b]. In addition, the consideration of GHG emissions is particularly important when considering different sources of water, as the GHG emissions for treatment are potentially significantly different.

### 3.2. Identifying Alternative System Designs

The proposed approach to identifying alternative system designs consists of the following six steps:

#### 3.2.1. Identifying Water Sources

Suitable water sources in the region of interest need to be identified, as shown in Figure 3b. For example, in the case study, potential water sources included the Streaky Bay centralized potable water system, roof collected rainwater, graywater, and blackwater (toilet discharge only) reuse. Two possible graywater streams were considered, one stream that comprises water discharged from the kitchen, bathroom, and laundry (termed dark graywater herein), and one that only includes bathroom and laundry discharge. These two streams were considered, as there is some debate over the inclusion of kitchen discharge due to the concentration of organics that arise from food matter [*NRMMC*, 2006].

#### 3.2.2. Classifying End Uses of Water

End uses should be classified based on what each water sources could be used for, as shown in Figure 3c. Such end uses would typically include bathroom, laundry, kitchen, toilet, and garden demands (car washing was not included, as it is a minor use and water use legislation has banned the use of centralized supplies for this purpose in many Australian jurisdictions). End uses should be understood in a very general sense, as environmental flows [e.g., see *Ladson et al*., 1999], public uses (e.g., for street cleaning or for the irrigation of public spaces), and discharges to centralized wastewater systems may also be considered an end use. In the case study, end uses were separated into toilet, outdoor, laundry, tap, and bathroom uses. It was assumed that outdoor use only incorporated garden irrigation. Tap use included all kitchen and bathroom handbasin demands. Consequently, bathroom use included the shower and bath, but not the handbasin or toilet.

#### 3.2.3. Mapping of Sources to End Uses, Forming Multiple Candidate Subsystems

Water sources need to be mapped to end uses, forming configurations, as shown in Figure 3d. In regard to this mapping, three details should be noted: (i) multiple end uses and sources may be mapped together. When done so, the preference order in which end uses extract water from the multiple sources needs to be defined, which is often expressed mathematically using virtual costs [*Efstratiadis et al*., 2004]; (ii) the selection of sources and end uses is undertaken *with replacement*; the mapping does not need to be exclusive. As such, a source may be present in a number of configurations, and an end use may also be present in a number of configurations because the purpose of this phase is to form a number of candidate configurations that need to be analyzed in order to determine the best overall combination; and (iii) the mapping process must ensure that sufficient configurations exist so that all required functions can be performed by a complete system. For instance, a configuration that maps wastewater to conventional sewer networks may be required for collecting wastewater generated from end uses which are not reclaimed. In addition, centralized potable supplies may be needed for end uses that are not supplied through alternative streams. In the case study, 14 configurations were developed, as shown diagrammatically within the three columns labeled "configurations" in Figure 4. The configurations were based on typical uses of alternative water streams in Australia, and include a number of configurations that focus on roof-collected rainwater, graywater, and configurations that include different combinations of rainwater, graywater, and blackwater. From herein, each configuration is denoted by an abbreviation, as given in Figure 4.

#### 3.2.4. Exploring the Level of Decentralization

The level of decentralization at which each configuration could operate at needs to be determined, as shown in Figure 3e. This enables the development of a number of water subsystems. In this paper, a subsystem is defined as an organization of components that collects water from some identified sources and supplies it to end uses for a specified subcluster of allotments. In the final plan, one or more subsystems are implemented in combination, forming a *system*. Such a system is described as a "set of subsystems that fully provide water and wastewater services in a given area." This definition is an adaptation of that used in

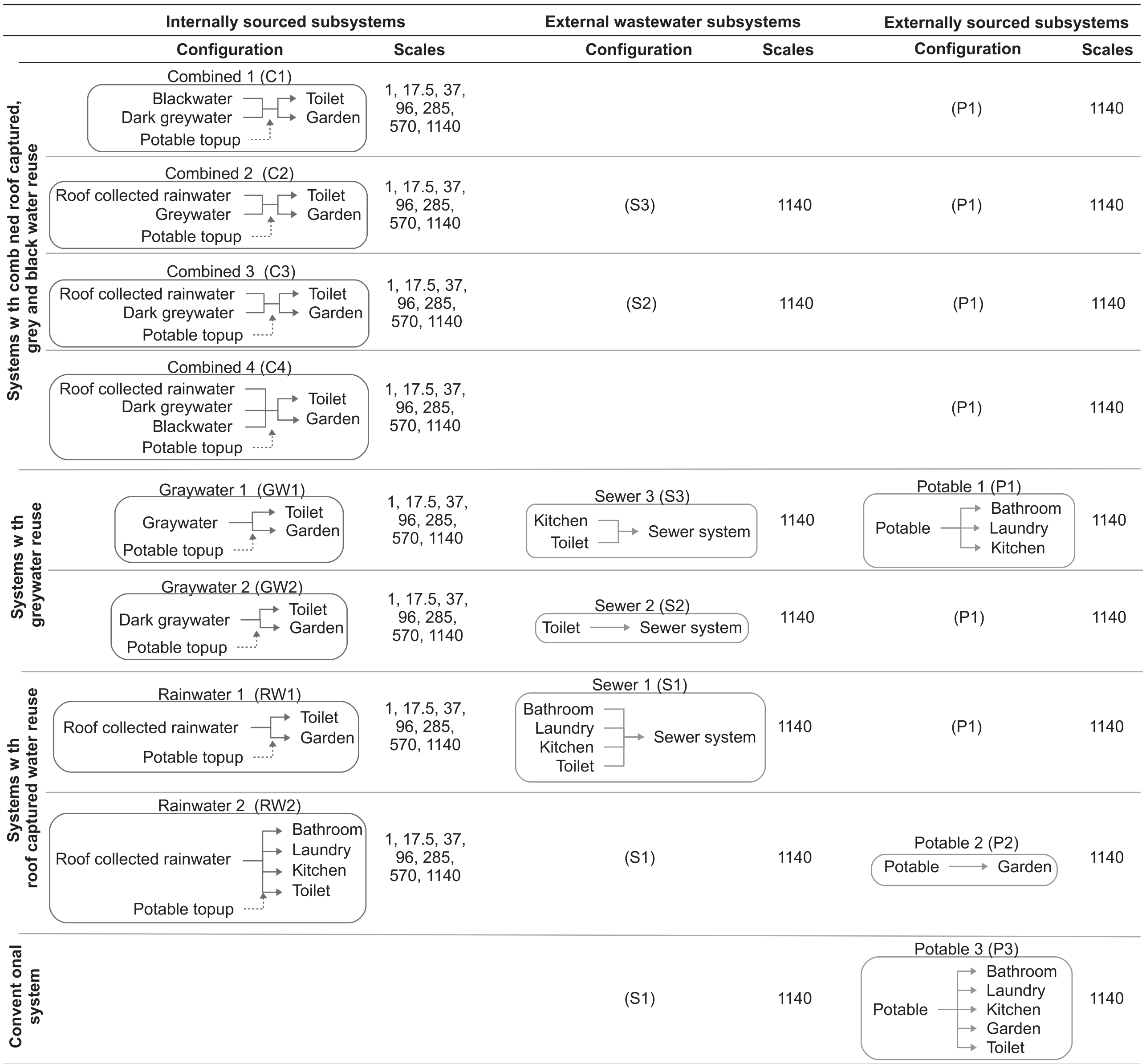


**Figure 4.** The complete urban water systems listed by row, with their constituent subsystems and scale of implementation as analyzed for the Streaky Bay case study.

*Sharma et al.* [2009]. In the case study, nine different decentralization levels (herein termed "spatial scales") were explored for each recycled water configuration. These were created by subdividing the development into progressively smaller clusters of allotments; at each spatial scale, the 1140 allotments were divided into an increasing number of subclusters. The subclusters that were formed through this process had a median of 1140, 570, 285, 96, 37, 17.5, and one allotment(s) at each spatial scale.

### 3.2.5. Specifying Subsystem Topology

The layout of each subsystem has to be specified, as shown in Figure 3f. This would normally be dictated by the topography of the land and the layout of allotments. For example, storage facilities for recycled streams would tend to be located at low elevations to allow gravity flow through collection systems. Sewer mining facilities would be located in the vicinity of large sewer mains, and the harvesting of stormwater would occur adjacent to major flow paths. In addition, water treatment facilities and water storage facilities would typically be placed next to each other. Collection and distribution networks would then connect each allotment to the treatment and storage facilities. Usually such networks would be acyclic, except where reliability issues are

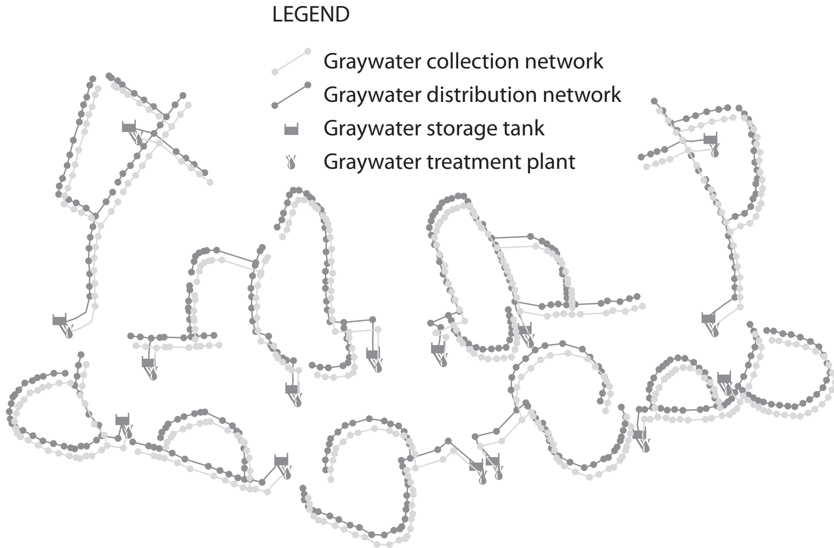


**Figure 5.** The layout of graywater recycling clusters for the GW1 configuration implemented at the 37 allotment scale. The potable and blackwater collection network is not shown. Due to symmetry in the case study, only half of the study area is shown

pertinent (such as in the potable distribution network, where loops prevent stagnation and increase system reliability when breakages occur). In this regard, gravity-driven collection networks were used in the case study to convey gray, black, and roof-collected rainwater from allotments to the most conveniently located sewer main or treatment plant, as appropriate. Treatment plants were located adjacent to tank storage at positions of low elevation with respect to the surrounding topography. These recycled water streams were then redistributed to the user through a distribution network, involving the use of pump stations. The position of pipes within the collection and distribution systems followed the road network, but with pipes laid in separate trenches to prevent cross contamination between each water stream.

### 3.2.6. Combining Subsystems, Forming Complete Systems for Analysis

A number of subsystems need to be grouped to form candidate water systems. Indeed, many combinations of subsystems will normally be possible, resulting in multiple feasible water systems. The process of forming such water systems from subsystems is shown in Figure 3g and would normally be mediated through the guidance of decision makers and/or designers to select the systems of most interest. Once a list of candidate water systems has been formed, their performance needs to be evaluated.

In the case study, eight configurations at seven different scales, in addition to a conventional system, were considered, as shown in Figure 4 (resulting in a total of 57 systems). Figure 4 does not map out each of these systems individually, but shows the configurations and scales at which each constituent subsystem would be implemented. For the sake of illustration, consider the systems that encompass the GW1 configuration, as shown in Figure 4(top row). These systems would encompass a number of GW1 subsystems implemented at one of the nine spatial scales and an S3 and P1 subsystem implemented across the entire development. For example, as shown in Figure 5, the system that forms when choosing to implement the GW1 configuration at the 37 allotment scale would include: (1) 30 separate water distribution networks for supplying graywater, (2) 30 separate water collection networks to convey graywater discharges to the 30 separate satellite water treatment, storage, and pumping facilities that service each cluster, (3) one water distribution network connecting all allotments to the centralized mains supply, and (4) one water collection network for discharging blackwater from each allotment to the centralized sewerage system. When designing systems and evaluating their performance (as described in the subsequent section), the collection, treatment, storage, and distribution infrastructure in every cluster was considered. In regard to the single allotment scale, each household had a small treatment plant and pump for graywater reuse.

## 3.3. Designing Systems and Evaluating Their Performance

At this stage of the framework, each water system, which consists of a specific combination of subsystems, needs to be designed, and its performance evaluated. This process involves five main facets, as shown in

**Table 1.** Itemization of Cost and GHG Emissions Components Evaluated for the Streaky Bay Study

| Component | Item |
|---|---|
| Centralized services | PV cost of charges and emissions associated with centralized potable supply |
| | PV cost of charges and emissions associated with centralized wastewater charges |
| Collection networks | Costs and emissions of trenching and refill operations |
| | Capital and laying costs, and emissions associated with embodied energy of pipes |
| | PV costs and emissions from network maintenance |
| Cluster treatment | Capital cost of treatment plants and emissions associated with construction |
| | PV costs and emissions from maintenance and operation of plants |
| Water storage | Capital cost of tanks and emissions associated with tank fabrication |
| Distribution networks | Costs and emissions of trenching and refill operations |
| | Capital and laying costs, and emissions associated with embodied energy of pipes |
| | PV costs and emissions for installation and running of pump stations |
| | PV costs and emissions from network maintenance |

Figures 3h–3l, being: Figure 3h criteria functions have to be specified; Figure 3i various infrastructure options need to be identified; Figure 3j infrastructure needs to be selected and sized, using optimization, forming alternative system *designs*; Figure 3k designs have to be analyzed in terms of the selected sustainability indicators using water balance simulations; and Figure 3l criteria functions need to be evaluated.

### 3.3.1. Specifying Criteria Functions

In the case study, items that were included in the evaluation of costs and GHG emissions of each system are presented in Table 1. For each system, the net economic and GHG emission objectives were calculated by summing the present values of these objectives across every item. Present value analysis (PVA) was chosen to take into account the time preferences of costs and greenhouse gas emissions [see *Wu et al*., 2010] over a planning horizon of 50 years.

For this study, a discount rate of 6% per annum was chosen for the economic objective. This is an intermediate value from among the range used by government agencies. For example, discount rates used among U.S., English, and Dutch government agencies range from 2 to 10% *per annum* [*Rambaud and Torrecillas*, 2005] and South Australia's water supply utility uses discount rates between 6 and 8% for their projects.

A discount rate of 2% was selected for discounting greenhouse gas emissions, which is within the range of suggested values. For example, while *van Kooten et al*. [1997] suggested that carbon should be discounted at the same rate as money, others argue that the discount rate for greenhouse gas emissions should be different than that for capital, while a zero discount rate is often used in practice [*Fearnside*, 1995, 2002].

Choosing the value of the discount rate, *i*, for economic costs and especially for greenhouse gas emissions is not without difficulties, and is significant as PVA is highly sensitive to the value of the discount rate. Many studies have addressed the choice of discount rate [*Simpson*, 2009; *Wu et al*., 2010; *Azar and Sterner*, 1996; *Conceição et al*., 2007; *Weitzman*, 2007; *Rambaud and Torrecillas*, 2005; *Guo et al*., 2006; *Wu et al*., 2010; *Cai et al*., 2002], and a discussion of the issues surrounding this is beyond the scope of the current paper.

### 3.3.2. Determining Available Infrastructure Options

The infrastructure options that were considered for the collection networks, treatment plants, storage facilities, and distribution networks are listed below with a description of how the costs and GHG emissions for these infrastructure items were calculated. The costs of infrastructure items were calculated considering both the capital and operation/maintenance aspects. Calculation of GHG emissions included two aspects: emissions associated with operation, and emissions associated with the embodied energy of infrastructure (capital emissions), for which GHG emissions are calculated using an emissions factor. In this study, the full fuel cycle emission factor for electricity users in South Australia was used [*Wu et al*., 2010].

#### 3.3.2.1. Pipe Options

A number of pipe materials are suitable for use in water supply and wastewater collection systems. Of these materials, only PVC pipes were considered, because they have the lowest capital GHG emissions (GHG emissions associated with the embodied energy of the infrastructure), moderate costs, and are available across a wide range of diameters. Models were developed for estimating the cost and embodied energy of piping (from which capital GHG emissions were calculated from through using a appropriate emissions factor), and are presented in the supporting information. The discrete pipe diameters considered ranged from

40 to 575 mm with the specific diameters, together with their costs and embodied energy presented in supporting information.

The cost of repairing pipe breakages was based on the model presented by *Kim and Mays* [1994], and the expected breakage rate (breaks/m/yr) was based on the model presented in *Su et al*. [1987]. Both models, described in detail within the supporting information, were updated for SI units and for contemporary Australian conditions. These models were chosen as they were recently applied to costing dual water distribution networks by *Kang and Lansey* [2012]. The annual embodied energy for repairs was calculated in a similar fashion, based on the breakage frequency in *Su et al*. [1987], and by assuming 5 m of pipe was replaced for every break, following the study of *Filion* [2007].

Trenching and backfilling also contribute to capital costs and energy requirements of distribution and collection networks. The cost for trenching was estimated at AU$50 per $m^3$ based on data from *Rawlinsons* [2007]. The GHG emissions resulting from the excavation of each cubic meter of soil were estimated at 43 kg $CO_2$-e; this value is derived in supporting information. Trenching volumes were based on minimum covers, as specified by manufacturer data.

#### 3.3.2.2. Pumping Options

Only variable speed pumping (VSP) options were considered because VSP allows for more flexible control over the distribution system; drive speeds can be altered to reduce pump power when flow rates and pressure requirements are low, thus saving energy and reducing costs [*Wu et al*., 2012b].

A generic model for calculating the capital costs and embodied energy was used, which was based on the hydraulic requirements for pumping and used data from *Wu et al*. [2010] and *Rawlinsons* [2007]. The justification of this approach, and details of the models developed are given in supporting information.

The ongoing operating costs and GHG emissions of distribution networks are mainly due to electricity consumption at the pumping station. A model for pump efficiency (which included the pump, motor, and variable speed drive efficiency) was used in calculating pump energy use, as detailed in supporting information.

Energy cost for pumping was calculated using the annual energy consumption and average electricity tariff, being AU$0.14 per kWh for South Australia, as reported in *Wu et al*. [2012b].

#### 3.3.2.3. Water Treatment Options

Complete packaged solutions were considered for water treatment. A generic model was developed to predict the capital costs, embodied energy, and the operational and maintenance costs of the treatment plant, in addition to the power consumption required to run the treatment processes, as presented in supporting information. These models were calibrated using data from *Holt and James* [2006] in addition to *NRMMC* [2006], *Treloar* [2000], and *Fagan et al*. [2010].

It was assumed that rainwater supply only required disinfection and not treatment. For this, cost and GHG emission models, as found in supporting information, for UV treatment were developed based on data from the *United States Environmental Protection Agency* [1999] and Davey's Steriflo product range [*Davey Water Products Pty. Ltd*., 2014].

#### 3.3.2.4. Water Storage Options

At the allotment level, water storage options consisted of above-ground water tanks fabricated from PVC-lined galvanized steel. For clustered systems, above-ground tanks fabricated from PVC-lined zinc coated steel were considered. For both individual allotment and cluster-scale tanks, capacity per allotment was used as a design variable rather than the actual capacity of the tank. Thirteen different capacities were considered ranging from 0.75 to 48 kL/allotment, as given in supporting information. The actual capacity of the storage placed in each clustered subsystem was subsequently calculated from the number of allotments in the cluster and the capacity per allotment. Cost and GHG emission models for tank storage are given in supporting information.

### 3.3.3. Cost and GHG Emissions for Centralized Services

Although not a technological option to be optimized, there are costs and GHG emissions associated with centralized services that need to be accounted for, if used. In this study, the wastewater collection and the centralized supply costs were calculated based on the tariff structures of SA Water, the water utility that provides these services at Streaky Bay. Supply charges for the centralized potable supply were tiered, and ranged from AU$2.42 per kL to AU$3.73 per kL. Sewerage connections were charged at an average of AU$204 per annum, irrespective of discharge volume. The net GHG emissions associated with water supply and wastewater collection were estimated at 1.8 kg $CO_2$-e/kL and 1.1 kg $CO_2$-e/kL, respectively, based on data from *Owens* [2011].

### 3.3.4. Water Balance Modelling

At the core of the evaluation process is a water balance model, shown in Figure 3k. A water balance model is needed to simulate the system, including the flows through the various subsystems and their components over an extended period, and may also be used for calculating the design flows for sizing these components.

In general, a water balance model should be developed so that its outputs are able to evaluate the criteria and constraint functions. Consequently, the functionality that is required in the model, in terms of what processes are modeled, is dependent on these criteria and constraints. For the case study, a water balance model was developed to estimate the amount of water supplied from the external centralized potable water source, and to estimate the design flows within the infrastructure to be sized. This model, similar to the urbanCycle model described in *Hardy* [2008] and *Hardy et al*. [2005], simulated the water cycle *sequentially* over a continuous 30 year period using discrete daily time steps (1 January 1981 to 31 December 2010) in order to evaluate the performance of each system over a variety of conditions. Inputs to the water cycle simulation included residential demand and wastewater discharge at each allotment, in addition to rainfall. The formulation of the model is described in more detail within the supporting information.

In terms of consumer water use patterns, water use from laundry, bathroom, toilet, and kitchen must be known in order to estimate the flows in reuse supply and waste streams that only partially serve household end uses. Unfortunately, consumer water use patterns are rarely available at this level of detail; the smallest scale at which water use is recorded is usually at the entire household level, and the temporal resolution of records is usually greater than monthly. Therefore, stochastic models that predict water use for each end use at an appropriate temporal scale will often be required. Such models have been identified by *Rathnayaka et al*. [2011] and include the models introduced by *Blokker et al*. [2010, 2011]; *Duncan and Mitchell* [2008]; *Thyer et al*. [2009]; and *Micevski et al*. [2009]. For the case study, a daily time step stochastic water use model, as formulated in supporting information, was developed to estimate residential water use based on water use data for South Australia. Data requirements for this model included rainfall and pan evaporation data, which were obtained from the Patched Point Data set (PPD) [*Jeffrey et al*., 2001].

The daily time step used for water balance modelling in the case study was too coarse for representing the hydraulic operation of the distribution and collection networks. Therefore, these networks were sized based on peaking factors applied to the peak daily flows calculated from the water balance, as described in the following sections.

### 3.3.5. Sizing Treatment, Storage, and Conveyance Infrastructure

The sizing of infrastructure involves the specification of design conditions, objectives, and constraints and subsequent use of models to evaluate several alternative combinations of product options and component sizes to determine the best performing system design. Because of this, the sizing of infrastructure is, in general, an optimization problem. However, due to the large number of infrastructure options normally available, selecting optimal choices for such infrastructure options are not trivial, and yet important for understanding the trade-offs in performance objectives in an unbiased way, and to ensure the most efficient use of resources is achieved.

In the proposed framework, the sizing of infrastructure is formulated as an optimization problem. The decision variables include the selection of infrastructure options and infrastructure design variables, which are both discrete and continuous. The constraints will generally be related to the technical performance of the system, including pressure and velocity constraints in conveyance networks, and water quality as delivered to end uses.

Solving this optimization problem is difficult. Not only is the solution space large, but also many of the constraints are nonlinear, especially in regard to the hydraulics in conveyance networks. Furthermore, difficulty arises due to the consideration of multiple, often competing, objectives, and there may be strong interaction between decision variables. Evolutionary algorithms (EAs), such as genetic algorithms (GAs), ant colony optimization (ACO), and differential evolution (DE), are suitable solution techniques for solving such difficult optimization problems. This is because they tend to be robust toward problems characterized by nonlinearity, multimodality, large decision spaces, and interactions between decision variables. In addition, EAs can be adapted for multiobjective optimization with relative ease. Multiobjective approaches find *multiple* (as opposed to a single) efficient (Pareto-optimal) designs that cannot be said to be better or worse than each other [*Efstratiadis and Koutsoyiannis*, 2010]. Furthermore, they have been used extensively in the water resource literature for the optimization of infrastructure components contained within the optimization problem described in this framework (except in this framework, such components may need to be optimized simultaneously, due to interactions between the collection, distribution, storage, and treatment components). For example, they have been used for designing water distribution systems [see, for example *Simpson et al*., 1994; *Eusuff and Lansey*, 2003; *Zecchin et al*., 2005, 2012; *Farmani et al*., 2005; *Khu and Keedwell*, 2005; *Wu et al*., 2010, 2013] and in their operation [see, for example *Broad et al*., 2010; *Kang and Lansey*, 2012], for determining the location and layout in addition to the diameters of pipes in collection systems [see, for example *Diogo et al*., 2000; *Diogo and Graveto*, 2006; *Afshar*, 2008, 2010], for specifying the capacity of storages [*Rozos et al*., 2010], for the selection of in-house water use devices [*Makropoulos et al*., 2008], and for designing treatment plant process trains [*Dinesh*, 2002].

The following paragraphs describe how water collection networks, treatment systems, storages, and distribution networks were sized in the case study.

#### 3.3.5.1. Sizing Water Collection Networks

In regard to sizing water collection networks, two design variables needed specifying, including (1) the diameter of each pipe segment, and (2) the depth below ground level at the endpoints of each pipe segment. The depth controls the slope of the pipe segment, and together with the diameter, affects the velocity and depth of flow.

Collection networks for roof collected rainwater conveyance were allowed to flow full, while systems conveying household discharges were allowed to flow a maximum of half full. The minimum and maximum velocity constraints were 0.6 and 2 $ms^{-1}$, respectively. These constraints were applied only during peak flow conditions, and checked by assuming steady state flow using the Colebrook-White friction formula. The Colebrook-White formula was chosen in accordance with Australian Standard 2200-2006, as it is an accurate basis for hydraulic design and has had sufficient experimental confirmation over a wide variety of conditions for gravity flow systems [*Council of Standards Australia*, 2006].

Design flows were calculated differently for household discharge and roof-collected rainwater. For systems that collected household discharge, design was based on peak flow calculated from a peak flow factor applied to the maximum daily flow. The maximum daily flow in a pipe segment was determined from the water balance simulation, as stated previously, and the peak flow factor chosen was 5.5 as used for sizing distribution networks. Peak flows for roof collected rainwater were based on 1-in-1 year rainfall intensities. This rainfall intensity was based on a design storm duration whereby all connecting roofs are fully contributing to the flow in each pipe segment to be sized. This methodology was in accordance with that specified in *Pilgrim* [1997]. Once the rainfall intensity was calculated, the design flows corresponding to these intensities were calculated using a roof runoff model discussed in supporting information. Design flows for collection networks that conveyed both roof-collected rainwater and household discharge were calculated as the sum of the peak flows for the two discharges.

Because the roof-collected rainwater collection networks were designed for 1-in-1 year flow events, surcharge during larger storms would be relatively frequent. Consequently, this surcharge was assumed to be conveyed in the regular stormwater network. This stormwater network was neither included in the water cycle simulation, nor sized, as it was external to the system boundary considered in this study. Stormwater networks were not included within the system boundary because none of the systems analyzed used stormwater as a supply. Because the stormwater catchment area in this study was large compared to the roof area, any surcharge flow was assumed to have little impact on the design, and hence on the embodied energy (which gives rise to GHG emissions) and cost performance of the stormwater subsystem. Therefore, these costs and GHG emissions have been assumed to be uniform across all compared systems.

#### 3.3.5.2. Sizing Treatment Systems

For the purposes of the case study, rainwater was assumed to only require disinfection through chlorination or UV treatment. However, the other subsystems required biological and/or membrane treatment, to achieve *class A* water quality standards. In order to specify treatment efficiency, the required pathogen log

**Table 2.** Required Log Reduction of Treatment by Configuration

| System Type | GW1 | GW2 | C1 | C2 | C3 | C4 | RW1 | RW2 |
|---|---|---|---|---|---|---|---|---|
| Log reduction | 4.0 | 4.1 | 6.0 | 3.8 | 3.9 | 5.9 | D | D |

[a]D: Only disinfection (using UV) was considered for these subsystems.

reduction was estimated for each treatment plant, based on: (1) the water quality characteristics of the source water streams; (2) the relative mix of each water source, as calculated from water balance results; and (3) manufacturer specifications reported in *Holt and James* [2006]. The required log reductions for each water source are given in Table 2.

#### 3.3.5.3. Sizing Storage Infrastructure and Distribution Systems

Optimization was used for sizing storage tank capacities and pipe diameters in distribution networks. In regard to objectives, the cost and GHG emissions associated with storage and distribution of water were minimized, as was the demand from the Streaky Bay potable water system. The optimization problem included hydraulic constraints, including minimum and maximum pressure heads of 15 m and 70 m, respectively, and a maximum velocity of 2.0 $ms^{-1}$. EPANET2 was used for solving the hydraulic heads and velocities needed for evaluating these constraints. In addition, the minimum pipe diameter in the potable distribution networks was specified at 100 mm, to cater for fire flows. Key characteristics of the problem formulation are given in Table 3.

Calculating the hydraulic heads and flow velocities within a distribution network is dependent on the value of nodal demands. These were estimated using the water use model and peaking factors. A peak demand factor of 5.5 was used, in accordance with the *Water Services Association of Australia* [2002] recommendation for use in dual reticulation systems.

To minimize both costs and GHG emissions, a multiobjective genetic algorithm (GA) was developed in the Paradis-EO software framework [*Cahon et al*., 2004] that utilized *Deb et al*.'s [2002] Non-Dominated Sorting Algorithm, creep mutation, and a two-point crossover. The genetic algorithm contained a number of parameters that affected the way it searched for optimal solutions, of which optimal values were determined by means of sensitivity analysis. From this analysis, a population size of 200, a probability of mutation per chromosome of 0.2, a downward creep probability of 0.6, and a crossover probability of 0.9 were chosen for all optimization runs, which were terminated at 300 generations. For each system, the GA was run a minimum of 30 times with different random number seeds, with results collated for determining one Pareto-optimal set.

#### 3.3.5.4. Sizing Pump Stations

Pump stations were sized based on peak pump power. A generic approach to calculating this value was adopted, similar to that described in *Wu et al*. [2012b], as formulated in supporting information. This sizing approach was embedded within the optimization problem, as just described.

### 3.3.6. Evaluating Criteria Functions

As shown in Figure 3l, the criteria functions were calculated and the constraints checked based on the sizes and outputs from the water balance model obtained in previous steps.

## 3.4. Identifying the Pareto-Optimal Solutions

At this point in the framework, a number of candidate systems have been elucidated and potential designs identified for each using formal optimization techniques, resulting in the identification of Pareto surfaces for each of the systems analyzed. Now, those designs that remain Pareto-optimal when compared with all other designs from the other systems are identified to form a single Pareto surface representing the optimal trade-offs for different water system designs, as shown in Figure 3m. The dimension of the Pareto surface is equal to the number of criteria functions used as objectives,

**Table 3.** Key Characteristics of the Optimization Problem Formulation

| Number of Decision Variables | |
|---|---|
| For cluster tank capacity (capacity per allotment) | 1 |
| For pipe diameters in potable distribution systems | 51 |
| For pipe diameters in nonpotable distribution networks | 48–75 (depending on scale) |
| Number of constraints | |
| For minimum and maximum pressure (property boundary of each allotment for each network) | 914–1827 (depending on scale) |
| For maximum velocity (in each EPANET pipe link for each network) | 935–1869 (depending on scale) |

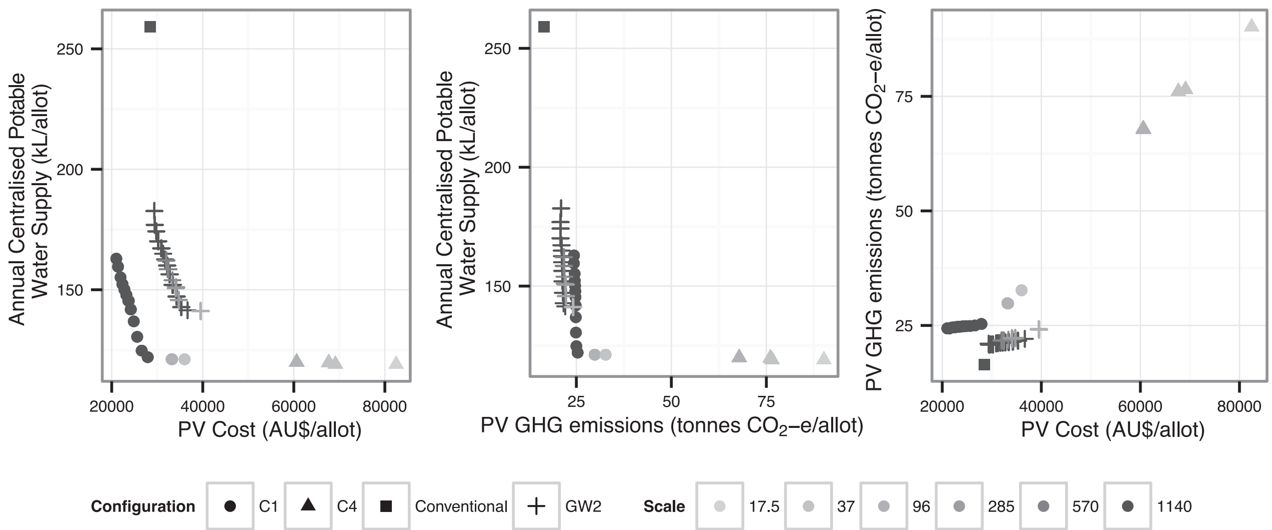


**Figure 6.** Pareto-efficient solutions plotted with respect to cost, GHG emissions, and water supply objectives across all configurations and scales considered for the case study.

and when these exceed three, spider plots and parallel axis (value path) plots may be helpful for the visualization of the trade-offs between objectives that these surfaces describe. The Pareto surface is the outcome of this framework, which is subsequently used by decision makers in coming to a preferred design for implementation.

## 4. Results and Discussion

This section presents the results of the case study, and discusses these in three parts, in line with the objectives of the paper, as given in the Introduction. First, the nature of the Pareto-efficient designs is discussed to understand the trade-offs between the performance objectives. Second, the performance of different source-end use configurations is compared to understand how alternative water sources perform in regard to water savings, costs, and GHG emissions. Third, the change in costs and greenhouse gas emissions with levels of decentralization is shown in order to determine the marginal benefits at each level of decentralization.

### 4.1. The Nature of the Pareto Surface

The objective values for the Pareto-efficient water system designs are plotted in Figure 6 (a rotating 3-D version may be accessed through supporting information), about which a number of generalizations can be made: (1) the Pareto surface is rather narrow, and is formed by two roughly parallel curves; (2) only some of the source-end use configurations are represented; (3) the conventional system is included in the Pareto set, because it has the lowest GHG emissions, (4) the Pareto set contains a disproportionate number of solutions from larger scale systems; (5) a trade-off is present between costs and GHG emissions in regard to distribution systems; and in contrast to the previous item, (6) net GHG emissions and costs of a system are more generally correlated. These outcomes are now discussed in more detail.

As shown in Figure 6, the Pareto surface is narrow as there was little trade-off between costs and GHG emissions. Consequently, optimal water systems that had higher costs are also likely to have a corresponding increase in GHG emissions. When one considers the regions in the objective space wherein a broadened Pareto surface could sit, these regions are characterized by solutions that would either (1) have lower external water supply and lower costs than the solutions on the Pareto surface, but with higher GHG emissions; or (2) have higher external water supply and costs, but with lower GHG emissions. However, it was not possible to select infrastructure options that gave rise to these types of solutions, because of the correlation that *also* existed between costs and GHG emissions for the construction and operation of each infrastructure item (as shown in the cost and GHG emissions models within supporting information).

In regard to the clustering, this is formed by the discrete nature of the decision variables. For example, plotted in Figure 7 are the cost and GHG emissions for the Pareto-efficient solutions at the GW2–96 system

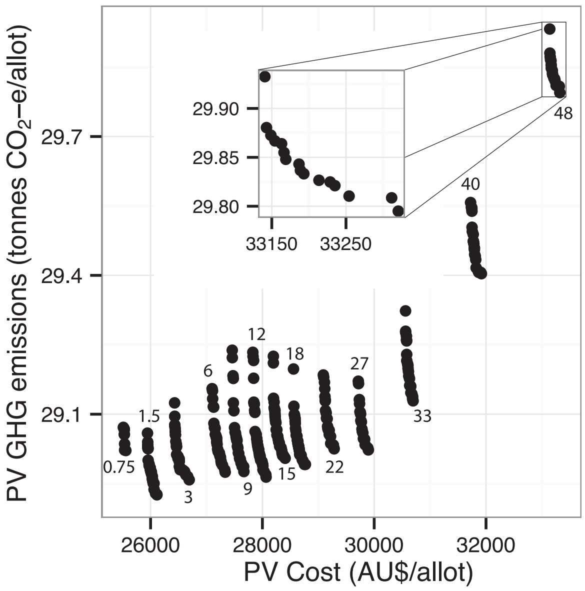


**Figure 7.** Pareto-efficient solutions for the GW2–96 system, plotted with respect to cost and GHG emission objectives. Each cluster of points is annotated with the tank capacity per allotment (in kL/allotment) corresponding with that cluster.

level. Each cluster in this plot corresponds to a particular tank capacity. Therefore, tank capacity is a salient decision variable, which the GA is using to explore the trade-offs between water supplied from the potable networks and the cost and GHG objectives.

In regard to the scatter within each cluster, this is formed through the selection of pipes with different diameters in the distribution network. Small diameter pipes reduce capital costs, at the expense of greater GHG emissions, as pumps need to provide greater pressure, forming a trade-off between cost and GHG emissions. However, it should be noted that the trade-offs between cost and GHG emission objectives were both less than 1% of their values, and are virtually unobservable in Figure 6c, where GHG emissions and costs were seen to be correlated. Therefore, the choice of an optimal set of pipe diameters is not particularly important for decision making in this case study, as this choice does not produce much trade-off between performance indicator values.

Other researchers have predicted larger trade-offs between cost and GHG emission objectives in distribution systems. For example, *Wu et al*. [2010] found that the PV of greenhouse gas emissions could be reduced by approximately 15–35% percent at an increase in PV of costs of approximately 35–62%. However, these results were obtained for a water transmission system (WTS), rather than a cluster-scaled residential distribution system (RDS). The flows in WTSs tend to be less variable than those in RDSs, and therefore WTSs tend to run closer to system capacity for a greater proportion of the time. Conversely, the RDS simulated for this case study is operating well below capacity for most of the time—at each allotment, water demand is zero for the majority of the time, and garden watering, which uses the highest flow rates, occurs infrequently. Consequently, for the RDSs simulated in the Streaky Bay case study, the heads produced by, and flow rates through variable speed pumps at peak conditions are substantially larger than at average conditions. For example, in this study, 30 year time averaged flows within the distribution networks were orders of magnitude lower than at peak hourly conditions. Therefore, the costs and GHG emissions for pumping contributed proportionally less to the total costs and GHG emissions in small clustered RDSs, and therefore, the trade-off between costs and GHG emissions was less prominent.

As just mentioned, the 30 year time-averaged flows within the distribution networks were orders of magnitude lower than at peak hourly conditions. While such variability in flow is not common in conventional water distribution networks, it is not unusual in small-scale networks. For instance, when considering a single allotment, water use occurs, in general, for short periods of time (such as using a tap, flushing a toilet, taking a shower, or watering the garden), distributed throughout the day. Therefore, for most of the time, there is no water supply to a household, and this causes the large variability in supply flow. However, as more households are connected to the network, this variability in flow becomes smaller. Consequently, this study observed that the levels of decentralization affected the variability in flow within distribution networks.

In regard to the two curves that make up the Pareto surface (Figure 6), one is associated with solutions from the C1 configuration, and the other with solutions from the GW2 configuration. In addition, both curves had a disproportionate number of solutions at the 1140 allotment scale. Explaining these two observations is better left to the following subsections, the first of which provides a comparison of different source-end use configurations. This comparison will help explain why the C1 and GW2 configurations were most favorable. The last subsection addresses the marginal benefits of the levels of decentralization on the

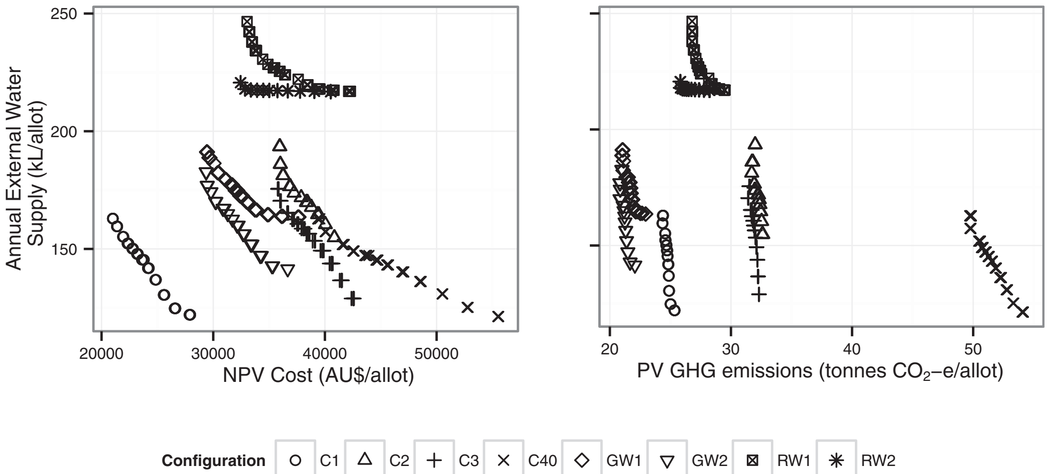


**Figure 8.** Pareto-efficient solutions for each configuration at the 1140 allotment scale.

performance of water systems, and therefore will show why the larger spatial scale systems tended to perform better.

### 4.2. Comparing the Performance of Alternative Water Sources

Figure 8 shows the Pareto sets for each of the eight source-end use configurations at the 1140 allotment scale. It was chosen to show the 1140 allotment scale when comparing configurations, as solutions from this scale were prevalent within the Pareto surface. However, similar trends to those discussed below were also evident in the smaller-scaled systems, as may be observed in supporting information. The relationship between these Pareto sets is now discussed, first in relation to water savings, and then in relation to costs and GHG emissions.

#### 4.2.1. Performance in Regard to Water Savings

Systems that incorporated rainwater were shown to least reduce water supply from the centralized source. On the other hand, all other source-end use configurations tended to reduce demand by similar amounts. This is because RW configuration tanks were empty for a greater amount of time during summer than the equivalent C or GW configuration tank, as less rainwater falls on a roof per year at Streaky Bay (41.5 kL/annum/allotment fell, as calculated by the average roof area) compared with the yield from household discharges, such as graywater (118.7 kL/annum/allotment). In addition, rainwater has strong seasonal dynamics at Streaky Bay, as shown in Figure 1. In winter, rainwater supply is highest but demand lowest, therefore rainwater is often lost through tank overflow. However, demand is highest but supply lowest in summer, increasing the likelihood of shortfall. Consequently, rainwater tanks empty during summer, and stay empty until the next winter rains. The efficiency of supply is also dependent on the temporal separation between each supply and demand event. The period between rainfall events is generally larger than that between wastewater discharge events. However, demand for water occurs daily. Therefore, larger storages are needed for rainwater systems, so that sufficient water from storage is available between rainfall events. Table 4 shows the amount of water available for each configuration, and how much of this water was supplied to allotments at the 1140 allotment scale. From this table, it is evident that systems that incorporated rainwater were less efficient at supplying water when small tank capacities were chosen (capacities smaller than 1.5 kL/allotment), and this is because of the temporal separation between supply and demand events.

These results are in agreement with those obtained in other studies. In a study that compared rainwater and graywater at the allotment scale for a rural Western Australian township (Cranbrook), graywater was able to supply 7% more water than rainwater [*Zhang et al.*, 2010]. While rainfall in both Streaky Bay and Cranbrook are winter-dominated, average rainfall in Streaky Bay was approximately half that in Cranbrook (annual rainfall of 739.4 mm). Therefore, rainwater use at Streaky Bay was able to supply a smaller percentage of water requirements than at Cranbrook (16.4% rather than 25.1%). As expected, the performance of rainwater was clearly sensitive to the climatic regime [*Rozos et al.*, 2010].

  

**Table 4.** The Net Volume of Water Available From Unconventional Sources for Each Configuration, and the Percentage Utilization Across all Tank Sizes and Configurations at the 1140 Allotment Scale

| | Configuration | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | C1 | C2 | C3 | C4 | GW1 | GW2 | RW1 | RW2 |
| Source availability (kL/annum/allotment) | 149 | 137 | 160 | 190 | 95.3 | 119 | 41.5 | 41.5 |
| Tank capacity (kL/allotment) | Percentage supplied to end uses | | | | | | | |
| 0.375 | 41% | 37% | 32% | 28% | 59% | 50% | 30% | 37% |
| 0.75 | 56% | 48% | 45% | 40% | 71% | 64% | 41% | 56% |
| 1.5 | 64% | 53% | 52% | 50% | 74% | 69% | 51% | 77% |
| 3 | 67% | 57% | 55% | 53% | 76% | 72% | 60% | 93% |
| 6 | 70% | 60% | 59% | 56% | 81% | 75% | 69% | 99% |
| 9 | 71% | 62% | 60% | 58% | 84% | 77% | 74% | 100% |
| 12 | 73% | 64% | 61% | 59% | 86% | 79% | 78% | 100% |
| 15 | 74% | 65% | 63% | 60% | 89% | 81% | 81% | 100% |
| 18 | 76% | 67% | 64% | 61% | 91% | 83% | 85% | 100% |
| 22 | 78% | 69% | 66% | 62% | 94% | 87% | 89% | 100% |
| 27 | 82% | 72% | 69% | 64% | 97% | 90% | 95% | 100% |
| 33 | 86% | 76% | 72% | 67% | 99% | 94% | 99% | 100% |
| 40 | 90% | 81% | 76% | 70% | 100% | 98% | 100% | 100% |
| 48 | 92% | 87% | 81% | 72% | 100% | 99% | 100% | 100% |

### 4.2.2. Performance With Regard to Costs

As shown in Table 1, the costs and GHG emissions for water systems arise from the fabrication, construction, and operation of many infrastructure components. Costs, broken down into these respective components (as specified in Table 1), are plotted in Figure 9 for all systems that were optimal at the 1140 allotment scale. In the following paragraphs, the costs of water systems are discussed in regard to (1) the cost of centralized services charges and water treatment, (2) the effect of storage capacity on costs, and (3) the costs of distribution networks.

#### 4.2.2.1. Centralized Services Charges and Water Treatment Costs

From the ordering of the stacked bars in Figure 9, it is evident that the largest costs are charges for (1) centralized water supply and sewerage and (2) local water treatment. Therefore, a reduction in the amount of water supplied from the centralized potable network or the amount of water sourced from within the development would reduce costs the most; yet these two reductions are in competition with each other, as a reduction in the amount of water sourced locally implies a greater reliance on the centralized potable supplies. Consequently, the configuration that reduced overall costs the most was the C1 configuration. Only the C4 configuration reduced centralized potable demand more than the C1 configuration, therefore centralized supply charges were relatively low. While the C1 system did include the low quality blackwater source, rainwater was not included, reducing the volume of water requiring treatment. Therefore, local treatment costs were modest, being lower than those required for the C2, C3, and C4 configurations. In addition, because the C1 configuration recycled all household discharges, connections to the centralized

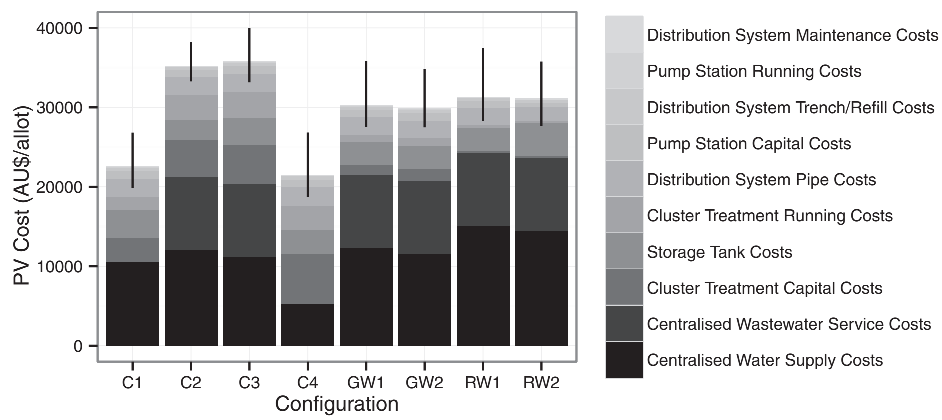


**Figure 9.** Average costs across all 1140 allotment-scale systems, broken down into subcomponents for each configuration. Lines indicate the range of net costs across optimal designs for each system.

  

wastewater system were assumed to be unnecessary, which further reduced costs and GHG emissions. By comparison, the RW1 and RW2 configurations, which had the lowest local treatment costs due to high source water quality, were unable to supply as much water as the other configurations, which increased the cost of centralized potable supply, leading to higher overall costs than for the C1 configuration as well. Because the C1 configuration had the least overall costs, it was prevalent among solutions on the Pareto surface (Figure 6).

#### 4.2.2.2. Effect of Storage on Costs

As mentioned earlier, the cost of a system was strongly affected by the choice of tank capacity. In addition to the cost of the tank itself, the chosen capacity also affected the cost of external water supply, the cost of centralized wastewater treatment, and the cost of distribution. This is because tank capacity has an influence on the dynamics of supply and demand, and therefore affects the relative demand from external and internal water sources, and consequently the flows in treatment and distribution systems. Therefore, costs were very sensitive to storage capacity in the system, causing up to AU$1328/allotment difference between the largest and smallest capacity tanks in distribution network costs, AU$2532/allotment in centralized supply charges and AU$8771/allotment difference in storage costs at the 1140 allotment scale.

#### 4.2.2.3. Distribution Costs

For most systems analyzed, two distribution networks were required, one for the centralized water source, and one for the alternative water source. The cost of distribution networks conveying the *centralized potable water supply* was nearly the same across all configurations, despite design flows being different. This is because the constraints regarding minimum pipe diameters for fire flows were generally binding. In contrast, the cost of distribution networks conveying locally sourced supplies was sensitive to design flow values. Distribution networks supplying graywater, blackwater, and rainwater sources did not supply fire flows, therefore, the 100 mm minimum diameter constraint did not apply and pipes at diameters down to 40 mm could be used. This allowed the pressure constraints, which are related to design flows, to be binding. Design demand for these alternative water streams were related to how prone local water storages were to emptying during summer. Peak demand for water occurs during the hottest weeks of summer; therefore, if storages were empty, these demands were placed on the centralized potable sources, rather than local sources. Consequently, rainwater configurations had the lowest design demands, as rainwater was least able to supply demand during mid summer. Therefore, rainwater distribution systems had the greatest proportion of smaller diameter pipes, and the lowest proportion of larger diameter pipes, resulting in the lowest capital costs for distribution.

### 4.2.3. Performance With Regard to GHG Emissions

The breakdown of GHG emissions is plotted in Figure 10. Most of the generalizations made for costs are also applicable to GHG emissions, as GHG emissions and costs were highly correlated. In this regard, (1) centralized services and treatment aspects dominated the overall GHG emissions, (2) storage tank capacity influenced GHG emissions relating to distribution in addition to GHG emissions associated with storage, and (3) distribution aspects contributed relatively little to overall GHG emissions.

In contrast to the trends observed in the cost data, GHG emissions for treating locally sourced water streams were higher than GHG emissions for external wastewater treatment. By implication, systems that reduced the intensiveness of local treatment and deferred more treatment to centralized systems were favored. Therefore, the GW1 and GW2 configurations had lower GHG emissions than the C1, C2, C3, and C4 configurations (the C1, C2, C3, and C4 configurations included lower-quality blackwater sources and larger treatment volumes from the inclusion of rainwater). The RW1 and RW2 configurations had the lowest GHG emissions for treatment, but were uncompetitive overall, due to the large GHG emissions associated with high demand from the centralized water supply. Overall, the GW2 system had the lowest GHG emissions: when compared with the GW1 system, the additional savings in GHG emissions associated with lower centralized potable demand were greater than the additional GHG emissions for treatment associated with the reuse of kitchen wastewater. Because the GW2 configuration had the least overall GHG emission requirements, it was prevalent within the Pareto surface (Figure 6).

## 4.3. Level of Decentralization and Costs

Figure 11 presents the breakdown of costs for Pareto-efficient GW2 designs classed according to the seven levels of decentralization considered in this study. A similar plot for GHG emissions may be found in

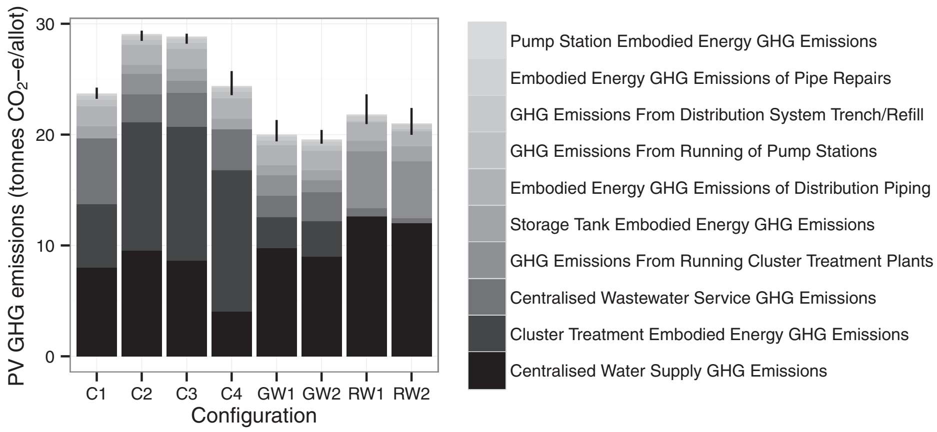


**Figure 10.** Average GHG emissions across all 1140 allotment-scale systems, broken down into subcomponents for each configuration. Lines indicate the range of net emissions across optimal designs for each system.

supporting information. Economies of scale in costs were evident for wastewater treatment and pump stations. Diseconomies of scale in costs were generally noticeable in constructing and maintaining distribution systems. Centralized charges were invariant with scale. These same economies and diseconomies of scale were also found for GHG emissions.

The economies of scale present in treatment dominated economies and diseconomies of scale present in the other infrastructure components. This dominance is caused by two factors. First, as mentioned earlier, treatment aspects are some of the largest contributors to the total costs and GHG emissions of a system. In addition, the trade-off between scale and costs are strong for treatment aspects. For example, GHG emissions per allotment for local treatment plants at the 17.5 allotment scale were approximately triple those at the 1140 allotment scale.

Because the economies of scale in treatment dominate the overall cost and GHG emissions, larger scale systems were favored for the GW2 configuration. Similar trends to those presented here for the GW2 configuration were also observed for the other configurations, as can be seen in supporting information. In this regard, the 1140 allotment scale was optimum for all configurations except the RW1, RW2, and C4 configuration. For the RW1 and RW2 configuration, the 96 allotment scale was optimum, and for the C4 configuration, the 570 allotment scale was optimum. This is because the diseconomy of scale in collection networks that conveyed roof-collected runoff was more pronounced in these systems, due to the large peak flows that occur during rainfall events. In addition, the costs and GHG emissions for treatment were proportionally smaller for the RW1 and RW2 configurations, making the economies of scale for treatment less significant.

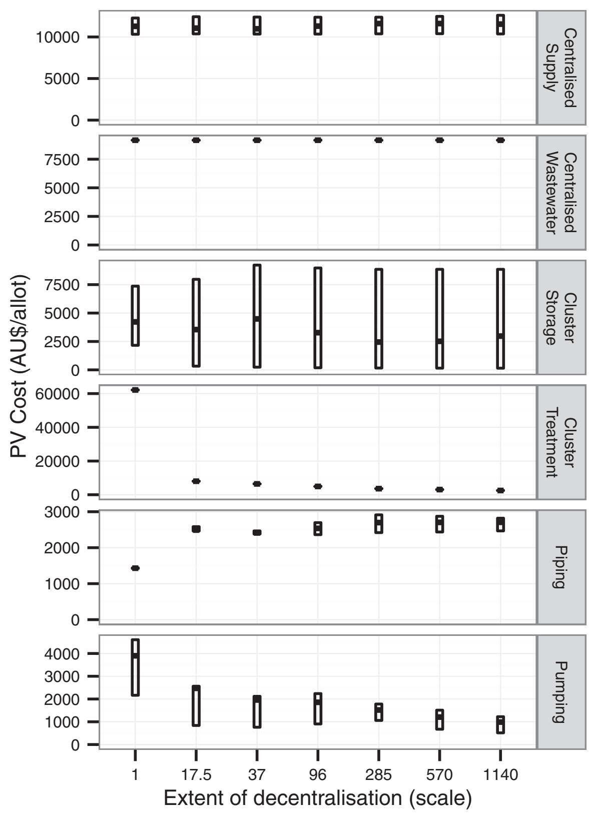


**Figure 11.** Breakdown of costs into subcomponents for the GW2 configuration across all scales analyzed. Crossbars indicate range and mean in costs for each component. Piping includes pipe capital, trenching and refilling and maintenance costs for the distribution and collection networks. Pumping includes cluster pump station capital and running costs for the graywater network. Cluster Treatment includes capital and running costs of the graywater treatment plants.

Figures showing the optimal scale for each configuration can be found in supporting information.

In order to draw out the pertinent information to address the objectives of this paper, the discussion so far has focussed on comparing different configurations at the 1140 scale, in addition to comparing the performance of the GW2 configuration across all scales. This represents a small subsection of all the systems considered as part of this study. Plots, similar to those presented here, but with data from all other systems explored, are available in supporting information.

## 5. Benefits of the Proposed Framework

There are a number of benefits of the proposed framework, which are discussed below and illustrated with examples from the case study.

### 5.1. Managing Increased Complexities Associated With IUWM

The framework is beneficial because it is able to manage the complexities inherent when planning and designing greenfield water systems with IUWM principles. However, this framework, in managing these complexities, does not simplify the problem by overly reducing the size of the decision space. Rather, the framework encourages broadening the scope of options relating to water sources, infrastructure technologies, and scales.

This is illustrated in the case study through the consideration of 57 systems for analysis, for which optimization yielded a large number of Pareto-efficient designs. In particular, the framework encouraged the consideration of blackwater, which has not been considered in greenfield developments to the extent that graywater and rainwater has. This was shown to be beneficial, as the case study results indicated that the inclusion of blackwater within the recycled water stream had significant benefit. With the inclusion of blackwater, only a single collection network was required, and no centralized wastewater charges were present. In addition, the inclusion of blackwater enabled greater water recycling and therefore reduced demand from centralized supplies.

### 5.2. Focusing on Local Conditions

As costs, energy values, and physical and hydrological factors will vary with location, there are no general answers in terms of what is the best configuration or scale of integrated urban water systems. A major advantage of the framework is its focus on local conditions; that is, it considers the climatic regime, source availability, the end uses of water and the technical feasibility at each site to which it is applied. Therefore, water systems developed by applying the framework are differentiated from standard solutions that do not consider local circumstances, resulting in greater resource use efficiency.

This is illustrated in the framework with respect to the performance of roof collected rainwater. In South Australia, 86% of households outside of the capital, Adelaide, have a rainwater tank. Therefore, rainwater is a standard solution to water supply in this state. However, the case study suggested that roof-collected rainwater did not perform well when compared with other options when the local conditions at Streaky Bay were taken into account. This is because roof-collected rainwater was neither cheaper, nor emitted fewer GHGs per unit volume than recycling household wastewater streams. Roof-collected rainwater was only able to reduce demand by 16.4%, compared with a conventional network, due to the seasonality of rainfall and household demand at Streaky Bay. In other regions, where rainfall is more uniform throughout the year, roof-collected rainwater may be more competitive.

The consideration of local conditions will become increasingly important in the context of resource limitation, increasing urban pollution, and heightened desire for environmental amenity. Consequently, decision makers are increasingly required to understand the consequences of their choices across a broader range of sustainability criteria. Two types of information will become critical for this. First, understanding the trade-offs between sustainability criteria across candidate water systems, and second the interactions between choices of water sources, source-end use mappings and infrastructure in terms of sustainability criteria. As illustrated above, the framework ensures these two types of information are identified.

### 5.3. Explicit Consideration of the Level of Decentralization

The framework also specifies that different levels of decentralization should be explored, as the performance of water systems may be sensitive to the level of decentralization at which they operate.

For the case study, results indicate that economies of scale in treatment tend to outweigh diseconomies of scale in pipe networks. This suggests that, for most water sources, cluster-scaled systems for which treatment plants serve alternative water streams to entire rural townships or suburbs are favorable with regard to costs and GHG emissions. However, roof-collected rainwater systems had the lowest costs at the 570 allotment scale, and therefore communal rainwater systems developed at the greenfield scale are most favorable. Care should be taken, however, in the practical implementation of this finding—there are, for example, governance issues relating to metering, payment, and water quality assurance that would need to be considered.

### 5.4. The Use of Multiobjective Optimization to Understand Performance Trade-offs

The use of multiobjective optimization helps manage the difficulties associated with the size of the decision space, and the development of Pareto surfaces presenting the trade-off between performance objectives.

In the case study, the development of these Pareto surfaces helped build the case that local water sources, such as graywater and blackwater, not only reduced dependence on centralized systems, but were also competitive in regard to costs and GHG emissions. For this case study, the PV of cost of water systems that used alternative water sources was often cheaper than that of conventional systems, and alternative water sources were able to reduce water supplied from centralized systems by as much as 54%.

### 5.5. Potential Development of Generic Guidelines

The application of the framework across (1) multiple case studies, with (2) additional criteria, and (3) accompanied by sensitivity analysis will generate domain knowledge regarding the performance of greenfield water systems that encompass IUWM concepts. Replication across multiple case studies will elucidate general trends from those which are case study dependant. The addition of other criteria, such as maximizing distribution network reliability/resilience [*Todini*, 2000; *Shinstine et al*., 2002], minimizing the change in the stormwater hydrograph from predevelopment status [*Burns et al*., 2012; *Mobley et al*., 2013], and minimizing pollutant load [*Oraei Zare et al*., 2012] will enable the understanding of broader trade-offs in design variables. The use of sensitivity analysis for investigating the impact of discount rates, climate change [*Paton et al*., 2014a], changes in the price of energy [*Wu et al*., 2012a], and the impact in regard to the trade-off between greenhouse gas emissions and costs based on the mix of energy sources used, and the carbon footprint of these energy sources, will be beneficial [*Stokes et al*., 2014a]. Sensitivity analysis, such as these, will be beneficial as there is much uncertainty regarding appropriate values of discount rates, and because of the increasing use of renewable energy resources into the future, which are both likely to have a large impact on outcomes.

## 6. Conclusions

This paper proposed a framework for the initial planning and design of water supply systems that incorporate IUWM elements, which is systematic and enables the identification of novel solutions for water supply through a broadened search space and use of evolutionary optimization techniques. This is important, because if the search space is constrained by not considering feasible water supply, treatment, storage, and conveyance options, innovative solutions that perform well may be excluded a priori.

The application of the framework to a case study added evidence that local water sources, such as rainwater, graywater, and blackwater, not only reduce dependence on centralized systems, but can also be competitive with regard to costs and energy use. The study also indicated that including blackwater within a recycled water stream was more beneficial than rainwater harvesting when considering greenhouse gas emissions, cost, and water savings. This finding is interesting, as the recycling of backwater has not been considered in greenfield developments to the extent that graywater and rainwater have.

In regard to the scale of local water systems, the case study has shown that economies of scale in treatment tend to outweigh diseconomies of scale in pipe networks. Therefore, these results suggest, tentatively, that cluster scaled systems at the rural township or suburban level are favorable in regard to costs and GHG emissions. Therefore, the installation of additional treatment processes at the treatment plants servicing rural townships, and the recycling of these waste waters would appear to be the most cost and energy effective means to reduce a township's water footprint.

With regard to further work, the robustness of solutions identified by the framework should be investigated with respect to sensitive parameters (such as interest rates), governance issues (such as ensuring source water quality in household discharges and roof runoff), and consumer preferences (such as the value consumers place on the effortlessness of public water and wastewater services).

Finally, one of the difficulties the framework may pose is the complexity inherent in, and computational demand required for optimizing the designs for (what could be) a very large number of alternative systems when the broadened decision space is considered; this may even be considered impractical by some, as conventionally only one system is considered without optimization. However, the authors argue that the framework is beneficial, and will become more readily adopted in practice due to (1) resource scarcity and environmental concern, which will encourage greater investment in planning that increases resourcefulness and reduces degradation, (2) continued increases in computational power and the efficiency of optimization algorithms, (3) the availability of software, such as eWater Urban Developer (http://www.ewater.com.au/products/ewater-toolkit/urban-tools/urban-developer/), that provide an integrated modeling environment, with built-in optimization capability, for water system planning. In addition, the application of the framework to a number of case studies in different parts of Australia could yield some generic conclusions that would assist in reducing the complexity and computational demand of planning and design of integrated urban water systems.

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