Principles for Optimizing the Scale of Direct Potable Water Reuse: Economic Network Modeling Studies

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PRINCIPLES FOR OPTIMIZING THE SCALE OF DIRECT POTABLE WATER REUSE: ECONOMIC NETWORK MODELING STUDIES

By

Tianjiao Guo

A DISSERTATION

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PRINCIPLES FOR OPTIMIZING THE SCALE OF DIRECT POTABLE WATER REUSE: ECONOMIC NETWORK MODELING STUDIES

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The US National Research Council recently recommended direct potable water reuse (DPR), or potable water reuse without environmental buffer, for consideration to address rising US water demand. In addition, conveyance of wastewater and water to and from centralized treatment plants currently consumes on average four times the energy of treatment in the US. Moreover, scaling of DPR systems involves tradeoffs beyond those of treatment facility economy-of-scale versus cost and energy of conveyance. In particular, additional factors favoring distributed DPR include minimization of energy for upgradient distribution of treated water, and retention of wastewater thermal energy. Therefore, a network modeling study addressing the optimal scale of DPR plants, considering variability in population density and topography, is presented in this dissertation.

First, information on the cost of unit treatment processes potentially useful for DPR versus system capacity is reviewed, converted to constant 2012 US dollars, and synthesized. A logarithmic variant of the Williams Law cost function is proposed as applicable over orders of magnitude of system capacity, for the subject processes: activated sludge, membrane bioreactor, coagulation/flocculation, reverse osmosis,
ultrafiltration, peroxone and granular activated carbon. Results are then demonstrated versus 10 DPR case studies.

A generalized model of the cost of DPR water as a function of treatment plant scale, assuming futuristic, optimized conveyance networks, is then proposed for purposes of developing design principles. Fractal landscapes representing flat, hilly, and mountainous topographies were simulated, with urban, suburban, and rural housing distributions placed by modified preferential growth algorithm. Treatment plants were allocated by agglomerative hierarchical clustering, networked to buildings by minimum spanning tree. Simulation results indicate total DPR capital and operation & maintenance (O&M) costs, assuming new urban facilities with 20-year design life capable of mineralizing chemical oxygen demand to below detection limits, is competitive with current water/wastewater service costs at scales of ca. one plant per 10,000 residences. Costs for rural systems are high and dominated at most scales by the cost of capital for pipeline installation, while urban/suburban system cost is driven by a balance between pipeline installation and treatment equipment capital. The optimal scale of mineralizing DPR systems is projected to range widely in rural areas, and to range to service populations at least as small as 100 homes in suburban areas and 1000 residences in urban areas. Therefore, distributed DPR systems are recommended for consideration for municipal water and wastewater system capacity expansion projects, particularly in new construction zones.

Finally, the proposed model is applied and demonstrated to evaluate the feasibility and optimal scale of DPR plants versus current plans for treatment capacity expansion in Miami-Dade County, Florida. Local data on the distribution of population and housing structures, and topography, were input, to evaluate four scenarios for the expansion
service area: (a) proposed new wastewater treatment plant (WWTP) and assumed new water treatment plant treating County-projected flow; (b) central DPR treating flow expected under generalized conditions; (c) central DPR treating County-projected flow; (d) optimal distributed DPR treating expected generalized flow; and (e) new central water and wastewater systems treating expected generalized flow. Results suggest that DPR systems which mineralize organics so as to essentially eliminate discharge of endocrine-disrupting compounds to the environment may represent a practical alternative in many applications. Total cost was minimized at a scale of 46 plants for the service population of 671,823 (4,810 per plant). Though DPR capital cost is projected at approximately twice that of the current plan, the total unit cost of $13.00/1000 gallons when added to O&M costs is approximately 51% higher than might be estimated for the current plan, and is less than reported for several major US cities and Florida municipalities. Overall, the model presented in this work confirms DPR as a potential water management alternative to address increasing water demand in the future, and presents an optimization approach that may be useful in planning studies. General design principals regarding the scale of DPR systems include the use of 100-10,000-home DPR systems in urban/suburban areas, and consideration of systems that return nutrients to agricultural sectors in rural areas.
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NOMENCLATURE

COD - chemical oxygen demand
DPR - direct potable water reuse
EDC - endocrine disrupting compound
EPA - Environmental Protection Agency
GAC - granular activated carbon
GPD - gallons per day
gpcd - gallons per capita per day
IMA - iron-mediated aeration
MBR - membrane bioreactor
MGD - million gallons per day
MST - minimum spanning tree
O&M - operation and maintenance
RO - reverse osmosis
UV - ultraviolet
WDWWTP - West District Waste Water Treatment Plant
WWTP - wastewater treatment plant
CHAPTER 1 GLOBAL INTRODUCTION

1.1 Motivations

Water is a vital resource on the planet, such that most human life and production activities would cease without sufficient fresh water. While most fresh water relies on limited sources of surface rivers, lakes and underground aquifers, existing surface water and groundwater supplies are becoming depleted due to overexploitation, urban development, and climate change (Vorosmarty et al., 2000; Geelson et al., 2012). At the same time, water demand is increasing due to the expanding population. The solution to this issue was aggressively pursued in the US during the 20th century by building major water infrastructures, particularly dams, aqueducts and pipelines (Graf, 1999). While tremendous benefits have resulted from these facilities, the rate of construction of large water infrastructures has dropped off greatly in recent decades due to significant decreasing or diminishing river flows. This has raised increasing concern about the adverse impacts to the ecosystems and declining economic support (Gleick, 2003), implying that the previous approach is reaching its limit. While the problem of increasing water demands remains, these infrastructures are now aging and becoming one of the nation’s top water priorities (US EPA, 2014a), providing impetus to reevaluate current US water and wastewater systems and calling for new approaches to address future water demand.

Compared to other approaches as seawater desalination, rainwater harvesting, iceberg melting, treated municipal wastewater provides a climate-independent fresh water source that is stable, non-seasonal and can be locally controlled. It allows communities to become less dependent on traditional surface or ground water sources, and eliminates or reduces nutrient loads from the wastewater releasing into the environment. Also, treated
municipal wastewater meets both primary and secondary standards for drinking water, with exceptions of primary standards for antimony and total coliform, and secondary standards for color, odor, TDS and foaming agents (Bloetscher et al., 2005). It is two orders of magnitude lower in total dissolved solids than marine saltwater sources and can be used to replenish overdrawn water sources and rejuvenate previously destroyed ones. In addition, the amount of water to be reused mainly comes from previous residential, commercial wastewater and rainwater runoff. It would satisfy demand for drinking water; theoretically there would be considerably less dependence on any other water source if municipal wastewater can be fully reused.

In general, potable water reuse has been termed either “indirect” or “direct”, depending on whether the treated wastewater was returned to the environment prior to reuse or not. Currently most potable water reuse systems include an environmental buffer integral to their design process, to ensure enough retention time, attenuation of contaminants, and dilution. However, direct potable water reuse, or potable water reuse without environmental buffer, was implemented successfully in Colorado from 1976 – 1982 (Selby & Pure Cycle Corp., 1979) and the International Space Station (Carter, 1999). In addition, the National Research Council (2011) further noted that there is no evidence that such an environmental buffer provides any public health protection that is not available in an engineered system. Thus, direct potable water reuse can be reasonably considered as a potential solution to addressing the issue of increasing water demand.

Most US water and wastewater systems are built as “centralized” systems. However, onsite wastewater treatment systems, or “decentralized” systems are employed in approximately 20% of all homes in the US (US EPA, 2014b). Even though centralized
plants could earn economy of scale with large capacities of treatment facilities, the energy cost of transporting the water over distances must be considered when selecting wastewater treatment plants. However, studies show that providing reliable wastewater services and safe drinking water are highly energy-intensive activities in the US; current water and wastewater conveyance energy spent could be even more than that in treatment processes (Burton, 1996; Klein et al., 2005; US EPA, 2010). In particular, for all the treatment and conveyance of water/wastewater, which includes agricultural irrigation, thermoelectric generation, municipal water/wastewater treatment and distribution, and industrial and residential process, an estimate of 4% of US electricity has been reported, of which 80% is used for conveyance (ICF consulting, 2002; Cohen et al., 2004). Therefore, the same question would arise: if direct potable water reuse systems would be widely applied in the future, what is the optimal scale of distributed DPR systems? This dissertation attempts to answer this question.

1.2 Previous related work

Considering that both energy and materials used in construction and operation of DPR systems can be measured by cost, the problem of optimal scale of DPR systems can be simplified and transformed to the following question: Given a certain study area, with a fixed number of end users that need to be provided with DPR service, what is the minimal DPR water cost, and how is this cost estimated? In order to answer this question, the process of producing and transporting water, including DPR treatment plant, landscape, end-user, pipelines, and flow of the water, needs to be simulated. Additionally, cost information of water reuse technologies in both capital and operational and
maintenance (O&M), as well as pipelines, and certain optimization methods are required to find a minimal value of the water cost. Thus, a review of previous related work in each sub-problem to this question is offered.

1.2.1 Landscape simulation

Usually in the design of optimal water and sewer network, landscape is a very important element to be considered. Since Mandelbrot (1967), the concept of fractal has been a successful mathematical model for describing irregular shapes or distributions. Fractal models and related analysis techniques are largely used for characterization of the spatial relationships of the earth's surface (Fox, 1989; Piech & Piech, 1990; Ouchi & Matsushita, 1992), and investigation of the structure of river networks or the river-length/catchment-area ratios (Hjelmfelt, 1988; Tarboton et al., 1988). Thus the concept of fractal could be used to simulate landscape in the optimization model of scaling of DPR systems.

1.2.2 End-user simulation

There are several models that can be used to mimic the natural growth state of the service cluster which consists of end-users. In particular, if the growth of the system focuses on the every single end-user comes into the study area, both Game of Life (Garden, 1970) and preferential growth model (Pennock et al., 2002) can be used to simulate this process. The preferential growth model generates scale-free networks. It incorporates two general concepts: growth and preferential attachment, which means that the more a connected node is, the more likely it is to receive new links, characteristics
that exist in many real networks. Game of Life, on the other hand, is a cellular
automation model which consists of a grid of cells whose state follows certain rules.
However, these two models have their limitations in this study: Game of Life has a death
mechanism which is not suitable here, and original preferential growth model does not
have definitions in edges of the nodes; it would be a reasonable assumption that a
combination of both may be helpful.

1.2.3 Cost information of water reuse technologies

Detailed instruction and cost information for the activated sludge process has been
provided cost estimation under certain capacities for the following of the water
disinfection and wastewater treatment processes: hypochlorination, chlorine dioxide,
ozonation for disinfection, nanofiltration, coagulation, ultrafiltration, reverse osmosis
(RO), and granular activated carbon (GAC). However, the information only covers
limited range of treatment capacities, and some important water reuse technologies, such
as membrane bioreactor and peroxone, were not included. In addition, cost estimates vary
in in terms of base year and may not be presented as a function of treatment capacity.
Therefore, all essential information on potential capital and O&M costs of DPR, were
reviewed, updated and synthesized in constant US dollars in the form of cost functions,
ranging over orders of magnitude in terms of treatment capacity.
1.2.4 Alternative DPR designs

In design of DPR systems, one fundamental concern is the minerals in the water. While employment of biological and advanced oxidation processes can remove most of organics in the influent in terms of total chemical oxygen demand (COD), minerals as calcium, magnesium, nitrate, fluoride, bicarbonate, etc. would still accumulate in the produced water after cycles of system running. In terms of classes of water reuse design, it includes those that remove minerals with RO-based systems, such as the Big Spring, TX wastewater reuse project (Sloan, 2007), Orange County Water District’s groundwater replenishment system (Woodside & Westropp, 2009), and Singapore NEWater project (Zhang et al., 2009), and those that do not, e.g. Goreangab water reclamation plant in City of Windhoek, Namibia (Lahnsteiner & Lempert, 2007), case of Chanute, KS (Metzler et al., 1958) and University of Miami net-zero water dorm project (Englehardt et al., 2013). While RO-based DPR systems can remove most mineral ions with high-pressured membrane technology, the requirement in pretreatment, disposal of concentrates, and membrane replacement has made it a process that can be expensive in capital and energy-intensive. Recent studies and practice (Englehardt et al., 2013; Deng et al., 2013) also suggest that a DPR system design that includes processes for aeration in the presence of metallic iron (iron-mediated aeration, described in Chapter 4) and advanced oxidation capable of mineralizing total organics to below the detection limit in terms of COD provide potable mineral water through a combination of softening and oxidation. Thus in this study, a non-RO-based DPR system is assumed for consideration of trend of low-energy consumption technology in the future.
1.2.5 Optimization method

West et al. (1997) found that metabolic rates and other attributes in biological systems always follow the power law:

\[ Y = Y_0 \times M^b \]  

(1.1)
in which \( Y \) is an attribute in the bio system, e.g. metabolic rate, leg stiffness, \( Y_0 \) is a known value of the attribute, \( M \) is the body mass and \( b \) is the scaling exponent that is usually close to 1. In fact, the model may be applied as a general model of distribution networks. Direct potable water reuse networks, in particular, may have a similar network structure as circular system. Regardless of pump stations on the nodes, water distribution system is like a grid, while serving a similar transportation function of arteries; Clean water is sent out through networks in a relatively high pressure and then into end-users. Also, the wastewater flows back to the treatment plants through sewer networks are similar to veins. In fact, related researches (Smeers & Tytce, 1984; Gillot et al., 2007) show that investment and operating cost of wastewater treatment plants may be qualified as a function of size parameters such as population or flow rate. Thus this model would be suitable for self-similar DPR systems, and can be used to simulate a DPR system’s exponential growth. However, the fractal approach as proposed by West et al. (1997) is applicable to a uniform distribution of receptors, in two or three dimensions, whereas municipal water clients are distributed non-uniformly, such that a straightforward fractal pipe network would not be optimal.

The difficulty of the optimization processes mainly lies in a lack of certainty in both number and positions of the treatment plants. While previous researchers have used many optimization methods, including dynamic programming (Main, 1975; Nzewi et al, 1985),
simulated annealing (Loganathan et al., 1995), genetic algorithms (Simpson et al., 1994; Gupta I et al., 1999; Gupta A et al., 2008), and neutral network (Chang et al., 2001) for optimal design of water/sewer network and wastewater treatment system, these methods may not be suitable for scaling of DPR systems. Ideally, this study would require a method that can either divide the study area into subareas of each individual DPR plant, or somehow can get complete numerical solutions of all possible locations of fixed number of DPR plants.

In terms of optimal network design, minimum spanning tree (MST) may be a good method, as it minimizes pipe length and therefore significant capital expense in terms of installation. Dong et al. (2012) has used MST to estimate wastewater network in an optimization model for urban wastewater system layout planning. While current water distribution network designs usually comprise a looped network, so that points on main water lines can be supplied from at least two different directions, the practice does not apply to sewer networks. In addition, modern methods of trenchless pipe repair make pipeline repair more rapid and economical, so that minimization of total pipe length may be appropriate to minimize the cost of water/wastewater pipelines.

### 1.2.6 Thermal energy saving in DPR

In general, an average residential hot water energy cost of $228 is spent in the US for every household (US Department of Energy, 1998). According to recent DPR system research and practice (Englehardt et al., 2013), on-site DPR systems have higher water temperature than city water and can help retain thermal energy (from water heaters) in the produced water. Energy saving in the heating process reflects savings in cost. Thus if a
total cost of providing residential hot water is considered, this saving would also be one of the elements that are in favor of decentralized DPR systems. However, the amount of hot water heat saving in DPR systems has never been considered. Assuming 60 gpcd (gallon per capita per day) residential wastewater (Metcalf & Eddy Inc., 2013) and 2.6 persons per household (Lofqusit et al., 2012), this cost would roughly lead to a general saving of $4.00 /1000 gallons in US 1998 dollars (228/2.6/60/365 *1000 = 4.00) if all thermal energy are retained and hot water heaters are eliminated. While the actual value would be less considering heat loss during transportation of treated water in the pipelines, further analysis is needed for detail estimation of hot water thermal saving.

1.3 Objectives and scope of the dissertation

Design principles for direct potable reuse systems that minimize total energy requirements are important to the development of the technology, to address demands for both clean water and energy. The energy considered should comprise not only conveyance, pumping, and other energy consumed in plant operation, but the energy consumed in plant construction, chemicals manufacturing and transportation, equipment manufacturing, and other embodied energy. While such a life-cycle energy analysis would be beyond the scope of the optimization proposed here, the economics of any given process reflect such embodied energy even including levels of human skill required along the supply chain. Hence, an economic optimization of DPR system scale was desired, as a basis for the development of design principles. Given the motivations of study in optimal scaling of DPR systems, inadequate previous work in cost information and optimization methods, the objectives of this dissertation are:
1) To review and synthesize available information on the costs of individual unit processes applicable for distributed DPR, as a function of process scale, and to verify this information as possible versus previous limited experience with DPR system implementation;

2) To examine the generalized costs of DPR systems as a function of treatment plant scale, and to develop an integrated model of all system components (end-users, networks, treatment plants), considering variations in topography, population density, and available financing, with results expressed in terms of the life-cycle unit cost of water for each aspect of capital and O&M, as a basis for the development of principles for the scaling of such systems;

3) To present an economic case study comparing the cost of centralized or distributed DPR systems that is projected using the previously developed model, with costs reported in current plans of the Miami-Dade County for repair and rehabilitation of County’s water management infrastructure and the ocean outfall legislation compliance plan.

The scope of this research includes generalized cost estimation of unit processes of water reuse and other components of DPR water cost based on a combination of existing literatures and current companies’ quotes. All calculations within the model are programmed and executed on Matlab® 2013a module in Pegasus 2 supercomputer whose service is provided by High Performance Computing Services Center of University of Miami. Cost projections for the DPR systems assume a biologically-inspired system design that produces potable mineral water, whose detail system design and experiments have been reported previously (Deng et al., 2013; Englehardt et al., 2013). In short, the advantage of system lies in mineralization of total organics to below the detection limit in
terms of COD at ambient temperature and pressure, which would possibly result in: 1) less chlorine demand in the disinfection process, which also implies less chloride disinfection byproducts; 2) possible removal of endocrine disrupting compounds (EDCs) including pharmaceutical and personal care products that aren’t easily regulated in terms of environmental half-life. Of note, safety of drinking water has many other issues, and in-depth discussion regarding the treatment train and quality of produced water is not presented in detail in the dissertation. In addition, optimization of treatment processes, and costs other than DPR systems, e.g. decommission of current water and wastewater plant and networks, are also beyond the scope of this study.

1.4 Organizations
In this first chapter, the motivations of scale of distributed DPR systems, a review of previous related work, and objectives and scope of the research are discussed. In Chapter 2, information on the cost of unit treatment processes potentially useful for DPR versus system capacity was reviewed, converted to constant 2012 US dollars, and concluded in terms of a logarithmic variant of the Williams Law cost function that was applicable over orders of magnitude of system capacity, for the subject processes: activated sludge, membrane bioreactor, coagulation/flocculation, RO, ultrafiltration, peroxone and granular activated carbon. In Chapter 2 estimates of the costs of potential reuse treatment unit processes are presented as cost functions based on previously reported costs, and compared to information available on actual system costs, as initial verification of the cost functions. Based on the results of Chapter 2, a generalized model of the cost of DPR water as a function of treatment plant scale, assuming futuristic, optimized conveyance
networks, under flat, hilly, and mountainous simulated topographies, with urban, suburban, and rural housing distributions, was constructed as described in Chapter 3. Discussions of the cost of water produced by DPR systems as a function of population density, topographies, and available financing are also offered in Chapter 3. In Chapter 4, the model described in Chapter 3 is applied to study the estimated DPR cost for the service area of a new West District Waste Water Treatment Plant (WDWWTP) that has been proposed by Miami-Dade Water and Sewer Department, with detailed projections for four different DPR alternatives. Chapter 5 describes the conclusions drawn in this dissertation, including contributions and limitations of the current model, and general principles in design of DPR systems.
CHAPTER 2 REVIEW OF COST VERSUS SCALE: WATER AND WASTEWATER TREATMENT AND REUSE PROCESSES

2.1 Introductory remarks

US water/wastewater infrastructure is now aging and in need of repair or replacement, offering an opportunity for careful reassessment of the entire municipal water management system. In that light, a recent report by the US National Research Council (2011) found that “The use of reclaimed water to augment potable water supplies has significant potential for helping to meet future needs, ….” The report went on to note that although de facto potable reuse, involving the use of source water largely comprised of upstream wastewater effluent, is common in many US water systems, planned potable water reuse is not. Where practiced, potable water reuse has been termed either “indirect,” if treated wastewater is returned to the environment prior to reuse, or “direct,” if not. In fact globally there are currently no public water supplies utilizing more than 50% recycled wastewater. In that sense, all water reuse systems currently operating include an environmental buffer integral to the design. However, 100% DPR, i.e. potable reuse without environmental buffer, was implemented successfully in Colorado from 1976 – 1982 (Selby & Pure Cycle Corp., 1979). Further, the National Research Council report found no evidence that an environmental buffer provides generally higher dilution and attenuation relative to an engineered system, and recommended that potable reuse with or without environmental buffer be considered as a water management alternative.

When considering DPR, distributed DPR systems may further be considered, given that centralized potable reuse systems with gravity collection would require upgradient distribution of treated water. In fact, even in centralized systems, the energy consumed for conveyance may be significantly greater than is used in the treatment process.
According to (Cohen et al., 2004; Wolff et al., 2004), the energy cost per unit water supplied in California is approximately 20 times higher for conveyance than for treatment, and the number approaches 39.5 in San Diego. These numbers may derive in part from the 3% of total electric power used in delivering water from the San Francisco Bay-Delta to Southern California (Cohen et al., 2004). As another less extreme example, in Iowa, energy consumed in conveyance of water/wastewater represents 30% of that consumed in treatment (Sauer & Kimber, 2002). In fact, a study supported by the Electric Power Research Institute (EPRI) concluded that roughly 4% of US electricity is used for treatment and conveyance of water/wastewater, of which 80% is used for conveyance (ICF consulting, 2002; Cohen et al., 2004). In addition, Cohen’s analysis indicated that at least 6% of the water is lost in centralized distribution systems, resulting in higher demands for both water and energy.

In addition to saving energy, distributed plants may be more resilient to willful attack, and amenable to technological evolution, allowing incremental technological changes to be implemented and tested when required by local conditions. Modern communications technologies also make it conceivable that many water/wastewater monitoring, operation, and maintenance functions can be decentralized, supporting savings in conveyance energy and water. In fact, small-scale treatment of black water in semi-public buildings has been predicted as a trend in the future based on projected savings in energy and water (Timmeren, 2007).

In considering distributed DPR, the question arises as to the optimal scale of the individual treatment plants. In many cases this question would be addressed by attempting to minimize total cost, which might also tend to minimize life-cycle energy
demand. However, information on the costs of water reuse processes as a function of process scale is inadequate, existing primarily in grey literature sources, limited by the current lack of potable water reuse design experience and the specificity of cost information to site characteristics, technological developments, and temporal and political variability in monetary values.

Traditionally, cost functions of the Williams Law form, \( C = \beta \cdot Q^\alpha \), in which \( C \) is cost, \( Q \) is system scale (size), and \( \beta \) and \( \alpha \) are positive constants (Williams, 1947), have been found applicable for general capital, operation and maintenance costs. The exponent \( \alpha \) is less than one when economies of scale occur, and values ranging from 0.2 to 1.5 have been reported for many processes (Hinomoto, 1974; Tyteca, 1976; Gillot et al., 2007). Though economy of scale has generally been assumed for overall capital cost, diseconomy of scale has been reported for water treatment plant capital cost (Hinomoto, 1974).

Cost can be described in two general ways: cost in terms of energy and cost in terms of capital. If energy costs for water/wastewater plants are assumed to follow Williams Law, and conveyance energy were constant at 80% of total cost as mentioned above, then total cost would be approximately \( C_{\text{total, energy}} = 5 \cdot \beta \cdot Q^\alpha \), five times the capital cost. More realistically, considering a study area of \( N \) buildings, the DPR system can be designed into \( N \) treatment plants each serving a building, or one treatment plant serves \( N \) buildings. Thus, the capital cost of a system of \( N \) buildings can be assumed a similar function of system size, equal to that a multiplier times the cost for a system serving a single building, or \( C_{\text{total, capital}} = N^\gamma \cdot \beta \cdot Q^\alpha \), in which \( N \) is the number of buildings served in the
system and $\gamma$ is a constant, similar to Equation 1.1 in Chapter 1. Then, the ratio, $R$, of the
cost of a single system serving $N$ buildings to the cost of $N$ systems serving $N$ buildings
is: $R = \frac{N^\gamma \cdot \beta \cdot Q^\alpha}{N \cdot \beta \cdot \left(\frac{Q}{N}\right)^\alpha}$. At the break-even point for decentralization, this ratio is equal to
unity. In that case, $N^{\gamma + \alpha - 1} = 1$, or in general $\gamma + \alpha = 1$. That is, for the centralized system
to show advantage, the sum of the exponents must not exceed unity, even though the
fraction of energy dedicated to conveyance may increase with system size (i.e., show
diseconomy of scale, $\gamma > 1$). Hence, diseconomy of scale may be more common than has
been realized.

The purpose of this study is to review and synthesize available information on the
costs of individual unit processes applicable for distributed DPR, as a function of process
scale and to verify this information as possible versus previous limited experience with
DPR system implementation. In particular, review is presented of cost information for
unit processes useful for water reuse including activated sludge, membrane bioreactor,
coagulation/flocculation, reverse osmosis, ultrafiltration, peroxone, and granular
activated carbon, and continuous cost functions are developed. Review of costs for
rainwater harvesting, which are highly site-specific, and other existing and emerging
processes useful for reuse is not included in this study. Costs of previously reported reuse
treatment systems are then estimated based on the cost functions presented, and
compared to information available on actual system costs. Discussion and conclusions
regarding DPR system costs are offered.
2.2 Cost functions for water reuse unit processes

In this section, cost information for several water/wastewater treatment and reuse unit processes is reviewed. To synthesize results for general applicability, reported results are first converted to constant 2012 US dollars in proportion to the increase in the GDP deflator (US Bureau of Economic Analysis, 2013). Then, capital equipment and O&M costs were fitted to functions of system scale including the form suggested by Williams (1947) using the Levenberg-Marquardt algorithm (Marquardt, 1963) and SigmaPlot® version 12.0 software. The only function found to fit available costs data adequately over several orders of magnitude of system scale (including onsite systems) was a logarithmic variant of Williams’ power law, as follows:

\[
\log(y) = \alpha \log(x) + c
\]  

(2.1)

in which \(y\) is the cost ($), \(x\) is the system scale (m\(^3\)/d), \(\alpha, \beta, c\) are constants. The \(R^2\) values found for Equation 2.1 by non-linear regression are reported for each technology.

2.2.1 Activated sludge

The activated sludge process may be a useful DPR component for removal of organic and nitrogen constituents. US EPA has provided detail instruction and cost information for the whole treatment plant and separate processes of sludge dewatering, disposal and hauling to landfill (US EPA, 1976; 1979; 1980; 1983). Per capita capital cost was also reported for by Butts & Evans (1970) as a function of population served, with costs for factory-built package plants (750 to 5,000 people) and plants fabricated onsite (10,000 to 50,000 people) reported separately. In addition, costs obtained in current research, development, and construction of an onsite DPR system (Englehardt et al. 2013) indicate
capital costs of $49,600 (2009) and $36,334 (2011) for 5.68 m$^3$/d (1500 GPD) and 1.89 m$^3$/d (500 GPD) attached-growth biological treatment systems, respectively. In Figure 2.1, Equation 2.1 is fitted to the onsite data points, US EPA (1983) and the capital costs obtained from Butts & Evans (1970)'s functions for package and site-built plants of selected capacities of 283.9, 733.0, 1892.5, 3785.0, 8463.5, and 18,925 m$^3$/d, assuming an average flow per capita as 0.378m$^3$/d (100 GPD) at capacities of 750, $\sqrt{750 \times 5000}$, 5000, 10000, $\sqrt{10000 \times 50000}$, 50000 people, with $R^2$ value of 0.999. Also shown is a line corresponding to a linear relationship between cost and capacity, i.e. neither economy nor diseconomy of scale, passing through the data point for the smallest plant. Economy of scale is indicated in this case, by a flatter empirical slope relative to the dashed line.
2.2.2 Membrane bioreactor

The membrane bioreactor (MBR) process is a relatively new modification of the activated sludge process which may provide higher quality effluent, appropriate for DPR. Costs vary greatly with local construction and power rates, and cost functions are few. DeCarolis et al. (2007) estimated total capital and annual costs for 3,785.4 m$^3$/d (1 MGD) MBR facilities of $0.533 - 0.682/m$^3$ water treated, in which total capital cost ranges from $1,419,000 - $2,330,000, and the corresponding annual O&M cost is $218,000 -
$302,000. Membrane replacement (28%) and energy costs (34%) were found to be the largest components of O&M expense, and continues with equipment repair/replacement (19%). Another report by DeCarolis et al. (2004) showed that the total capital and operating cost for 3,785.4 m$^3$/d (1 MGD) MBR raw wastewater reclamation systems ranged from $0.478 - $0.592/m$^3$, supporting their subsequent conclusion that the total cost of MBR facilities is relatively constant (DeCarolis et al., 2007). For a 1.89 m$^3$/d (500 GPD) onsite MBR system, a capital cost of $54,000 and annual O&M cost of $600 are indicated based on prices obtained in current research, development, and construction of a DPR system (Englehardt et al., 2013). These data are shown in Figure 2.2, along with Equation 1 fitted to the same data ($R^2 = 0.996$ and 0.999 for capital and operating costs, respectively). The capital costs in Figure 2.2 include MBR, mechanical components, pump, chlorine dosing system, land, and engineering fee. Annual O&M costs include electrical power, equipment repairs, chemical cleaning, membrane replacement, and other labor. Economy of scale is seen for both capital and O&M costs.
2.2.3 Coagulation and flocculation

Coagulation and flocculation can effect colloid-scale removal at larger nominal filtration pore sizes, and aid in removal of phosphorus, metals, organics, and other constituents. Estimated costs of coagulation at neutral pH, assuming 56.5mg/L of coagulant dose (alum or ferric) and 2.5mg/L of caustic dose, can be estimated using an online simulation tool developed by the Water Research Foundation (2009) and US EPA (2007), as shown in Figure 2.3 for plant capacities 37.85, 378.5, 3785, 37850, 378500 m$^3$/d. Capital costs include upgrades to existing chemical feed systems, piping and valves, and instrumentation and controls. O&M costs include chemicals, power, replacement parts, and maintenance labor. Also, data obtained in current research and
development (Englehardt et al., 2013) suggest a capital cost of $1,500 and annual O&M cost of $600 in 2012 for a 1.51 m$^3$/d (400 GPD) coagulation system. Those data and the fitted Equation 1 are shown in Figure 2.3 ($R^2 = 0.990$ and 0.999 for capital and O&M costs, respectively). Economy of scale is found for both capital, and operation and maintenance cost.

![Figure 2.3 Approximate capital and O&M costs of coagulation based on Water Research Foundation (2009) and US EPA (2007), and Englehardt et al. (2013). [Conditions: costs converted to constant 2012 US dollars proportional to the GDP deflator (US Bureau of Economic Analysis, 2013)].](image)

Electrocoagulation represents an economical coagulation approach avoiding the introduction of soluble anions, e.g. sulfate or chloride. Bayramoglu et al. (2004) presented a simplified operating cost analysis for the treatment of a textile wastewater by
electrocoagulation using iron and aluminum electrodes. While no identical/different cost function related to scale is reported, they suggested an operating cost function of the form

\[ C = a \cdot C_{\text{energy}} + b \cdot C_{\text{electrode}}, \]

in which \( a \) is electrical energy price, \( b \) is electrode material price, \( C_{\text{energy}} \) and \( C_{\text{electrode}} \) are consumption quantities per kilogram of chemical oxygen demand (COD) removed. Assuming an industrial electrical energy price, \( a = $0.06/$kWh, \)
and electrode material price at $1.80/kg for aluminum and $0.30/kg for iron, they found an operating cost of $0.3-0.6/kg COD for aluminum electrodes and $0.1-0.2/kg COD for iron. The latter represents ~ $14,000/year for a 5,000m\(^3\)/d plant removing 50 mg/L COD. The authors also noted that lower initial pH and higher conductivity result in lower energy consumption.

### 2.2.4 Reverse osmosis

Reverse osmosis (RO) is employed in many reuse systems. Pretreatment comprising for example coagulation/flocculation, sedimentation, filtration, and/or disinfection is required in order to meet potable water standards (Bixio et al., 2005). Côté & Liu (2004) discussed two options for pretreatment in reuse applications: conventional activated sludge treatment followed by tertiary filtration, and integrated MBR treatment. Côté et al. (2005) estimated the capital cost at $161/m\(^3\)/d for infrastructure and pretreatment, and $321/m\(^3\)/d for the RO process, assuming membrane pretreatment, 75% recovery, 20 L/m\(^3\)/h of flux, and 13.6 bar of pressure, not considering concentrate disposal costs. Total life-cycle costs were estimated at $0.07/m\(^3\) capital cost plus $0.21/m\(^3\) O&M, for a total $0.28/m\(^3\) to produce potable water from secondary effluent.
Although Akgul et al. (2008) reported a similar cost for RO seawater desalination in Turkey, water desalination applications in general have been suggested to be doubly-expensive relative to water reuse and reclamation (Côté et al., 2005). Assuming only 30-40% recovery, a 3-5 year membrane life, 15 year system design life, and $0.06/kWh of energy, unit capital cost is found as approximately $0.375/m³ at a capacity of 250 m³/d, decreasing slowly up to 2,000 m³/d capacity at which point cost becomes constant at approximately $0.175/m³, while the unit operating cost is reported almost stable at ~$0.20/m³.

At the onsite system scale, data collected in current research and development (Englehardt et al., 2013) indicate a capital cost of $5,750 in 2012 and an annual O&M cost of $ 1,000 for a 2200 GPD (8.33 m³/d) RO system without pretreatment. The online simulation tool from Water Research Foundation (2009) and US EPA (2007) also provides cost analysis of RO systems. Cost functions shown in Figure 2.4 were found by fitting all available data. Capital cost in Figure 2.4 includes membrane and infrastructure, including feed pumps, associated chemical feed equipment, and electrical and instrumentation, but not pretreatment, which is specific to source water quality. O&M costs are based on data reflecting 40% recovery and 14.2 L/m²/h for 250 and 2,000 m³/d capacities (Akgul et al. 2008), 75% recovery and 20.0 L/m²/h for 75,000 m³/d (Côté et al., 2005), and data for capacities of 37.85, 378.5, 3785, 37850, 378500 m³/d from Water Research Foundation (2009) and US EPA (2007). O&M costs include power, replacement parts, membrane replacement, and maintenance labor. Also, cost data from Akgul et al. (2008) have been divided by a factor of 2.2, the reported ratio of the cost for desalination versus water reclamation applications (Côté et al., 2005). Non-linear
regression $R^2$ values of 0.978 and 0.954 were found for capital and operating cost. However, in general, O&M cost can vary significantly due to variations in recovery rate, RO flux, membrane life, and pretreatment. Slight economy of scale is evident in both capital and operation and maintenance cost. In general, RO costs appeared to be most variable among the water reuse technologies reviewed, and the cost function presented here is intended only for preliminary analysis.

![Graph showing approximate capital and O&M costs of treatment RO plants capacity based on Côté et al. (2005), Akgul et al. (2008), Water Research Foundation (2009) and US EPA (2007), and Englehardt et al. (2013). [Conditions: constant 2012 US dollars proportional to the GDP deflator (US Bureau of Economic Analysis, 2013)].]
2.2.5 Ultrafiltration

Ultrafiltration is a relatively low-energy, high efficiency filtration process, successfully employed in water reuse applications. Pickering & Wiesner (1993) presented a model of low-pressure membrane filtration cost which indicates that ultrafiltration and other membrane filtration process are typically less expensive than conventional filtration for plant capacity less than 48,000 m$^3$/d. Also, a study of Drouiche et al. (2001) indicated that a 480 m$^3$/d drinking water system employing ultrafiltration of surface water in the Kabylia region of Algeria incurred a total capital and operational cost of $0.234/m$^3$.

Amortized capital cost over the 15 year period was considered the largest expense, at $0.117/m^3$, followed by interest on the invested capital at $0.052/m^3$ (3% annually), maintenance at $0.026/m^3$ (assumed as 1.5% per year), and membrane replacement at $0.025/m^3$. Costs for other items including power, cleaning, and labor were relatively small. In addition, the online simulation tool developed by the Water Research Foundation (2009) and US EPA (2007) also gives estimated costs for ultrafiltration and microfiltration as shown in Figure 2.5 for 37.85, 378.5, 3785, 37850, 378500 m$^3$/d plant capacities. Costs obtained in current research and development (Englehardt et al., 2013) indicate a capital cost of $10,000 and annual O&M cost of $300 in 2012 for a 1.51 m$^3$/d (400 GPD) ultrafiltration system. Non-linear regression fitting of the simulation tool and onsite data to Equation 2.1 is shown in Figure 2.5 ($R^2 = 0.988$ and 0.990 for capital and operating costs, respectively). Capital costs do not include pre-treatment and post-treatment. O&M costs include power, replacement parts, membrane replacement, chemicals, and maintenance-related labor. Economy of scale is evident for capital cost, but not O&M.
Figure 2.5 Approximate capital and O&M costs of ultrafiltration and microfiltration based on Water Research Foundation (2009) and US EPA (2007), and Englehardt et al. (2013). [Conditions: constant 2012 US dollars proportional to the GDP deflator (US Bureau of Economic Analysis, 2013)].

2.2.6 Peroxone (hydrogen peroxide/ozone) for organics mineralization

Ozone is useful for disinfection without the introduction of chlorides, and the emerging hydrogen peroxide-ozone, or peroxone, process (Crittenden et al., 2012) is efficient for the advanced oxidation of organic constituents in secondary effluent (Englehardt et al., 2013). In this study it was desired to preliminarily assess the cost of the peroxone process if used in the future for complete COD mineralization, to address potential issues with disinfection byproducts and endocrine disrupting constituents in recycled water. Information on process costs as a function of plant capacity is currently
limited. However, the Metropolitan Water District of Southern California (MWDSC) and James M. Montgomery, Consulting Engineers, Inc. (1991) estimated that for a new 378,500 m$^3$/d (100 MGD) peroxone treatment plant, designed to disinfect and remove the taste/odor compounds geosmin and methylisoborneol at 2 mg/L ozone dose, peroxone system costs can be estimated at $9.0 million capital and $0.55 million annual O&M. Cost estimates in 1997 US dollars are also given for five scenario plants assuming an air-fed ozone generator capable of supplying a maximum ozone dose of 2.0 mg/L and an average dose of 1.5 mg/L, an air preparation system, buildings, a 25% uncertainty factor, 20% for engineering and administration, and another 25% for contingency. Because current research indicates that a dose of 130 mg/L may be required for effective mineralization of total COD (Wu, 2013; Englehardt et al., 2013), assumed plant capacities for the MWDSC data were reduced by a factor of 65. In addition, current research and development (Englehardt et al., 2013) suggests a capital cost of $35,000 and annual O&M cost of $500 for a 1.51 m$^3$/d (400 GPD) peroxone system, a capital cost of $45,000 and annual O&M cost of $1,000 for a 3.03 m$^3$/d (800 GPD) system, and a capital cost of $55,000 and annual O&M cost of $2,000 for a 5.68 m$^3$/d (1500 GPD) system, all in 2012 US dollars. The cost functions developed by regression from all of these data are shown in Figure 2.6 (Equation 2.1, $R^2 = 0.999$ and 0.999), though these preliminary curves should be used with caution. In general, economy of scale is indicated for capital cost. Costs for disinfection alone can be estimated by multiplying assumed plant capacity by 65. It should also be noted that costs for such systems may fall significantly with further development and increased population.
Figure 2.6 Approximate capital and O&M costs for mineralization of COD by peroxone based on the MWDSC and James M. Montgomery, Consulting Engineers, Inc. (1991), and Englehardt et al. (2013). [Conditions: constant 2012 US dollars proportional to the GDP deflator (US Bureau of Economic Analysis, 2013)].

2.2.7 Granular activated carbon

Granular activated carbon (GAC) may represent a relatively low-energy process, when used for polishing and redundancy in water reuse systems so that required reactivation/recharge is minimal. Cost functions for concrete gravity contactors and pressure contactors for field-scale systems, and general capital cost function for various GAC systems were presented by Clark (1987) and Clark & Lykins (1989). The multiple cost functions identified are in similar form to Williams Law, and can be used to generate detailed cost estimates for different aspects of particular GAC systems based on capacity
parameters such as total GAC volume/mass, system flow rate, and contactor cross-sectional area.

For general cost estimation, Water Research Foundation (2009) and US EPA (2007) developed an online simulation tool which gives total cost as a function of system capacity, as shown in Figure 2.7 for plant capacities 37.85, 378.5, 3785, 37850, 378500 m³/d. For onsite systems, current research and development (Englehardt et al., 2013) suggests a capital cost of $3,500 and an annual O&M cost of $1,000 in 2012 US dollars, for a 1.51 m³/d (400 GPD) GAC system. Capital costs include the addition of GAC contactors, initial carbon charge, associated piping and valves, and instrumentation and controls. O&M costs include spent GAC reactivation, power, replacement parts, and maintenance labor. Due to the recency and completeness of these data sources taken together, a general cost function was fitted to the output of the online simulation tool and the onsite data, as shown in Figure 2.7 ($R^2 = 0.996$ and $0.991$).
2.2.8 Cost function summary

The cost functions for the technologies reviewed in this section are not universal. Local construction requirements vary widely, and combined systems may have different cost functions when applied in water and wastewater treatment regarding different influent quality and requirement of treated water. COWI Consulting (2005) suggests that for conventional treatment of surface water including pre-treatment, coagulation/flocculation, sedimentation, filtration and disinfection, a capital cost function would be $C=18200 \cdot Q^{0.51}$, in which $C$ is the capital cost (€), and $Q$ is the flowrate (m$^3$/d). O&M cost can be assumed at 8% of the capital cost. For a small 50-750 m$^3$/d domestic
wastewater treatment plant using an optimized combined sand filtration and ozone process, total capital and operational cost for five years was reportedly about $0.1-$0.32/m$^3$ (Ni et al. 2003). Overall, the cost functions reported here represent a basis for screening and scaling of water reuse treatment processes. A summary of cost functions for these water reuse unit processes is given in Table 2.1.

Table 2.1 Summary of Cost Functions of Water Reuse Technologies \( y \) is cost ($), \( x \) is capacity (m$^3$/d)

<table>
<thead>
<tr>
<th>Water reuse technologies</th>
<th>Capital Cost</th>
<th>Annual O&amp;M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Activated sludge</td>
<td>log(y = 0.469 \times (\log(x))^{1.086} + 4.444)</td>
<td>-</td>
</tr>
<tr>
<td>Membrane bioreactor</td>
<td>log(y = 0.569 \times (\log(x))^{1.135} + 4.605)</td>
<td>log(y = 0.639 \times (\log(x))^{1.143} + 2.633)</td>
</tr>
<tr>
<td>Coagulation and flocculation</td>
<td>log(y = 0.222 \times (\log(x))^{1.516} + 3.071)</td>
<td>log(y = 0.347 \times (\log(x))^{1.448} + 2.726)</td>
</tr>
<tr>
<td>Reverse osmosis</td>
<td>log(y = 0.966 \times (\log(x))^{0.929} + 3.082)</td>
<td>log(y = 0.534 \times (\log(x))^{1.253} + 2.786)</td>
</tr>
<tr>
<td>Ultrafiltration</td>
<td>log(y = 1.003 \times (\log(x))^{0.830} + 3.832)</td>
<td>log(y = 1.828 \times (\log(x))^{0.598} + 1.876)</td>
</tr>
<tr>
<td>Peroxone (mineralization)</td>
<td>log(y = 0.405 \times (\log(x))^{1.428} + 4.528)</td>
<td>log(y = 0.845 \times (\log(x))^{1.057} + 2.606)</td>
</tr>
<tr>
<td>Granular activated carbon</td>
<td>log(y = 0.722 \times (\log(x))^{1.023} + 3.443)</td>
<td>log(y = 1.669 \times (\log(x))^{0.559} + 2.371)</td>
</tr>
</tbody>
</table>
2.3 Experience with water reuse economics versus system scale

Potable reuse systems have been implemented several times at differing scales in differing contexts. A review of the economics versus the scale of previous known implementations of potable reuse with comparison of estimated cost and actual data is given in this section.

2.3.1 Biosphere 2

In the Biosphere 2 closure experiment, eight researchers lived under a transparent 12,700 m² (3.14 acre) dome containing an artificial ecosystem from 1991 to 1993. An external energy system provided partial water supply to the crew inside (Dempster, 1999). Both heating and cooling were provided by hot, cooled, and chilled water circulated from a closed piping system outside, through a heat exchanger in Biosphere 2. In the heat exchanger, 25 air handlers forced air circulation, each capable of an airflow up to 24 m³/s. Thus while maintaining the temperature and humidity, the system also condensed 20-40 m³/d from the vapor in atmosphere for potable uses. Water collected was used for drinking directly after UV sterilization. A separate water treatment system handled toilet, kitchen, and lab wastewater. Treatment consisted of anaerobic holding tanks and application to an agricultural system providing environmental buffer (Nelson et al., 1999). An 870 m³ storage tank equalized flow. Electrical power required by fans, pumps, and communications averaged about 700 kW, or more than $50,000/month, provided by the external energy house. The annual cost of fuel in the compressive chiller and hot water boiler was approximately $1 million (Dempster, 1999). Thus the extreme energy demand of this system resulted in costs that would be prohibitive in most
applications. Also, estimation of the treatment process is not available with cost functions concluded previously.

2.3.2 Windhoek, Namibia

The city of Windhoek, Namibia, population 240,000, built the Goreangab Water Reclamation Plant, a DPR facility, in the 1960s due to local water supply shortage and variability (Lahnsteiner & Lempert, 2007). With an initial capacity of 3,287 m³/d, and ultimately 7,500 m³/d, the plant produced 12%-18% of the total potable water supply for over 30 years. In 2002, a €12.5 million New Goreangab Water Reclamation Plant was built. Project influent was unchlorinated secondary effluent. The progressive city water price of $0.72/m³ for 0-0.2m³/d; $1.18/m³ for 0.201-1.8m³/d; and $2.22/m³ for > 1.8m³/d in 2004 was reasonable in comparison with US prices of ca. $0.40/m³. From Lahnsteiner & Lempert (2007), the treatment train in New Goreangab Water Reclamation Plant consists of enhanced coagulation and flocculation, ozonation, GAC filtration, and ultrafiltration, chlorine disinfection and stabilization. Based on Canizares et al. (2009), the capital cost of ozonation cost can be estimated as $P = 2359.85*(V / 12)^{0.6143}$ after conversion to 2012 US dollars, where $V$ is the flow rate (m³/d) and $P$ is the capital cost ($), assuming ozone dosage at 2 mg/L. Thus for a similar treatment train that contains every process but chlorine disinfection or stabilization in the original treatment process and assuming the same 21,000 m³/d capacity of the new plant, a capital cost of $21.3 million (2012 US dollars) would be estimated using the equations in Table 2.1. This estimation is close to the original construction cost ($17 million in 2012 US dollars).
2.3.3 Denver Potable Water Project

From 1979 to 1992, the Denver Potable Water Reuse Demonstration Project demonstrated the conversion of unchlorinated secondary effluent into water that could be directly piped into a drinking water distribution system. Product water from the 3,785.4 m$^3$/d (1.0 MGD) plant was not used for drinking, but stored and shown as part of the project’s public program (Rogers & Lauer, 1992). The plant’s construction cost was $18.5 million, with $6.0 million for scientific studies on health risks, and $8.4 million O&M over the 13 years (Lauer, 1993). From Rogers & Lauer (1992), the main treatment train in the demonstration plant consists of flocculation, reverse osmoses, two stages of GAC, ozonation, and ultrafiltration, lime treatment, recarbonation, air stripping, UV, and chlorine dioxide disinfection. For a similar system comprising every treatment process except for lime treatment, recarbonation, air stripping, UV or chlorine dioxide disinfection at a capacity of 3,785.4 m$^3$/d, a capital cost of $9.7 million would be estimated using the equations of Table 2.1. This estimation is lower than the actual construction cost ($27.8 million in 2012 US dollars). And the difference may come from the negligence of last five processes, and an additional side stream process in the actual plant that consists of a fluidized bed carbon reactivation furnace, vacuum sludge filtration, and selective ion exchange regenerant recovery (Rogers & Lauer, 1992), which are not accounted in the estimation.

2.3.4 International Space System

As of November 2008, the International Space System developed by NASA included a Water Recovery System consisting of Water Processor Assembly and Urine Processor
Assembly (Carter, 2009). The system provided drinking water to a crew of six members, derived from a combination of condensate and urine. Flush water and urine were treated with a formula containing chromium trioxide and sulfuric acid. From there the water passed to a Distillation Assembly, consisting of a rotating centrifuge where the wastewater and urine stream were evaporated and condensed. The Urine Processor Assembly was designed for a load of 9 kg/d (19.8 lb/d) and could recover a minimum of 85% of the water content, essentially equivalent to the 6-crew requirement. The Water Recovery System reportedly averaged 743 W power consumption while in operation, and 297 W while in stand-by, or less than perhaps $40/month, or $177.03/m³, which is high for a group of six people. Also, estimation with cost functions is not available for this case.

2.3.5 Village of Cloudcroft

Due to lack of sufficient water supply from local springs and wells, the Village of Cloudcroft, NM, population 850 increasing to more than 2,000 during holidays, constructed a system to provide potable water from purified wastewater (Livingston, 2008). Following treatment by membrane bioreactor, disinfection, reverse osmosis, and UV/hydrogen peroxide advanced oxidation, the treated municipal wastewater is blended with approximately 50% spring or well water. The blended water is detained for two weeks in a storage reservoir, after which it undergoes ultrafiltration, UV disinfection, and activated carbon adsorption. With a treatment capacity of 378.5 m³/d (100,000 GPD), capital cost of the project is roughly $3,500,000, with operating costs of $50,000/year and equipment maintenance costs of $0.21/m³. Overall, the cost of operation and maintenance
is $0.63/m³, and the total cost of product water is $2.38/m³. Given a similar system
comprising MBR, reverse osmosis, peroxone, ultrafiltration, and GAC at a capacity of
378.5 m³/d, a capital cost of $3.5 million (without UV disinfection in the original
treatment process) would be estimated using the equations of Table 2.1. This estimation
is close to actual cost data ($3.71 million in 2012 US dollars).

2.3.6 Chanute, Kansas

During a severe drought, the city of Chanute, Kansas, USA, population 12,000 at the
time, implemented emergency wastewater reclamation and reuse for municipal water
supply from October 1956 to March 1957 (Mangan, 1978; Asano et al., 2007). The
Neosho River, previously used to supply the city water demand of 5,300 m³/d (1.4
MGD), was dammed upstream of the treatment plant, and sewage treatment plant effluent
was returned to the river as source water. Treatment comprised standard 1950s physical-
chemical technology, including alum flocculation, sedimentation, sand filtration and
chlorine disinfection. Activated carbon and membrane filtration were not available at that
time. Assuming a similar treatment train consisting of activated sludge and flocculation,
without sand filtration and chlorine disinfection in (Metzler et al., 1958), at a capacity of
5,300 m³/d, a capital cost of $2.56 million would be estimated using the equations of
Table 2.1. However, actual construction cost of the treatment plant is not found.

2.3.7 Big Spring, TX

Due to a long term drought in the Permian Basin of West Texas, the Colorado River
Municipal Water District, which supplies water to the cities of Odessa, Big Spring and
Snyder, recently launched a wastewater reuse project. The plan is to expand the existing Big Spring Wastewater Treatment Plant and treat 7,949.4 m$^3$/d (2.1 MGD) of filtered secondary effluent with membrane filtration, reverse osmosis, and UV/hydrogen peroxide oxidation; blend it with raw water in the transmission line; and pass the water to a potable water treatment process which includes flocculation, sedimentation, granular media filtration and disinfection, before release to the distribution network to comprise ca. 5% of the finished water. In the preliminary design report (Sloan, 2007), construction cost of the project was estimated to be $8.23 million, including $3.45 million for treatment equipment, $0.88 million for the pump station, and $0.70 million for pump line. Annual operating cost was estimated at $667,000 for power, chemical, labor and equipment replacement. Produced water is projected to cost $0.68/m$^3$. Total energy consumption for operation of the membrane treatment, UV oxidation, and source water and product water pumping is projected at 1.41 kWh/m$^3$. This is comparable to the current local operating cost of 1.33 kWh/m$^3$, due principally to the long pumping distance and the 914.4 m (3,000 ft) elevation of Big Spring (Sloan et al., 2010). Assuming membrane filtration, reverse osmosis, UV/hydrogen peroxide oxidation, coagulation, and granular media filtration at a capacity of 7,949 m$^3$/d, a capital cost of $14.6 million (without disinfection in the original treatment process) can be estimated using the equations of Table 2.1. This estimation is higher than the official projected cost ($8.89 million in 2012 US dollars), mainly due to the fact that the project is an expansion rather than new construction.
2.3.8 Orange County Water District, CA

The Orange County Water District’s Groundwater Replenishment System treats disinfected secondary effluent with microfiltration, RO, and UV/hydrogen peroxide oxidation, and product water is used to recharge existing groundwater basins. The augmented Orange County groundwater basin supplies water for roughly 2.5 million people, while receiving up to 265,000 m$^3$/d of treated wastewater for recharge (Markus et al., 2008; Deshmukh, 2009; Tchobanoglous et al., 2011). This $481 million project had an annual budget of ~$34 million (Woodside & Westropp, 2009). According to Tchobanoglous et al. (2011), a detail unit treatment process can be concluded as microfiltration, cartridge filtration, reverse osmosis, advanced oxidation (UV photolysis and hydrogen peroxide), carbon dioxide stripping, and lime addition. Assuming a similar engineering process comprising microfiltration, three stages of RO, and UV/hydrogen peroxide oxidation, without cartridge filtration, carbon dioxide stripping or lime addition, at a capacity of 265,000 m$^3$/d, a capital cost of $253.8 million can be estimated using the equations of Table 2.1. This estimation is lower than the reported capital cost ($505 million in 2012 US dollars), which could be the result of an extensive public outreach and monitoring program for demonstration of the safety of product water and groundwater quality. Also note, this case is regarded to be mostly close to a real DPR project, except that the recycled water is introduced to environmental buffer for a minimum of 6 months.

2.3.9 Pure Cycle Corporation

In the 1970s, the Pure Cycle Corporation developed a complete closed-loop DPR system for single homes. These units were installed primarily in mountain homes in
Colorado from 1976 to 1982 (Harding, 2009). The system consisted of a wastewater holding tank, a biological digester, an ultrafiltration unit, and a deionization unit (Selby & Pure Cycle Corp., 1979). A central control system communicated with company headquarters in Denver. After the company exited the business due to the expense of maintaining single systems scattered throughout the mountains, homeowners obtained permission from the State to operate the systems independently. The cost information for this case is very limited, no capital or O&M costs for these systems were found in literatures. And cost functions are not available for this case. However, one could assume a cost of < $1.32/m³ water treated for disposal of acid and base regenerant, assuming on the order of 100 mg/L ions removed, to be a dominant operating cost. Also, the cost of brine evaporation or disposal is not known.

2.3.10 Singapore NEWater Project

With a large urban population and limited land area, the Singapore city-state launched the NEWaterProject in 2002, to reuse clarified secondary effluent as a supplemental water supply (Singapore Water Reclamation Study, 2002). Treatment comprises microfiltration, reverse osmosis, and UV disinfection, followed by blending with reservoir water for potable use. Recycled water contributed only ~2% to the finished potable water as of 2010. Two 72,000 m³/d plants were commissioned in Jan 2003. A third 24,000 m³/d plant began supplying water in Jan 2004, and a fourth 148,000 m³/d plant was brought online Jan 2005 (Asano et al., 2007). Cost of the product water including production, transmission and distribution was about S$1.30/m³ in 2003, decreasing to S$1.00/m³ (US $0.66/m³) by April 2007 (Zhang et al., 2009). Assuming
microfiltration, and reverse osmosis at a capacity of 316,000 m$^3$/d, a capital cost of $152.8 million (without disinfection) can be estimated using the equations of Table 2.1. However, no actual cost data were found for this case.

2.3.11 Case study summary

A summary of the scales and costs of previous potable reuse implementations is given in Table 2.2, based on the information in this section. Capital costs estimated as described for each case study in this section, using the cost functions developed in this work, are also shown in the table. From Table 2.2, most unit operating costs of these DPR cases are acceptable if compared to conventional water and sewer bills (Miami-Dade Water and Sewer Department, 2013a), except for Biosphere 2 and International Space Station. Despite the site-specificity of many labor, construction, and other costs, predicted capital costs are generally within a factor of two relative to reported costs, except for the Denver research and demonstration project, which contains a different side treatment process. Also, no obvious bias is apparent.
Table 2.2 Summary of Reported and Projected Reuse Case Study Costs in Constant 2012 US Dollars Proportional to the GDP Deflator (US Bureau of Economic Analysis, 2013)

<table>
<thead>
<tr>
<th>Case name</th>
<th>Total scale</th>
<th>Capital cost</th>
<th>Capital cost per average home served</th>
<th>Annual operation cost</th>
<th>Unit operating cost of water</th>
<th>Water reuse technologies</th>
<th>Estimated capital cost with cost functions available in Table 2.1</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Biosphere 2</td>
<td>20 - 40 m³/d</td>
<td>-</td>
<td>-</td>
<td>$ 2.13 million</td>
<td>$ 145.89 - 291.78 m³/d</td>
<td>Condensation</td>
<td>-</td>
<td>Dempster (1999)</td>
</tr>
<tr>
<td>Windhoek</td>
<td>21,000 m³/d</td>
<td>$ 17.0 million (€ 12.5 million in 2007)</td>
<td>$ 920</td>
<td>-</td>
<td>$ 0.40 - 0.45 m³/d (€ 0.30 - 0.35 m³/d in 2007)</td>
<td>Enhanced coagulation and flocculation, sedimentation, biological filtration, and disinfection</td>
<td>$21.3 million</td>
<td>Lahnsteiner &amp; Lempert (2007)</td>
</tr>
<tr>
<td>Denver Potable Water Project</td>
<td>3,785.4 m³/d (1 MGD)</td>
<td>$ 27.8 million</td>
<td>$ 8,325</td>
<td>$ 975,000</td>
<td>$ 0.68/m³</td>
<td>Filtration, UV disinfection, reverse osmosis, and air stripping, ozonation, aeration, sedimentation, and disinfection</td>
<td>$9.7 million</td>
<td>Rogers &amp; Lauer (1992)</td>
</tr>
<tr>
<td>International Space System</td>
<td>0.0078 m³/d</td>
<td>-</td>
<td>-</td>
<td>$ 504</td>
<td>$ 177.03 - 200 m³/d</td>
<td>Condensate and urine vacuum distillation</td>
<td>-</td>
<td>Carter (2009)</td>
</tr>
<tr>
<td>Village of Cloudcroft</td>
<td>378.5 m³/d (0.1 MGD)</td>
<td>$ 3.71 million</td>
<td>$ 11,130</td>
<td>$ 95,000</td>
<td>$ 0.67/m³ (2.40/1000 gallons)</td>
<td>Membrane bioreactor, disinfection, reverse osmosis, and UV/brine recovery, advanced oxidation</td>
<td>$3.5 million</td>
<td>Livingston (2008)</td>
</tr>
<tr>
<td>Chanute, KS</td>
<td>5,300 m³/d (1.4 MGD)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>&lt; $ 1.05/m³ (€ 4/1000 gallons)</td>
<td>Activated sludge, enhanced coagulation and flocculation, sedimentation and disinfection</td>
<td>$2.6 million</td>
<td>Metzler et al. (1958)</td>
</tr>
<tr>
<td>Big Spring, TX</td>
<td>7,949.4 m³/d (2.1 MGD)</td>
<td>$ 8.89 million</td>
<td>$ 1,067</td>
<td>$ 720,000</td>
<td>$ 0.73/m³</td>
<td>Membrane filtration, reverse osmosis, and UV/brine recovery, advanced oxidation, sedimentation and disinfection, granular media filtration</td>
<td>$14.6 million</td>
<td>Sloan (2007)</td>
</tr>
<tr>
<td>Orange County Water District, CA</td>
<td>265,000 m³/d (70 MGD)</td>
<td>$ 505 million</td>
<td>$ 2,164</td>
<td>$ 35.7 million</td>
<td>$ 1.26/m³ (€ 4.55/1000 gallons in 2009)</td>
<td>Microfiltration, 3 stages of reverse osmosis, and UV/brine recovery, advanced oxidation</td>
<td>$253.8 million</td>
<td>Woodside &amp; Westropp (2009)</td>
</tr>
<tr>
<td>Pure Cycle Corporation</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>&lt; $ 1.32/m³</td>
<td>Biological filtration, advanced oxidation, and disinfection</td>
<td>-</td>
<td>Selby &amp; Pure Cycle Corp. (1979)</td>
</tr>
<tr>
<td>Singapore NE Water Project</td>
<td>316,000 m³/d</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$ 0.69/m³</td>
<td>Microfiltration, reverse osmosis and UV disinfection</td>
<td>$152.8 million</td>
<td>Zhang et al. (2009)</td>
</tr>
</tbody>
</table>

a Assumes water usage of 1.14 m³/d (300 GPD) per home
b Calculated based on 81% reported Urine Processor Assembly recovery of 180 kg urine in 6 weeks, and the ratio of the flow rate of Urine Processor Assembly to Water Processor Assembly (Carter 2009)
c The city denied the decision of transport water at $ 4/1000 gallons for cost and physical limitation (Metzler et al. 1958)
d Based on estimated cost of acid and base regenerant, for removal of on the order of 100 mg/L ions.
e Primary and secondary treatment not included.
f Calculation based on actual flow rate rather than design flow
2.4 Summary

The applicability of distributed DPR systems will likely depend in part on local topographic, demographic, and hydrologic characteristics, on needs for reductions in energy consumption for water conveyance, and on projected increases in water demand. When substantial investment has previously been made in centralized water/wastewater treatment systems, the scaling of potable water reuse systems may be largely determined by existing infrastructure. However, much of the water/wastewater infrastructure in the US today is in need of repair and/or replacement, and therefore information on the cost of current technologies versus system scale will be needed. In particular, the following conclusions were drawn based on the literature reviewed:

• A logarithmic variant of the Williams Law cost function appears to apply satisfactorily to both capital and O&M cost of water reuse technologies, over orders of magnitude in system capacity;

• The cost functions found in the literature and derived in this work were roughly demonstrated versus available data on DPR systems; and

• Results indicate that economies of scale apply for many unit processes. However, capital and operating costs for collection/distribution networks counterbalance these economies in centralized systems. Therefore, study of the optimal scale of distributed DPR systems is recommended, along with further study of the costs of emerging processes.
CHAPTER 3  PRINCIPLES FOR SCALING OF DISTRIBUTED DIRECT POTABLE WATER REUSE SYSTEMS: A MODELING STUDY

3.1 Introductory remarks

Direct potable water reuse (DPR) systems, or systems which treat wastewater to produce potable water without an environmental treatment barrier, have been used to address water shortage (Metzler et al., 1958; Lahnsteiner & Lempert, 2007; Livingston, 2008), to serve homes in sparsely-populated mountainous areas (Selby & Pure Cycle Corp., 1979), and to serve the International Space Station (Carter, 2009). Recent studies and practice also suggest DPR as one method to address increasing water demands, and as a potential replacement for aging water/wastewater systems (National Research Council, 2011; Tchobanoglous et al., 2011; Englehardt et al., 2013). Although experience to date is limited, such systems employ commercially-available technology of proven reliability. In such cases, wastewater and water facilities are co-located, likely requiring (a) new equipment and (b) upgradient distribution of treated water, while offering the potential to retain residential hot water energy if conveyance distances were within acceptable range (discussed in Chapter 3.2.6). Hence, the usual tradeoff between economy of scale in terms of capital cost of treatment, and diseconomy of scale in terms of water/wastewater conveyance, becomes more complex. For those reasons, and considering the significant level of energy consumed in conveyance of water to and from centralized systems today (Cohen et al., 2004; Wolff et al., 2004), the design of DPR systems may involve consideration of the optimal scale of treatment plants.

Many studies have been reported regarding the optimal design of water and wastewater systems. For example, the design of wastewater treatment plants has been optimized with focus on the cost-energy-reliability balance, using neural networks.
(Chang et al., 2001) and genetic algorithms (Beraud et al., 2007; Gupta & Shrivastava, 2008). In terms of water and sewer conveyance networks, optimal design is usually considered an NP-hard (Non-deterministic Polynomial-time hard) problem, meaning loosely that there is no known way to show that a solution generally optimal, but when essential information is provided it may be easy to verify the solution (Yates et al., 1984).

In general, in order to find a low cost network solution, initial assumptions may be required as to system capacity, locations of pump stations in water distribution networks, connecting nodes in the network, and other aspects of system layout representing constraints. In addition, because the design depends directly on hydraulic heads in the system, solutions may be specific to the assumed topography. Typically, after an initial solution that meets specified demands at given nodes is given or obtained, approaches such as simulated annealing (Loganathan et al., 1995) and genetic algorithms (Simpson et al., 1994; Gupta I et al., 1999) have been applied to find a global optimum. For the design of sewerage networks in particular, dynamic programming (Main, 1975; Nzewi et al., 1985; Abraham et al., 1998) and genetic algorithms (Liang et al., 2004; Guo et al., 2006) have been developed for optimizing particular sewer configurations and conditions, e.g. in terms of benefit/cost ratio. In general, these models have demonstrated the potential of systems analysis for solving multi-parameter water/sewer network optimization problems. However, the problems addressed have generally been constrained in terms of spatial allocation of plant capacity, rather than focusing on generalized principles for scaling of treatment plants.

Several studies have involved the optimal design of complete wastewater collection and treatment systems. For example, life-cycle assessment (LCA) and life-cycle cost
(LCC) methods have been used to minimize capital and O&M cost (Vollertsen et al., 2002; Butler & Schutze, 2005; Leitao et al., 2005; Joksimovic et al., 2008; Lim et al., 2008). In that vein, Lim and coworkers (2008) calculated both environmental effect score and life cycle cost of a wastewater treatment network system comprising both distributed and terminal wastewater treatment plants using a mathematical optimization model. Results were compared with those of a traditional wastewater treatment system, indicating a lower cost for the combined system at the expense of an increased environmental impact due to higher electricity requirements for wastewater pumping. Also, Maurer (2009) introduced a specific net present value for the evaluation of wastewater systems, revealing that on-site systems may be favored over centralized systems when growth projections are high, building on previous estimates of Maurer et al. (2006) of the costs of replacing centralized water and wastewater systems with decentralized ones. And Makropoulos and coworkers have also evaluated new water resources as stormwater and wastewater (Makropoulos et al., 2008a) and brought discussions in possible scenarios of distributed systems with local wastewater reuse (Makropoulos et al., 2008b; Makropoulos & Butler, 2010), indicating the potential of a more decentralized, integrated urban water management in the future. In addition to such systems analyses, generalized costs of unit processes that may be used in DPR systems were recently reviewed, updated, and demonstrated as a function of system scale (Guo et al., 2014). However, no previous studies were found to have considered the optimal design of municipal water and wastewater management systems so as to minimize treatment and conveyance costs holistically, and no generalized study of the optimal scale of DPR systems was found.
The objective of the present study was to examine the generalized costs of closed-loop DPR systems as a function of treatment plant scale, assuming potentially futuristic, optimized conveyance networks, considering variations in topography, population density, and available financing. Additional sensitivity analysis was beyond scope due to computational limitations. Accordingly, an integrated model of all system components (end-users, networks, treatment plants) was developed, with results expressed in terms of the life-cycle unit cost of water for each aspect of capital and O&M, as a basis for the development of principles for the scaling of such systems. As such, the total cost is one indicator of the energy demand for a process, including energy embodied in chemicals, transportation, and human endeavor and expertise. Methods of midpoint displacement and preferential growth were adapted and implemented to generate simulated topographies and population distributions, respectively. Graph theoretical hierarchical clustering methods were used to locate treatment plants, and minimum spanning tree methods were used to layout pipe networks. Estimated costs of DPR systems assuming different levels of decentralization were computed in constant 2012 US dollars, as a basis for the optimal scaling of DPR systems without initial constraints on system layout. Initial general conclusions as to the scaling of distributed DPR systems are made. Finally, a brief case study and model demonstration are presented.

3.2 Methods and model description

A model (University of Miami, 2014) to assess DPR costs to communities as a function of the scale of individual treatment plants was programmed and executed in Matlab® version 2013a on a Pegasus 2 supercomputer at the University of Miami High
Performance Computing Services Center. A description of the overall model, including landscape generation, population placement, DPR pipeline network definition, system design and computation of costs, is presented below.

3.2.1 Landscape simulation

Generalized topographies are produced by the model using a fractal algorithm (Higham & Higham, 2005), adapted by adding a leveled random component. Accordingly, each landscape is defined by a square matrix \( Z \) with elements equal to the elevations at each corresponding point in space. The dimension of the matrix is \( 2^n+1 \), with total pixels \( N=4^n+2\cdot2^n+1 \), where \( n \) is the level number to be described, determined based on the size and resolution of study area. First, a representative number of key elevations at the points in the corners and/or the middle of the study area are fixed to represent a particular topography, i.e. landscape type. Then, generation of the matrix is completed by the midpoint displacement algorithm (Higham & Higham, 2005), which can be described as an updating of the matrix’s elements in positions \((1,d),(d,1),(d,d),(d,N)\) and \((N,d)\), where \( d=(1+N)/2 \), with the mean value of the sum of the elements on the edges plus a random noise contribution, at each iteration, as follows:

\[
\begin{bmatrix}
  a & b \\
  c & d
\end{bmatrix} \rightarrow \begin{bmatrix}
  a & \frac{a+b}{2} & b \\
  a+c & \frac{a+b+c+d}{4} & \frac{b+d}{2} \\
  \frac{2}{c} & \frac{c+d}{2} & d
\end{bmatrix} + C \cdot 2^n \cdot \text{randn}
\]

in which \( C \) is a constant, \( \text{randn} \) is a random variable with a standard normal distribution, \( \mathcal{N}(0,1) \), and the exponent, \( n \), in the term \( 2^n \) indicates that the noise has a scaling factor decreasing by 2 at each level of iteration. This process is repeated until the matrix is
complete. After adjustment of the constants $C$ and $n$, the generated matrix $[Z]$ was able to mimic the terrain of a natural landscape with given elevation difference and ruggedness (defined later in Chapter 3.2.7). All elevation information is then loaded at the beginning of the model as a fixed variable.

### 3.2.2 Population generation

A modified preferential growth algorithm (Pennock et al., 2002) applied to a two-dimensional grid is implemented in the model to generate simulated population distributions of selected population densities on the landscape matrix just described. Thus, the algorithm is roughly similar to the Game of Life (Garden, 1970), with no death mechanism. A one-story household having the US average 2.6 persons (Lofquikit et al., 2012), recorded as story degree of $S_i=1$, is set as the basic unit. Starting with $n_0$ buildings randomly placed on the $N$-pixel study area, additional one-story buildings are placed on the landscape at each time increment with a combined probability of preferential growth and baseline increase, until the desired population density is reached. The preferential element is determined by the pixel degree, $k_i$ ($0 \leq k_i \leq 8$), which equals the number of buildings on the surrounding 3x3 pixels (omitting itself in the center). At this point in the computation, $k_i$ is assigned only to empty pixels. Then the probability of placing a new building, $P_t(i)$, is expressed as the degree at this pixel, $k_i + \delta$ ($\delta$ is a constant baseline degree applied in all empty pixels), divided by the total degree across the study area, as follows:

$$
P_t(i) = \frac{k_i + \delta}{\sum k_i + \delta \cdot (N - n_t)}
$$

where $n_t$ is the number of buildings at time $t$. 

When the population density reaches a threshold of 579 /km$^2$ (1500 /mile$^2$), a second population generation process (Figure 3.1) is initiated under the premise that multi-story buildings can be constructed, in which additional floors are indicated by incrementing by unity the story degree, $S_i$, representing the number of stories in the building. In this process the preferential probability of residential unit placement based on degree $k_i$ is assigned to all pixels of the entire simulated study area. The probability of a change from $S_i$ to $S_i+1$, or of the addition of a new vertex at pixel $i$, $P_2(i)$, is thus:

$$P_2(i) = \frac{k_i + S_i}{\sum k_i + \sum S_i + \delta \cdot (N - n_t)}$$

(3.3)

for pixels with a building, and:

$$P_2(i) = \frac{k_i + \delta}{\sum k_i + \sum S_i + \delta \cdot (N - n_t)}$$

(3.4)

for pixels without a building.

Figure 3.1 An illustration of two population generation processes.

### 3.2.3 Service clustering method

Following simulation of the population distribution, an agglomerative hierarchical clustering method is initiated to determine the desired number of treatment plants within the simulated study area or, equivalently, the number of clusters of buildings each of which is served by one treatment plant. The algorithm starts with the assumption of $n_t$ clusters, which indicates each building is served by one treatment plant. At each time
increment two clusters, selected based on an optimal objective function described next, are merged to form one such that buildings in these two clusters come to share one treatment plant. The treatment plant number, i.e. the total cluster number, decreases to \( n_t - m \) after \( m \) steps. The process is continued until all clusters are merged to form a single large cluster. Accordingly, the number of clusters represents the extent of decentralization.

In the process, the metric for optimizing the formation of hierarchical clusters is defined as the Euclidean distance between pairs of observations, representing the required water conveyance distance. For example, the distance between the points (0, 0, 1) and (1, 2, 3) is calculated as \( \sqrt{(1-0)^2 + (2-0)^2 + (3-1)^2} = 3 \). Distances between each two pairs are calculated and ranked for purposes of selecting the next point to add to the cluster. Further, Ward's minimum variance method (Ward, 1963) was selected as the criterion upon which to select the next cluster to be merged. Accordingly, at each step total within-cluster variance is computed for the merger of every two possible pair of clusters, and the merger of the two clusters whose combination resulted in the minimum increase is selected. The objective function in this step is \( \sum_{i} \sum_{j} (X_{j \in i} - \bar{X}_i)^2 \), in which \( X_{j \in i} \) is vertex \( X_j \) in cluster \( i \) and \( \bar{X}_i \) is the centroid of all the vertices in cluster \( i \). In comparison with other criteria, including the average linkage, single linkage, and centroid linkage methods, all of which suppose an objection function proportional to the single power of distance, Ward’s objective function was considered most conservative and representative of realistic cost functions for DPR systems. In addition, the approach tends to create
clusters of nearly equal size, with solution sensitive to outliers (Rencher, 2002; Mooi & Sarstedt, 2011), characteristics which may be applicable in city planning.

Following the clustering process, a dendrogram, tree diagram illustrating the arrangement of clusters, is generated for the study area. The position of vertices on the dendrogram can be regarded as the extent of dissimilarity in Euclidean distance, or geographically how far apart building locations are (Figure 3.2). Because buildings that are closer may more economically share a treatment plant, two clusters having a smaller distance between them are merged sooner. In Figure 3.2, for example, building 5 and 6, and 7 and 8 are the first two pairs to be merged because the distances between them are 1, the least in the study area of 9 buildings. Thus the dendrogram can be regarded as one feasible distribution of treatment plants in the simulated study area. In order to maximize the use of gravity for the return of wastewater to the plant, treatment plants are positioned near the lowest building in the cluster, implicitly assuming no property constraints, to produce sets of arrangements of treatment plants from completely centralized to completely decentralized.
3.2.4  Pipe networks within clusters

Several studies have involved the use of virtual networks for the general design of water supply systems (Moderl et al., 2011), and for urban water management (Makropoulos et al., 2003; Sitzenfrei et al., 2010). In this work, a minimum spanning tree (MST) is used in this work to design the optimal water distribution/sewer collection network for each simulated cluster of buildings that shares one DPR treatment plant. To represent the water/wastewater conveyance network, buildings are represented as vertices and the weights of edges are set as the Euclidean distance between buildings, the same metric used in the hierarchical clustering described in Chapter 3.2.3. Current water distribution network design practice involves “looping”, or configuring mains in a loop such that water can be delivered in either direction, protecting against widespread water outages. However, in the model networks are constructed with branch design, to
minimize conveyance energy and installation costs while relying on modern trenchless repair technology to minimize outages.

Since an MST connects all vertices while minimizing the total weight of the edges, the simulated MST network will have the least total length of pipe and least total distance of water conveyance. Further, it has been proven that the MST is unique if all edges in the graph have distinct weights. Thus the algorithm for finding the MST will not affect the result in most cases. In particular, the classical Prim’s MST algorithm (Prim, 1957) was selected for use, as follows. First, the treatment plant is set as the starting point, and the edge connected to it having minimum weight is selected along with the vertex at the other end of the edge. These two vertices are said to be in the “arranged” group. After sorting the edges terminating at vertices of the “arranged” group from other vertices, the vertex with smallest weight is selected and the connected vertex is put into “arranged” group. This procedure is repeated until all vertices in one particular cluster, as determined by the agglomerative hierarchical clustering method (Chapter 3.2.3) are in the “arranged” group. In the process, the lengths of edges, representing pipelines, are also obtained.

Once the network is constructed, the water/wastewater flow rates are also calculated from the total story degree of buildings whose water supply depends on the pipe \( i \) in cluster \( j \),

\[
\sum_j S_{j(i)} , \text{ assuming each one story building has an occupancy of 2.6 persons and each person has a water demand of 0.23 m}^3 (60 \text{ gallons}) \text{ per day (Metcalf & Eddy Inc., 2013), so that } Q_i = 0.60 \times \sum_j S_{j(i)} (m^3/d).\]
3.2.5 System design

Treatment trains used for DPR were reviewed previously in (Guo et al., 2014; Englehardt et al., 2013). In the present model, a largely closed-loop treatment train comprising membrane bioreactor (MBR), electrocoagulation, peroxone oxidation sufficient for mineralization of chemical oxygen demand to below the detection limit, and chlorination is assumed for all treatment plants. While reverse osmosis (RO)-based systems are more common, an advanced oxidation-based treatment train allows implementation of DPR when disposal of RO concentrate containing endocrine-disrupting compounds, antiscalants, and excess salt is not permitted or is prohibitively expensive. Such a system produces potable mineral water at ambient temperature and pressure (Englehardt et al., 2013). If local water supplies are sufficient for blending with the reuse water, advanced treatment (e.g. peroxone or RO) treatment capacity and cost may be reduced.

Unit process capacities and water and sewer flow rates were calculated assuming average wastewater generation rates of 0.23 m$^3$/d per capita (60 gpcd, gallons per capita per day) for residential, 0.04 m$^3$/d per capita (10 gpcd) for commercial and small industry, and 0.15 m$^3$/d per capita (40 gpcd) for infiltration, for a total of 0.42 m$^3$/d per capita (110 gpcd) under generalized conditions (Metcalf & Eddy Inc., 2013). That is, the cost of onsite systems is increased to account for the commercial flow that would need to be handled by additional onsite commercial plants, and conservative in allowing for inflow at the level experienced by centralized systems. It was assumed that peroxone and chlorination tanks would be sized to provide operational, water shortage, and fire suppression storage. Also, an oversized biological treatment tank was assumed for
process flow equalization, and to accommodate sufficient primary biosolids to provide the COD needed for denitrification. In that case, applying an assumed 3.0 peaking factor to the residential flow component (Metcalf & Eddy Inc., 2013), and a normal residence time of 24 hours, MBR, electrocoagulation, and perozone unit processes were sized in the model at a capacity of 0.87 m$^3$/d per capita (230 gpcd), 2.09 times the average daily flow.

### 3.2.6 Cost estimation

Capital and O&M costs for each scenario analyzed were assessed in terms of net present value assuming plant design life at 20 years, based on reported lifespans of 30 years for centralized and 15 years for on-site wastewater treatment systems (Maurer et al., 2006; Maurer, 2009) and modern rates of technological change. Traditionally the Williams Law cost function, \( C = a \cdot Q^\beta \), where \( C \) is cost, \( Q \) is system scale (size), and \( a \) and \( \beta \) are constants, has been used in the estimation of water and wastewater treatment capital costs (Hinomoto, 1974; Tyteca, 1976; Gillot et al., 2007). More recently a logarithmic variant of Williams Law, \( \log(C) = a[\log(Q)]^\beta + k \), in which \( C \) may be either capital or O&M cost ($) and \( k \) is a constant has been found applicable over orders of magnitude of system capacity (Guo et al., 2014). Thus, capital and O&M costs are estimated using the logarithmic functions reported previously for each assumed unit treatment process (Guo et al., 2014), along with current chlorination cost data (US EPA, 2006). While capital costs reported there are based on literature references that generally include allowance for administrative costs, O&M costs include only labor. Therefore, O&M costs were increased by 20% to account for administration in the current model. Chlorine residual monitoring was assumed automated, with cost based on commercially-
available instruments (Worobetz, 2014). Accordingly, DPR treatment trains with capacities 3785.4 m$^3$/d (1 MGD, million gallons per day), 37854 m$^3$/d (10 MGD), and 378540 m$^3$/d (100 MGD) would have estimated capital costs of $21.4 million, $186.3 million, and $2170.2 million, and annual O&M costs of $1.0 million, $9.3 million, and $87.4 million, respectively, found by substituting the flow rate $Q$ into the cost functions for each unit process reported, and summing.

The capital cost of pipeline installation was estimated as follows. Assuming an average 2 m excavated depth, Tyteca (1976) estimated installation cost in Belgium as

$$C/L = K + \alpha D^\beta,$$

in which $C$ is the total capital cost ($), $L$ is the length (m), $K$ is the fixed cost with $K = 20$ in meadows, $K = 40$ in river banks, and $K = 144$ in urban areas, $D$ is the diameter of the pipe (m), and $\alpha$ and $\beta$ are parameters with $\alpha = 93, \beta = 1.681$ in meadows, $\alpha = 144, \beta = 1.197$ in river banks, and $\alpha = 180, \beta = 1$ in urban areas. However, these costs apparently do not include the installation of fire hydrants, service connections, and other appurtenances. Therefore, a current estimate for the US was obtained for a typical 0.203 m (8 inch) diameter water distribution line as $787$ per meter ($240$ per linear foot) for design, permitting, and installation (Vega, 2013). This estimate was used, along with the relationship between pipe diameter and capital cost per unit length in urban areas reported by Tetyca (1976), to obtain the function $C/L = 634.5 + 752.4D$, used to calculate the capital cost of both water and sewer lines. In Figure 3.3, these cost estimates are compared with data of Cadmus Group, Inc. (2006) and CDM Smith, Inc. (2007), indicating that the cost function used in the model, $C/L = 634.5 + 752.4D$, is generally acceptable.
Pipeline O&M costs reflect pipeline maintenance, as well as the cost of conveyance. The selection of pipe diameter, $D$ (m), is based on average flow rate in the pipe ($m^3/s$), using empirical data for water pipes and for sewer pipes separately (FlexPVC Company, 2013; Engineering toolbox, 2013). Further it is assumed that water pipes smaller than 2.54 cm (1 inch), and sewer pipes of less than 3.81 cm (1.5 inch), are not allowed by local code. The annual cost of maintaining these pipes was assumed at 4% of capital cost.

To assess conveyance energy in the present model, the elevation difference across each pipeline is assessed in all water/sewer networks. Where there is an elevation increase in the direction of wastewater flow, gravitational potential energy, kinetic...
energy, and friction losses are calculated to estimate conveyance energy. For water lines, conveyance energy is assessed similarly regardless of gradient, and accounting for the additional energy required to maintain a gauge pressure of 200,000 pascal (29 psi) to the top floors of the simulated buildings. Gravitational potential energy is calculated in both water and sewer pipes as

\[ \sum Q_i \cdot \rho \cdot g \cdot \Delta h_i \cdot t, \]

in which \( Q_i \) is the flow rate in pipe \( i \) (m\(^3\)/d), \( \rho \) is density of water (kg/m\(^3\)), \( g \) is gravity acceleration constant (N/kg), \( \Delta h_i \) is the height difference across the pipe (m), and \( t \) is the time (day). Kinetic energy is calculated as

\[ \frac{1}{2} Q_i \cdot \rho \cdot v^2 \cdot t \approx \frac{1}{2} Q_i \cdot \rho \cdot \left( \frac{Q_i}{A} \right)^2 \cdot t, \]

in which \( Q_i \) is the flow rate (m\(^3\)/s), \( \rho \) is density of water (kg/m\(^3\)), \( v \) is the average velocity in the pipe (m/s), and \( A \) is the cross-sectional area of the pipe (m\(^2\)). Friction loss energy is calculated by the Hazen-William equation:

\[ h_f = 6.82 \cdot \frac{L}{D_H^{1.17}} \left( \frac{v}{C_H} \right)^{1.85}, \]

where \( L \) is the length of pipes, roughness coefficient \( C_H \) equals 130, and hydraulic diameter \( D_H \) equals diameter \( D \) for estimation. After the energy for each pipe is calculated, a 50% wire-to-water efficiency (Roberts et al., 2011) is assumed, to assess total energy requirement. Electricity cost is assumed at the US national average rate of $0.12/kWh (US Energy Information Administration, 2013).

Little information was found on typical capital costs for pump stations. However, according to Miami-Dade Water and Sewer Department (2013b), $1.8 million in water pumping facilities capital assets was spent in 2012, and $10.7 million in wastewater pump station. Comparing to 2012 expenditures for water transmission mains, $37.7 million; wastewater force mains, $28.7 million; and wastewater gravity mains, $7.7 million, the expenditure in pump station capital assets was approximately 16.8% of those for water/wastewater mains. Thus, the capital cost for pump stations in the model is
estimated at 20% of pipeline installation.

Because on-site DPR systems offer the potential to retain thermal energy (from water heaters) in the water, the savings in residential water heating costs were considered in this study as a negative cost of providing water. While a complete and precise calculation of thermal energy retained in the MST network is beyond the scope of the current work, a simplified water/wastewater conveyance scenario can be assumed. That is, water attributed to each building is assumed conveyed via a separate constant-radius, having length equal to that in the original MST network. Fourier’s Law of heat transfer in cylindrical coordinates for 1-D conduction in the radial direction is then used to estimate the heat loss over unit distance \( q_r \) (W/m), assuming buried pipe:

\[
\frac{dQ}{dx} = \frac{2\pi k (T - T_0)}{r_1 h_1 \ln(r_2 / r_1) + \frac{k}{r_2 h_2} + \frac{k}{dh_3}}
\]

in which \( k \) is thermal conductivity of the material [W/(m*K)]; \( T \) and \( T_0 \) are temperatures of water and air (K); \( h_1, h_2, \) and \( h_3 \) are heat transfer coefficients at the interface of water and pipe, of pipe and soil, and of soil surface and air [W/(m²*K)], respectively, \( r_1, r_2 \) are inside and outside pipe radii (m), and \( d \) is the distance from the center of pipe to the surface of soil (m). If \( T_1 \) (K) is assumed equal to the initial water temperature, \( T_1 - T_0 \), the initial temperature difference, can be calculated based on the average residential hot water energy cost of $228 per household (US Department of Energy, 1998). Thus, \(~8,500 \) m is estimated as the furthest that reuse water can be conveyed before 50% of the thermal energy in the water is lost through 5.1 cm (2 inch) diameter copper pipes (Peterson Product Co., 2014) with insulation having methylene di-isocyanate (MDI)-based rigid polyurethane injected into the annulus between the service pipe and outer
casing by a one shot factory process and a thermal conductivity of 0.023W/(m*K) (Insapipe Industries Ltd., 2014) (Appendix A). In this way, an estimated savings of hot water heat, \( Q \) (W), is made in the model based on the average distance from buildings to treatment plant in the DPR systems, \( x \) (m), after integration of Equation 3.5:

\[
Q = mc_p \Delta T = mc_p (T - T_0) = mc_p (T_1 - T_0) \cdot e^{\frac{2\pi h_0 x}{\rho c_p v^2}}
\]

(3.6)

in which \( m \) is mass of water transported (kg), \( c_p \) is the heat capacity of water (J/kg), \( \rho \) is the density of water (kg/m\(^3\)), \( v \) is the average water velocity (m/s), and \( h_0 \) is the overall heat transfer coefficient [W/(m\(^2\)*K)].

3.2.7 Topographical cases

Landscapes having three different topographies ranging from flat to mountainous are simulated in the model using the midpoint displacement method. The three topographies are distinguished in particular in terms of maximum elevation difference, \( \Delta Z \), the exact value of which varies somewhat among individual simulated landscapes of each topographical type. Letting the matrix \([Z]\) represent the landscape of a study area, the maximum elevation difference, \( \Delta Z \), can be defined as:

\[
\Delta Z = \text{Max}(Z) - \text{Min}(Z)
\]

(3.7)

in which \( Z \) is the elevation that varies spatially.

In addition, the ruggedness of each simulated landscape varies somewhat, as well, and this was anticipated to potentially impact pipeline installation and conveyance costs. To define ruggedness in the present model, a measure, \( R_f \), based on the concept of information entropy (Shannon, 1948) and proposed by Malan & Engelbrecht (2009) is used. In the model, each column and row of the matrix \([Z]\) is analyzed to compute a mean
value of \( R_f \) value, as follows. First, each set of three consecutive Zs in a column or row is classified as a neutral, i.e. roughly \( Z_1 \equiv Z_2 \equiv Z_3 \), smooth i.e. approximately \( Z_1 < Z_2 < Z_3 \) or \( Z_1 > Z_2 > Z_3 \), or rugged, i.e. any other, shape. For every line analyzed, each shape is encoded as “0” if \( |Z_i - Z_j| \leq \varepsilon \), “1” if \( Z_i - Z_j > \varepsilon \), and “\( \overline{1} \)” if \( Z_j - Z_i > \varepsilon \), in which the accuracy parameter \( \varepsilon \) is an assumed fraction of the maximum elevation difference, \( \Delta Z \). The number of rugged shapes in the resulting string, \( A(\varepsilon) = a_1 a_2 a_3 \ldots a_n \) where \( a_i \in \{ \overline{1}, 0, 1 \} \), for example (“01”, “0 \( \overline{1} \)”, “1 \( \overline{1} \)”, “10”, “\( \overline{1} \) 0”, “\( \overline{1} \) 1”), is recorded as \( n_{[pq]} \) (\( p \neq q \)). The proportion of rugged shapes appearing in the string \( A(\varepsilon) \) is then defined as:

\[
P_{pq} = \frac{n_{[pq]}}{n-1}
\]

and an entropic function \( H(\varepsilon) \) is defined as:

\[
H(\varepsilon) = - \sum_{p \neq q} P_{pq} \log_6 P_{pq}
\]

The base of the logarithm in Equation 3.9 is six because there are six possible rugged shapes (Malan & Engelbrecht 2009), as listed above. Note that the value of \( H(\varepsilon) \) changes with the accuracy parameter \( \varepsilon \). Hence, considering the level number \( n \) in Equation 3.1 (described in Chapter 3.2.1), the dimension of the matrix is \( 2^n + 1 \), and the ruggedness value, \( R_f \), is defined as:

\[
R_f = \max \{ H(\varepsilon) \}, \forall \varepsilon \in \{ \frac{\Delta Z}{2^n}, \frac{\Delta Z}{2^{n-1}}, \frac{\Delta Z}{2^{n-2}}, \ldots, \frac{\Delta Z}{2}, \Delta Z \}
\]

When \( P_{pq} \in [0,1] \) the value of \( H(\varepsilon) \) in Equation 3.9 is in the range \([0, \frac{1}{e \ln(6)}]\), or approximately \([0, 0.2054]\).
To simulate flat, hilly, and mountainous topographies, representative values of \([Z]\) were obtained from elevation data for Pinecrest, FL, Berwyn, PA, and Telluride, CO, as examples of flat, hilly, and mountainous terrain, respectively (Figure 3.4 below). In these three topographies, the unit length of each pixel is set at 25 m, such that the 1025 x 1025 resolution rural study area has a total area of 25600 x 25600 m\(^2\). To maintain realistic, general areas for urban and suburban regions, and reduce computational time, the landscapes in urban and suburban cases were decreased to 1/4 of the original, resulting in 513 x 513 study areas with areas of 12800 x 12800 m\(^2\). In general, the three topographies have increasing \(\Delta Z\) values and decreasing \(R_f\) values, in the transition from flat to mountainous.
Figure 3.4 Proposed topographical cases (a) Rural, Topography 1 (Flat): $\Delta Z = 6.9$ m, $R_f = 0.1962$; (b) Urban and suburban, Topography 1 (Flat): $\Delta Z = 4.4$ m, $R_f = 0.1637$; (c) Rural, Topography 2 (Hilly): $\Delta Z = 153.0$ m, $R_f = 0.1556$; (d) Urban and suburban, Topography 2 (Hilly): $\Delta Z = 130.4$ m, $R_f = 0.1232$; (e) Rural, Topography 3 (Mountainous): $\Delta Z = 1665.8$ m, $R_f = 0.1470$; (f) Urban and suburban, Topography 3 (Mountainous): $\Delta Z = 1253.8$ m, $R_f = 0.1329$. 
3.2.8 Population density and Monte Carlo sampling

In defining urban and rural population densities, the US Census Bureau (2003) classifies an urban area as "territory, population, and housing units located within an urbanized area (UA) or an urban cluster (UC), which has: a population density of at least 1,000 people per square mile; and surrounding census blocks with an overall density of at least 500 people per square mile." In the model, the population density of rural areas was selected as similar to that of farmlands between Lancaster and Downingtown, Pennsylvania, having ~96 /km$^2$ (250 /mile$^2$), with suburban as similar to that of Berwyn, PA, ~772 /km$^2$ (2000 /mile$^2$), and urban as similar to that of the City of Philadelphia, ~3860 /km$^2$ (10000 /mile$^2$). Note that the locations are chosen due to their population densities and familiarity to the author.

Initial Monte Carlo analysis indicated that variations in topography and population placement, for a given topography and population density, had limited effect on estimated costs. Therefore, results were computed as the mean of five Monte Carlo runs. A general flow diagram for the algorithms of the model is shown in Figure 3.5.
Figure 3.5 A flow diagram of general algorithms used in the model.

3.3 Results

A depiction of one example simulated DPR treatment and conveyance network is shown in Figure 3.6 for three DPR plants serving 772 persons/km² (suburban) on Topography 2 (Hilly). As seen, the simulated networks are not realistic in terms of current practice, disregarding the locations of roads and property lines. Rather, the networks are optimized in terms of pipe length, and associated construction and conveyance cost and energy (though being subject to additional right-of-way and other costs). Such networks may eventually represent a viable alternative for new construction, given modern trenchless technologies for pipeline installation, maintenance, and repair. In fact, the UN Intergovernmental Panel on Climate Change (2014) recently noted the
crucial opportunity to slow global warming represented by sustainable approaches to new construction, given the current rapid expansion of urban areas in Asia and Africa.

![Diagram of DPR plants and pipeline network](image)

Figure 3.6 Example simulated design solution showing three DPR plants and corresponding pipeline network, found by the MST method for service of 772 persons/km² (suburban) on Topography 2 (Hilly).

Random initial population distributions sampled from uniform distributions on three topographical cases were simulated, and the model was run to assess the cost of providing DPR water, accounting for savings in domestic hot water energy, assuming 96 /km² (250 /mile²), 772 /km² (2000 /mile²) and 3860 /km² (10000 /mile²) as shown in
Figure 3.7 on semi-log axes. Costs were computed in constant 2012 US dollars in terms of GDP deflator (US Bureau of Economic Analysis, 2013), assuming O&M costs are to be drawn from annual revenue funds, that is at an assumed 0% real interest rate (available interest minus inflation). Capital costs were assumed funded by municipal bonds carrying interest at 5%, based on US data on 20-year AA rated municipal bond rates (Bondsonline, 2013). Total costs are shown in the figures, as well as costs for O&M and capital, and each component thereof for DPR systems including pipelines and treatment processes.

As shown in Figure 3.7, projected costs of DPR, assuming completely new, COD-mineralizing facilities in hilly urban areas at a scale of one plant per 10,000 homes, is ca. $15.1/1000 gallons. This cost is lower than reported for four of 16 Florida municipalities and major US cities in a survey of 2012-2013 combined water and sewer service rates by the Miami-Dade Water and Sewer Department (2013a). In addition, the costs projected with the model assume new plants with 20-year design lives, whereas most US water utilities operate facilities parts of which date back decades, and so current water bills may not reflect much of the original capital costs of the facilities. Further, the assumed advanced oxidation-based DPR technology is sized to provide mineralization of organics in terms of chemical oxygen demand to below the detection limit. Of note, safety of drinking water has many other issues, and is not discussed in detail in the study. However, likely no endocrine-disrupting pharmaceuticals are released to the environment other than in sludge, whereas significant concentrations may pass through conventional wastewater treatment plants. Costs may fall further due to the development of a mass market for advanced oxidation equipment Costs of RO-based systems may be slightly lower (Guo et al., 2014), if an inexpensive and environmentally-acceptable option for
concentrate disposal is available locally. Of course, local construction rates, selected unit treatment processes, and treatment goals will also affect the result.

In terms of the optimal scale of DPR systems, decreasing costs for pipeline installation and O&M, and savings of hot water energy, favor decentralization, while a general economy of scale in capital cost favors larger plants in urban and suburban areas as is found for current water and wastewater technology. However, accounting for a savings of ~$3.3/1000 gallons in hot water energy, the current model indicates that unit water cost does not increase significantly until plant service areas fall below 100 to 1000 homes in urban and suburban areas. In fact, averaging the optimal number of homes per treatment plant across all topographies, cost is lowest for distributed systems serving an average of 1.83, 2056, and 3333 homes per treatment plant at population densities of 96, 772, and 3860 persons/km². Such plants are small in comparison with current technology, indicating that such systems may make sense for capacity expansion projects. Counterintuitively, for flat and hilly terrain, optimal scale is indicated to be smaller in urban areas than in suburban areas. This result was attributed to the observation that shorter urban conveyance distances make hot water energy savings more significant.

Also, the cost for rural DPR systems is both high and relatively insensitive to scale. In these areas, septic systems and systems that convey nutrients back to local agricultural sectors may be logical alternatives to onsite and centralized DPR. However, note that costs shown for rural onsite and cluster systems assume system sizing adequate to accommodate commercial flow and infiltration.
As to the influence of topographical variation, topographies with a larger $\Delta Z$ value tended to have a higher projected water cost in centralized systems in general, due mainly to higher water conveyance and pipeline O&M costs, as shown in Table 3.1. However, this effect was relatively minor, appearing primarily for large plants in relatively less...
populated regions (Figure 3.8). Similarly, the effect of the ruggedness factor, $R_f$, is not strongly indicated in the current model.

![Figure 3.8 Projected cost of water at 96 persons/km$^2$ for different topographies.](image)

Table 3.1 Projected Cost of Water ($/1000$ gallons) Assuming 96 persons/km$^2$ (250 persons/mile$^2$)

<table>
<thead>
<tr>
<th>Topography</th>
<th>Pipeline installation</th>
<th>Pipe O&amp;M and water conveyance</th>
<th>Treatment plant Capital</th>
<th>Treatment plant O&amp;M</th>
<th>Total capital</th>
<th>Total O&amp;M</th>
<th>Hot water heat saving</th>
<th>Total water cost</th>
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</thead>
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<td>2.66</td>
<td>0.00815</td>
<td>5.87</td>
<td>0.0581</td>
<td>6.28</td>
<td>64.88</td>
</tr>
<tr>
<td>2</td>
<td>57.02</td>
<td>29.94</td>
<td>2.66</td>
<td>0.00815</td>
<td>5.87</td>
<td>0.0581</td>
<td>6.28</td>
<td>65.62</td>
</tr>
<tr>
<td>3</td>
<td>57.52</td>
<td>42.36</td>
<td>2.66</td>
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<td>5.87</td>
<td>0.0581</td>
<td>6.28</td>
<td>48.63</td>
</tr>
<tr>
<td>On-site</td>
<td>$\frac{1}{2}$</td>
<td>$\frac{0.22}{3}$</td>
<td>46.33</td>
<td>1.006</td>
<td>32.04</td>
<td>3.38</td>
<td>28.80</td>
<td>82.98</td>
</tr>
</tbody>
</table>

$^a$ Capacities of centralized plants vary for different population densities
As noted earlier, the influence of population density on system cost is dominant, with rural systems costing more per unit of water produced. Results for Topography 2 and different population densities are shown in Figure 3.9 and Table 3.2 as an example. Note that the differences in water costs for on-site systems are principally due to assumptions as to the prevalence of multi-story buildings in urban and suburban regions, as described in Chapter 3.2.2.

![Figure 3.9 Projected cost of water on Topography 2 with different population densities.](image-url)
Table 3.2 Projected Cost of Water ($/1000 gallons) Assuming Different Population Densities on Topography 2

<table>
<thead>
<tr>
<th>Population density (persons/km²)</th>
<th>Pipeline installation</th>
<th>Pipe O&amp;M and water conveyance</th>
<th>Treatment plant Capital</th>
<th>Treatment plant O&amp;M</th>
<th>Total capital</th>
<th>Total O&amp;M</th>
<th>Hot water heat saving</th>
<th>Total water cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centralized†</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>96</td>
<td>57.02</td>
<td>29.94</td>
<td>2.66</td>
<td>0.00815</td>
<td>5.87</td>
<td>0.0581</td>
<td>6.28</td>
<td>65.62</td>
</tr>
<tr>
<td>772</td>
<td>7.51</td>
<td>4.69</td>
<td>2.36</td>
<td>0.00705</td>
<td>6.41</td>
<td>0.0604</td>
<td>6.16</td>
<td>16.34</td>
</tr>
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<td>3860</td>
<td>0.78</td>
<td>1.13</td>
<td>1.74</td>
<td>0.00513</td>
<td>8.69</td>
<td>0.0771</td>
<td>5.97</td>
<td>11.29</td>
</tr>
<tr>
<td>On-site</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>96</td>
<td>0.22</td>
<td>0.17</td>
<td>46.33</td>
<td>1.006</td>
<td>32.04</td>
<td>3.38</td>
<td>28.80</td>
<td>82.98</td>
</tr>
<tr>
<td>772</td>
<td>0.21</td>
<td>0.16</td>
<td>45.25</td>
<td>0.967</td>
<td>31.06</td>
<td>3.24</td>
<td>28.33</td>
<td>80.74</td>
</tr>
<tr>
<td>3860</td>
<td>0.15</td>
<td>0.12</td>
<td>38.39</td>
<td>0.729</td>
<td>24.99</td>
<td>2.38</td>
<td>25.20</td>
<td>66.63</td>
</tr>
</tbody>
</table>

† Capacities of centralized plants vary for different population densities
‡ On-site systems may serve a building with several homes in urban and suburban cases

In some cases plants may be constructed using available funds, so that a zero interest rate for capital might apply. In that case, the capital cost of both pipelines and treatment plants would be 37.6% less expensive, a major savings to taxpayers, while O&M costs would remain the same. The marked effect of such differences in interest is shown in Figure 3.10 and Table 3.3, for 772 /km² and Topography 2.
Figure 3.10 Cost of water for 772 persons/km² and Topography 2, assuming 0% interest and 5% interest.

Table 3.3 Cost of Water ($/1000 gallons) at 772 persons/km² and Topography 2 assuming 0% and 5% interest

<table>
<thead>
<tr>
<th>Interest rate</th>
<th>Pipeline installation</th>
<th>Pipe O&amp;M and water conveyance</th>
<th>Treatment plant Capital</th>
<th>Treatment plant O&amp;M</th>
<th>Total capital</th>
<th>Total O&amp;M</th>
<th>Hot water heat saving</th>
<th>Total water cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centralized²</td>
<td>0</td>
<td>4.69</td>
<td>4.69</td>
<td>1.47</td>
<td>0.00440</td>
<td>4.00</td>
<td>0.0377</td>
<td>6.16</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>7.51</td>
<td>4.69</td>
<td>2.35</td>
<td>0.00705</td>
<td>6.41</td>
<td>0.0604</td>
<td>6.16</td>
</tr>
<tr>
<td>On-site</td>
<td>0</td>
<td>0.13</td>
<td>0.16</td>
<td>28.24</td>
<td>0.604</td>
<td>19.38</td>
<td>2.02</td>
<td>28.33</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>0.21</td>
<td>0.16</td>
<td>45.25</td>
<td>0.967</td>
<td>31.06</td>
<td>3.24</td>
<td>28.33</td>
</tr>
</tbody>
</table>

² Capacities of centralized plants vary for different population densities
3.4 A case study and model demonstration

Although the model was developed primarily to assess the generalized trends just presented in terms of cost versus plant scale, a case study is presented to demonstrate application to a specific case. In particular, assessment of the optimal scale of one or more DPR systems to serve the six residence halls on the University of Miami Coral Gables campus is offered as follows. First, population information for each residential hall was obtained from the University of Miami Office of Housing and Residential Life; in total the six residence halls are comprised of 17 buildings housing 4,319 residents. These are the University Village, consisting of ten buildings, Stanford and Hecht residential colleges, each consisting of two, and Eaton, Mahoney and Pearson residential colleges, each consisting of one (Figure 3.11). Then the model was run to estimate the unit cost of water, assuming from one to 17 DPR plants (from one centralized system to one onsite system for each building). Site-specific model inputs for the small study area included (1) the population matrix previously described in Chapter 3.2.2 was generated assuming that the population in each residential hall is evenly distributed within the projected area of the buildings; (2) considering the negligible variation in elevation on campus, the landscape (described in Chapter 3.2.1) was regarded as a pure flat terrain; and (3) DPR plants (Chapter 3.2.3) were located near the center of serviced buildings, assuming that these locations, more-or-less, can be accommodated during new building design. Costs would be more site-specific for retrofit of existing residence halls.
Results suggest that the cost of water is lowest for a centralized system ($10.2/1000 gallons), gradually increasing as the number of the DPR systems increases (Figure 3.12). At an overall population density of ~1800 persons/km², the study area is most similar to case (d) Suburban, with Topography 1 (Flat), in Figure 3.7, and the projected optimal scale, equivalent to one plant per 1660 home assuming 2.6 persons/household, is consistent with previous results.

To evaluate the model further, the projected costs of a 14 m³/d (3700 gpd, gallons per day) design capacity treatment plant assumed to serve a single residence hall building in the case study can be compared to flow-adjusted data obtained for an existing 1.51 m³/d (400 gpd) DPR project at the University of Miami (Englehardt et al., 2013). Costs of each unit process in the real system were individually adjusted slightly to represent a 14
m$^3$/d process, by fitting the intercepts of the cost functions presented in Guo et al. (2014) to the actual cost data. As shown in Figure 3.13, cost projections of the model presented here are indicated to be reasonable, considering that O&M costs for the real system may be more representative of a research prototype than of an optimized water utility of the future.

Figure 3.12 Projected cost of water for residential halls in University of Miami Coral Gables campus.
Figure 3.13 Comparison of the projected costs of a 14 m$^3$/d (3700 gpd) DPR plant to serve a single residence hall building with flow-adjusted actual costs of an actual DPR system serving a single residence hall apartment (Englehardt et al., 2013).

### 3.5 Summary

While most treatment processes have an associated economy of scale favoring centralization, costs of the network tend to favor decentralization. In the case of DPR, two additional factors motivate consideration of distributed systems. First, treated water conveyance back to the home would in general be upgradient. Second, energy required for residential hot water can be retained in the treated water if pipes are insulated and losses minimized.

Based on results presented here, the following conclusions can be drawn:

- The cost of water supplied with completely new, closed-loop, COD-mineralizing DPR systems as assumed in this study, at a scale of one plant per 10,000 homes, is projected at $15.1/1000 gallons, consistent with 2012-2013 combined sewer and water bills reported by Florida municipalities and major US cities. That is, these projected costs assume new facilities with 20-year design lives, whereas current water bills may reflect limited new construction. Hence the technology is likely competitive in many applications today, while offering the additional advantage of eliminating
environmental releases of endocrine disrupting pharmaceuticals in treated wastewater. Note that costs are estimated based on branched networks that optimize water and wastewater conveyance without consideration of rights-of-way. Also, costs are likely to fall further due to development of a mass market for advanced oxidation equipment;

- Costs for rural advanced oxidation-based DPR systems that accommodate commercial flow and infiltration are high, dominated by the cost of capital for pipeline installation. Septic systems and systems designed to return nutrients to local agricultural sectors may be alternatives to DPR in such areas;

- Suburban system costs are driven by a balance between pipeline and treatment equipment capital costs. Because of this dominance of capital cost, water and wastewater services could be provided at much lower cost if new facilities were purchased with cash reserves rather than financed through bond issues;

- Results of the current model did not indicate a major influence of topography on optimal size of DPR plants. However, factors such as difficulties encountered in the excavation of pipelines in mountainous regions were not able to be considered in detail; and

- In terms of approaches to the optimization of DPR system scale, the preferential growth algorithm was modified in this study for application to problems in which spatial and boundary constraints are important, and/or in which vertices are added with Euclidean definition. By this approach, weight with physical meaning is given to network edges, and vertices are added to Euclidean grids with three-dimensional topographic coordinates. Also, vertices are restricted in terms of both direction of
growth and maximum degree, but are not required to be connected initially. Finally, vertices can emerge with or without attaching to, or adding to, existing vertices, while each new vertex adds influence to all its neighbors. The proposed approach may be useful for generalized planning problems related to, e.g., in traffic management, warehouse siting, and subdivision layout, particularly in conjunction with hierarchical clustering and MST methods.

General principles for the design of DPR systems derived from this study include:

- Distributed DPR systems are projected to be competitive with centralized DPR systems, and more economical than on-site DPR systems, in urban and suburban areas as a result of savings in energy for residential hot water due to thermal energy retention, reductions in infiltration occurring in large wastewater collection networks, and reduction in upgradient distribution of treated reuse water;

- Given the potential economy of distributed DPR systems in urban and suburban areas, their gradual introduction is recommended to address municipal water and wastewater system capacity expansion requirements, particularly in new construction zones;

- The optimal scale of DPR systems is projected to range widely in rural areas (~96 persons/km²), with on-site and cluster systems generally preferred, and to range to service populations at least as small as 100 homes in suburban areas, and 1,000 residences in urban areas; and

- In strictly rural areas, systems that return nitrogen to agricultural sectors may be more economical than advanced oxidation-based DPR approaches to sustainable water management.
CHAPTER 4 MODELING THE ECONOMIC FEASIBILITY OF LARGE-SCALE DIRECT POTABLE WATER REUSE: A CASE STUDY

4.1 Introductory remarks

Recent research and studies have indicated that direct potable water reuse (DPR) systems may be viable, cost-competitive systems in urban and suburban areas (National Research Council, 2011; Guo & Englehardt, 2014). Such systems offer the promise of satisfying urban water demand without the need for treatment of toxic chemicals and pesticides found in environmental waters, while potentially addressing the accumulation of endocrine-disrupting chemicals (Englehardt et al., 2013). Further, networks of smaller-capacity, distributed DPR treatment plants, operated remotely via web-based control and maintained centrally, may be currently feasible, and may offer savings in terms of the roughly 3% of US electric power currently used for conveyance of water and wastewater (ICF consulting, 2002; Cohen et al., 2004). However, the only known true public water supply DPR implementation is currently the International Space Station, in that other implementations recycle less than 50% of wastewater, hence drawing the majority of source water from environmental sources such as reservoirs (Carter, 2009; Englehardt et al., 2013; Guo et al., 2014). In contrast, a research DPR system that recycles 85 – 90% of comingled domestic wastewater is currently being demonstrated at a University residence hall (Englehardt et al., 2013).

Essentially closed-loop DPR entails additional challenges due to the potential for the accumulation of constituents that would be diluted in a system designed to use substantial environmental water, and meeting these challenges requires additional treatment energy and expense. More generally, it is not clear on what scale DPR plants should be built in order to minimize the consumption of embodied energy, particularly in light of the need
for upgradient distribution of the treated water. In addition, while the cost of a process is an estimable indicator of its embodied energy, no comparison of DPR versus conventional water and wastewater technology has been conducted. Hence, the economics and energetics of reuse, including those of closed-loop systems, require further study in actual design scenarios.

Current water/wastewater infrastructure in Miami-Dade County is aging and in need of repair and rehabilitation. With much of the system built during the 1970’s and earlier, Miami-Dade County Water & Sewer Infrastructure Report (Miami-Dade Water and Sewer Department, 2012) assessed that deterioration has occurred principally at six sections of transmission water main, two water storage tanks and pumps stations, three major water plants, nine sections of wastewater collection and transmission line, and seven sections of wastewater pumps. Repair and rehabilitation costs are estimated at $1.1 billion in total, of which $364 million is for water projects and $736 million is for sewer projects. These capital costs will account for 10% of the total capital needs identified in the Department’s Multiyear Capital Budget over the next 15 years.

In addition to the need for rehabilitation, Chapter 2008-232, Laws of Florida, established the Leah Schad Memorial Ocean Outfall Program which requires wastewater utilities to cease disposal of treated wastewater to ocean outfalls by 2025, excepting peak flows not to exceed 5% of the annual baseline flows, and to reuse 60% of wastewater by 2025 [Section 403.086 (9), Florida Statutes and Amendment CS/SB 444]. Thus, a compliance plan by Miami-Dade Water and Sewer department (2013c) was submitted to Secretary of the Florida Department of Environmental Protection on June 28 2013. In the plan, several options were analyzed for additional wastewater treatment facilities that were required to
meet new reuse or disposal standards in accordance to the statue, and increasing flow demands. Potential impacts of sea level rise and storm surge were also evaluated, with the conclusion that existing plants are at a sufficiently high elevation to avoid inundation at 2 feet (year 2060) and 3 feet (year 2078) of sea level rise, and that current vulnerability to storm surge will not increase substantially (Miami-Dade Water and Sewer Department, 2013c). The plan involves providing 75 million gallons per day (MGD), 90 MGD peak flow, of treated wastewater to Florida Power and Light for use as cooling water at the Turkey Point power plant, and replenishing the Floridan Aquifer with an additional 27.5 MGD through deep injection wells to the “Boulder Zone”. One base option is to retrofit and expand the three existing wastewater treatment plants, and two other options involve construction of a new plant in the western part of the county, in order to reduce storm surge risks by moving substantial treatment capacity away from coastal treatment plants. While the complete decommissioning of the Central District plant and relocation of all treatment facilities to the new West District Wastewater Treatment Plant (WDWWTP) entails a high projected cost of > $7.6 billion, the recommended alternative in the plan envisions a new WDWWTP of smaller peak treatment capacity, while remaining peak flows would be directed to the North District and Central District plants, at a cost of $5.2 billion by the year 2025. This latter alternative is recommended to avoid premature investment, place treatment closer to the origin of flows, and utilize the existing ocean outfall for more cost-effective peak flow management. However, this plan is currently facing delays in securing the property for the WDWWTP.

Direct potable water reuse (DPR) represents another possible alternative to address Miami-Dade County plans for both the repair and rehabilitation of municipal water
management infrastructure, and ocean outfall legislation compliance. In fact, DPR has been recommended in general for addressing increasing water demand and replacement of aging water/wastewater infrastructure (National Research Council, 2011; Tchobanoglous et al., 2011; Englehardt et al., 2013). While the cost of DPR technology remains less certain than those of conventional systems, generalized costs of unit processes that may be used in DPR systems were reviewed recently, updated, and demonstrated as a function of system scale (Guo et al., 2014). In addition, an optimization model was presented to provide generalized principles regarding the scaling of distributed DPR systems, so as to optimize the balance between economy of scale in terms of capital cost and diseconomy of scale in terms of water conveyance energy and costs (Guo & Englehardt, 2014). Results indicate that centralized and distributed, non-reverse osmosis, advanced oxidation-based DPR systems can be competitive with centralized conventional systems, in urban and suburban areas. Further, although costs are highly site-specific, urban/suburban system cost is driven by a balance between pipeline installation and treatment equipment capital such that the optimal scale of mineralizing DPR systems is projected to range to service populations at least as small as 100 homes in suburban areas and 1000 residences in urban areas.

The objective of this study is to present an economic case study comparing the cost of centralized and distributed DPR systems, projected using the previously developed model, with costs projected in current plans of Miami-Dade County for expansion, repair, and rehabilitation of the County’s water management infrastructure, including construction of a new wastewater treatment plant, so as to achieve compliance with recent Florida ocean outfall legislation just described. The study area is in northwest Miami-Dade County, FL,
USA. To develop DPR cost projections, landscape and population distribution information for the area were obtained and input as parameters of the original optimization model. Costs are projected for a closed-loop DPR design reported previously (Englehardt et al., 2013) that produces mineral water, while mineralizing total organics, including pharmaceuticals not easily regulated in terms of environmental half-life, to below detection limits in terms of chemical oxygen demand (COD). Of note, safety of drinking water has many issues than COD, and is not discussed in detail in the study. To compare this cost with the equivalent cost of conventional water and wastewater infrastructure as planned, additional cost projections of new water treatment plants with similar treatment train to current water treatment plants in Miami-Dade County, and assumed distributed water/wastewater systems are also provided. Note that these plants are not anticipated in the current County plan, and were estimated based on previously reported cost functions for a typical unit process train. The cost of land acquisition for proposed plants, and the cost of decommissioning current water and wastewater plants, were not considered for either DPR or conventional infrastructure in the current study.

4.2 Methods: the optimization model

To estimate the cost of DPR technology as applied to the Miami-Dade County’s compliance plan to the ocean outfall legislation, the model presented previously for optimizing the cost of DPR systems as a function of treatment plan capacity was used. A detailed description of the model is presented in Guo & Englehardt (2014). Briefly, the model has the capability to generate fractal landscapes simulating actual landscapes in
real locations, and having flat, hilly, or mountainous topography. The model can then simulate urban, suburban, or rural housing distributions on the simulated landscapes, placed using a modified preferential growth algorithm. Varying numbers of treatment plants are then allocated to serve the study area by an agglomerative hierarchical clustering algorithm, and networked to buildings by minimum spanning tree methods assuming no looping of sewer lines. Capital and O&M costs are estimated based on previously published cost functions (Guo et al., 2014); conveyance energy computed based on potential and kinetic energies, friction losses, and required water line gauge pressure; pipeline installation costs; and Fourier’s Law estimates of energy savings due to retention of wastewater thermal energy. Total unit water costs are then found by amortizing capital over a 30 year planning period, summing costs in constant 2012 US dollars, and subtracting the cost of energy saved due to retention of wastewater thermal energy in the treated water. Thus, the costs of DPR systems can be projected assuming different levels of decentralization, broken down in terms of each aspect of capital and O&M. In addition, model-projected costs for conventional water and wastewater technology, assuming a simulated branched sewer network, are shown for comparison with model-projected DPR costs under the same assumption.

The closed-loop, mineralizing DPR process design assumed in the model comprises MBR, iron-mediated aeration (IMA)/sand filtration, peroxone mineralization of organics, and residual chlorination (Englehardt et al., 2013). The IMA process was described in detail previously (Deng et al., 2013; Englehardt et al., 2013). Briefly, an IMA process consisting of aerated electrocoagulation is assumed, primarily to provide aeration softening (Chao & Westerhoff, 2002), in which calcium carbonate precipitates through
equilibration with atmospheric carbon dioxide, and coagulation. Secondary benefits include phosphate precipitation and organics reduction. Projected average daily and design flows in Section 4.1 & 4.2 are based on the assumption of generalized average wastewater generation rates, as follows: 0.23 m$^3$/d per capita (60 gpcd, gallons per capita per day) for residential, 0.04 m$^3$/d per capita (10 gpcd) for commercial and small industry, and 0.15 m$^3$/d per capita (40 gpcd) for infiltration, for a total of 0.42 m$^3$/d per capita (110 gpcd), with a peak factor of 2.09 design flow to average daily flow (Metcalf & Eddy Inc., 2013; Guo & Englehardt, 2014).

Given the assumed typical infiltration rates, infiltration is expected to increase residential/commercial wastewater flow by ~57% with Northwest Miami-Dade County surficial Biscayne Aquifer water, low in sodium and chloride but high in calcium and carbonate. If lime softening and/or coagulation with iron or aluminum salts were employed, as local utilities do, sulfate and other salts would accumulate in the recycled water in the absence of reverse osmosis. In contrast, IMA precipitates excess calcium hardness regardless of the level of carbonate remaining following nitrification, resulting in a slightly-soft water (~50 mg/L Ca$^+$ as CaCO$_3$). Preliminary ionic mass balance modeling as described previously (Englehardt et al. 2013) indicated that such treatment with such infiltration would produce water of 500 – 600 mg/L total dissolved solids (data not shown). Therefore, process trains and flows as shown in Figure 4.5 (c) and (d) were assumed. Note that the process can be considered closed-loop, as no make-up water is intentionally added.

In general, the population densities and topography of the Miami-Dade County study area are similar to the generalized urban/suburban cases with “flat” topography presented
previously. However, many of the parameters, including population density and distribution, physical layout of the area, assumed treatment technologies, and other factors are site-specific. Hence a description of the model parameters and settings used to simulate costs for the area served by the WDWWTP, reflecting corresponding topographical, population, and housing data, is presented in this section.

4.2.1 Landscape data

The new WDWWTP has an estimated site size of 0.99 km$^2$ (245 acre), and is located west of NW 137 Ave at NW 6 St in Miami, FL, USA. An original map of the proposed service area was provided by the Miami-Dade Water and Sewer Department (Fallon, 2014), see Figure 4.1 below. The service area of the plant, corresponding to the study area for this work, ranges from SW 72 St to NW 90 St in the south-north direction, and 167 Ave to 42 Ave in the east-west direction, estimated at approximately 244.7 km$^2$ (94.52 mile$^2$). Note that the population to the northeast, east, and south of the proposed service area will be served by the North District, Central District and South District Wastewater Treatment Plants, respectively, and these other service areas are not considered in this study.
Figure 4.1 Proposed serviced area of WDWWTP, outlined by black dashed lines, with the locations of the WDWWTP indicated by red pentagon, and North District, Central District and South District plants by blue pentagon. Figure provided by courtesy of Miami-Dade Water and Sewer Department (Fallon, 2014).
In order to simulate the topography of the study area, elevation data for 9 x 9=81 key locations across a grid covering (and extending beyond) the study area were obtained from Google Earth® software. From these a fractal landscape was simulated using an adapted fractal midpoint displace algorithm (Higham & Higham, 2005; Guo & Englehardt, 2014). This landscape was represented by a matrix \([Z]\) whose elements equal to the elevations at each corresponding point in space. Note that, as indicated in in Google Earth®, some streets (and avenues) are not strictly parallel, whereas in the model streets and avenues are all taken as orthogonal when projected to sea level (Figure 4.2). Also, the original map provided by Fallon (2014) was found to be slightly re-scaled in terms of length/width ratio as received. Therefore, all distances measured were verified versus Google Earth®. All other measurements, including the distance between buildings and streets discussed in Chapter 4.2.2, were approximated with best engineering judgment.
Figure 4.2 Proposed service area of WDWWTP: (a) Partial image from map provided by Miami-Dade Water and Sewer Department (Fallon, 2014); (b) Google Earth® image with boundaries accurate to respective streets and avenues according to (a); (c) Simulated landscape of the service area based on measurements of (b).
4.2.2  Population and housing data

Residential structures were simulated and distributed across the simulated study area with three-dimension topographic coordinates using the modified preferential growth algorithm described previously (Guo & Englehardt, 2014). To define the resolution of the simulated study area, the distance between homes across the actual study area were estimated using the ruler function in Google Earth® at 250 m height eye view. At least six categories of building spacing were observed, as shown in Figure 4.3. Of those, the 25 m x 25 m spacing shown in Figure 4.3 (c) was selected as the basic unit cell used in the model. Thus, between every eight streets, roughly measured in both Google Earth® software and via automobile odometer as ca. 800 m (0.5 mile), up to 32 rows of buildings may be placed in the model, and between every five avenues, measured similarly at ca. 800 m (0.5 mile), a maximum of 32 columns of buildings may be placed. Thus in the model, the study area is divided into blocks, 800 m x 800 m (0.5 mile x 0.5 mile) square, to ensure that the simulated population is in accordance with housing and population data reported by the US Census Bureau (2013) with accuracy to “Census Block Group” level (Table 4.1). Census Blocks of irregular shape reported in the data of US Census Bureau (2013) were merged to form larger rectangular blocks, with housing and population numbers represented as a whole.
Figure 4.3 Different neighborhood layouts in the service area of the WDWWTP near: (a) SW 143 Pl and SW 35 St; (b) SW 136 Ave and SW 1 St; (c) SW 112 Pl and SW 49 St; (d) SW 114 Ct and SW 43 St; (e) Alhambra Cir and Baracoa Ave; (f) SW 56 Ave and SW 6 St.
Table 4.1 Data on Population (at the top of cells with yellow background) and Housing (bottom) in the WDWWTP Service Area in 2010 with Accuracy to “Census Block Level” (US Census Bureau, 2013). Total population: 529,098. Total housing: 185,222.

| Population | 167 | 162 | 157 | 152 | 147 | 142 | 137 | 132 | 127 | 122 | 117 | 112 | 107 | 102 | 97 | 92 | 87 | 82 | 77 | 72 | 67 | 62 | 57 | 52 | 47 | 42 | 37 | 32 | 27 | 22 | 17 | 12 | 7 | 2 |
|------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|

| Housing | 167 | 162 | 157 | 152 | 147 | 142 | 137 | 132 | 127 | 122 | 117 | 112 | 107 | 102 | 97 | 92 | 87 | 82 | 77 | 72 | 67 | 62 | 57 | 52 | 47 | 42 | 37 | 32 | 27 | 22 | 17 | 12 | 7 | 2 |
|---------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|

### Notes
- The table demonstrates the distribution of population and housing across various blocks in the WDWWTP Service Area.
- The data is based on the 2010 U.S. Census, providing accuracy at the Census Block Level.
- The total population in the service area is 529,098.
- The total number of housing units is 185,222.

### Data Analysis
- The table highlights the density of population and housing across different blocks, indicating areas with higher concentrations.
- The data is crucial for urban planning, resource allocation, and community decision-making processes within the service area.
As shown in Table 4.1, the total population in the service area in 2010 was 529,098 with 185,222 housing units, averaging 5597.9 persons/mile$^2$ and 2.856 people per household, which is higher than the US average number of 2.6 (Lofqusit et al., 2012). Also, population is distributed across the service area quite unevenly. While most areas between NW 12 St and W Flagler St are very crowded, Census Blocks near the Miami International Airport, Tropical Park, and the area south of Kendall Regional Medical Center have low population densities. In addition, based on a projected population increase from 2,496,435 to 3,172,406 for Miami-Dade County from year 2010 to 2035 (Miami-Dade Water and Sewer Department, 2013c; US Census Bureau, 2014), the population of the service area was assumed in the model to grow similarly by a factor of 1.27. In simulating housing unit distribution, the number of people in a household, i.e. the basic population unit in the model, was set at 2.856. Then, the preferential growth process was terminated when the number of housing units in each Census Block reached the projection, which was 1.27 times the corresponding number in Table 4.1. In this way the model guaranteed a reasonable total population projection for 2035, retaining the general population distribution of 2010. In total, the model generated a simulated population of 673,219, residing in 235,721 simulated housing structures, very close to the County projection of 671,823 people in 235,232 housing structures. Additionally, five different projections, all based on the population information in Table 4.1, were simulated to assess variability in the result. One of these housing structure projections is shown in Figure 4.4.
Figure 4.4 Example simulated population distribution across the service area of the WDWWTP in a 3-D view, in which each blue point represent a residential building.

4.3 Cost function verification

Before running the model, the cost functions for wastewater treatment unit processes embedded in the model (Guo et al. 2014; Guo & Englehardt, 2014) were used to estimate the capital cost of the proposed new WDWWTP, and compared with the current cost estimate obtained by the County, for verification of the cost functions as possible for application in the Miami-Dade County study area. The County proposes to construct an 153 MGD MBR plant to treat normal municipal wastewater flow, with a parallel biological aeration process to be activated to address peak wet-weather flows of up to 62 MGD. During parallel operation, both flows are combined to pass through the 215 MGD
capacity chlorination process. As a note, the County plan also calls for the ability to transfer up to an additional 10 MGD, or 5% of total treatment capacity, to the North District plant for treatment and disposal; this small excess peak flow is neglected in this study.

The average daily flows and design capacities of components of the proposed WDWWTP are shown in Figure 4.5 (a) as reported by Miami-Dade Water and Sewer Department (2013c), together with an assumed single new water treatment plant of 386,112 m³/d (102 MGD) treatment capacity. Principal unit wastewater treatment processes include pre-treatment comprising coarse screen, grit/grease removal, and fine screening (Côté et al., 2006), MBR, Cl₂ contact, odor control, and clarification. The smaller, 62 MGD, part-time peak flow plant comprises aeration and ballasted high-rate flocculation (a high rate chemical/physical process which may include several continuous tanks for coagulation, flocculation and clarification) (Tetra Tech Inc., 2013). Other projected costs include class AA biosolids improvement for production of agricultural-quality sludge product (Florida Department of Environmental Protection, 2014), deep injections wells for disposal of disinfected treated effluent (up to 205.8 MGD); Floridan aquifer recharge wells for reuse and disposal of advanced-treatment effluent (up to 9.2 MGD); on-site Cl₂ generation for effluent disinfection; and thickening, dewatering and digestion facilities for sludge reuse.
Figure 4.5 Unit treatment processes and flows for (a) the current County plan with proposed new water treatment plant; (b) central DPR to treat 215 MGD with disposal of excess treated effluent for surficial aquifer recharge; (c) central DPR to treat 156.1 MGD with disposal of excess treated effluent, assuming reduced infiltration and separate treatment of excess commercial and wet weather flow; (d) distributed DPR to treat 156.1 MGD with disposal of excess treated effluent, assuming reduced infiltration and separate treatment of excess commercial and wet weather flow; (e) central water/wastewater system assuming reduced flow. Average daily flow is shown above process connector arrows, with design capacity including peak flow shown below.
For the wastewater treatment system proposed, excluding deep injection wells and aquifer recharge facilities, a total budget of $1.64 billion from July 2013 to December 2024, including $1.31 billion for the initial treatment plant, $182.6 million for the peak flow treatment, and $150.7 million for biosolids AA improvement, is projected by Miami-Dade Water and Sewer Department (2013c). The budget for the principal wastewater treatment plant includes the construction fee ($1.2 billion from February 2021 to December 2024) and design fee ($97 million from March 2017 to September 2019) as major components, whereas land acquisition ($15.6 million from August 2013 to August 2014) and permitting ($4 million from April 2014 to February 2017) are relatively minor. To be noted is that the overall cost of the plant is reduced by the inclusion of a second, much smaller plant for peak flow treatment.

A net present value was estimated for a system similar to that proposed by the County, using the literature information (Côté et al., 2006; US EPA, 2006; Guo et al., 2014) that formed the basis of simulated costs of DPR systems. That is, an MBR treatment plant of the same average daily flow, 386,112 m³/d (102 MGD), with pretreatment, a 234,696 m³/d (62 MGD) ballasted high-rate clarification process for which capital cost was roughly estimated based on the cost of three flocculation tanks, and a Cl₂ contact basin, were assumed. Total cost was estimated at ca. $874 million in 2012 US dollar. Assuming an inflation factor of 1.28 from 2012 to 2024 US dollars based on the GDP deflator in 2012 and the projected value in 2024 (US Bureau of Economic Analysis, 2013; World Bank, 2014), the total cost would be $1.45 billion in 2024 US dollars, for a system including the same peak flow treatment and biosolids AA improvement processes. In general, this total cost is quite close to the total budget in the County’s projection, $1.64
billion. In terms of detailed cost components, the County projects the MBR process with a 1.5 peaking factor at a cost of $1.08 billion, while the estimate for an MBR process with 2.09 peak factor based on literature review is $964 million; the County projects a chlorine contact basin at $39.5 million and an on-site chlorine generation system at $56.4 million, while literature-based estimate assumes a chlorination system at $50.0 million in total. Such differences may stem from use by the County of a constant contingency of 58.6% for all processes, accounting for process control, field office, yard piping, site preparation, roadway, electrical distribution, sitework, engineering, survey, construction management, design, pilot study, permit, inspections, etc. (Coro, 2014), while literature-based functions assume a contingency of 30% for design and administration of MBR construction (Decarlolis, 2007; Guo et al., 2014) and ~18% for piloting and permitting of chlorination facilities (US EPA, 2006).

4.4 Results

The costs of centralized and distributed closed-loop, mineralizing DPR systems, assuming model parameter settings as described in Chapter 4.2, with payment over 30 years, are assessed and compared with the total cost of the proposed WDWWTP and associated water system in this section.

4.4.1 Simulated costs for a single DPR system

The projected cost of a centralized DPR system to serve the study area, with plant built on the location of the proposed WDWWTP, is listed in Table 4.2, assuming capital cost financed by normal 30-year municipal bonds at a 5% real annual interest rate. The
estimated total capital cost for a single DPR plant in the study area is ~$3.36 billion if paid in cash in 2012 US dollars. From Table 4.2, the projected total capital cost for a single DPR plant is $9.05/1000 gallons, and the total unit cost of water is $15.16/1000 gallons, acceptable for a closed-loop potable water reuse process that mineralizes organics.

Table 4.2 Projected Water Cost For DPR Plants, Assuming 30-year Municipal Bonds At A 5% Real Annual Interest Rate. ($/1000 gallons in 2012 US dollars)

<table>
<thead>
<tr>
<th>Number of DPR plants</th>
<th>Pipeline installation</th>
<th>Pipe O&amp;M and water conveyance</th>
<th>Treatment plant Capital</th>
<th>Total capital (with biosolids improvement)</th>
<th>Total O&amp;M</th>
<th>Hot water heat saving</th>
<th>Total water price</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.96</td>
<td>0.67</td>
<td>1.29</td>
<td>9.05</td>
<td>6.11</td>
<td>0</td>
<td>15.16</td>
</tr>
<tr>
<td>46</td>
<td>0.95</td>
<td>0.65</td>
<td>2.78</td>
<td>7.98</td>
<td>6.79</td>
<td>-1.78</td>
<td>13.00</td>
</tr>
</tbody>
</table>

4.4.2 Simulated costs for distributed DPR systems

The model was rerun to estimate costs for DPR service areas of all sizes, ranging from one centralized plants for the study area to on-site systems (one per building), as shown in Figures 4.6. The cost of providing water is assessed to be minimized for the case of 46 DPR systems serving the study area, i.e. at an average size of one plant per 4,810 homes (Figure 4.7). This optimal system size is relatively small compared to current water/wastewater systems. In total, the capital costs of all 46 DPR plants is projected to be ~$2.92 billion in 2012 US dollars, with component costs also listed in Table 4.2. Compared with a centralized system, the cost of 46 distributed DPR systems is projected to save money in terms of both capital cost for treatment plants and pipeline installation, and in terms of hot water energy savings as well.
Figure 4.6 Projected water cost for distributed DPR systems ($/1000 gallons in 2012 US dollars), assuming 5% annual real interest rate for capital cost and 30-year payment.
4.4.3 Cost projections for proposed WDWWTP plus simulated associated water system

In this section, a total cost of the proposed WDWWTP, based on according to Miami-Dade Water and Sewer Department (2012; 2013c), along with the estimated cost of a hypothetical associated water treatment plant for the study area, is assessed for comparison with costs estimated in the previous section for DPR systems. As calculated in Chapter 4.3, the total capital cost for the proposed WDWWTP without injection and recharge wells is $1.64 billion. When considered as a practical alternative to DPR, the total capital cost of the treatment plant would be $2.02 billion, including $118 million
injection well pump station, $259 million injection wells, and $25 million Floridan Aquifer Recharge. Also, there will be $500 million cost for all related pipeline upgrades and new pump stations in the WDWWTP service area (marked as CL-3, CL-4, CL-5, CL-6, CL-7, CL-8, CL-X, WP-1, CP-187E in Figure 4.1, total cost $41,370,000 + 157,380,000 + 30,730,000 + 16,370,000 + 99,580,000 + 5,000,000 + 100,000,000 + 50,000,000 = $500,430,000).

In addition to upgrade of wastewater service, water treatment plants in the proposed service area of WDWWTP are also in need of upgrades, repair and rehabilitation, to avoid further system failures that may result in loss of life or extended loss of service (Miami-Dade Water and Sewer Department, 2012). As shown in Figure 4.8, the study area overlaps parts of several County commissioners’ districts (Districts 6, 7, 10, 11, and 12). As reported by Miami-Dade Water and Sewer Department (2012) and with reference to Figure 4.8, estimated costs of upgrade, repair and rehabilitation of water pipelines and water plants nearby also add to the $2.02 billion cost of the WDWWTP, resulting in a total capital cost of water and wastewater treatment plants in the study area of $2.06 billion [2,018,215,307 (capital cost of WDWWTP) + 37,300,000 (estimated upgrade cost of Highleah Water Plant and John E. Preston Water Plant) + 1,000,000 (estimated upgrade cost of Alexander Orr, Jr. Water Plant) = $2,056,515,307]. In addition, the total capital cost for water and sewer pipelines would be 510 million, including the original $500 million for sewer pipes, proposed new water pipes in District 12, and water storage tanks and pipes in District 11 (500,430,000 + 4,850,000 + 4,530,000 = $509,810,000). Note that these costs and treatment capacities for water plants whose service areas extend beyond the boundaries of the study area are estimated based on best judgment.
Figure 4.8 Proposed new water tanks and transmission mains, and commissioners’ districts related to the study area, which has been outlined by black dashed lines. Figure provided by courtesy of Miami-Dade Water and Sewer Department (2012).

4.4.4 Comparison

In this section, the capital cost of treatment plant and pipeline networks for different alternatives, including construction of the WDWWTP with associated water and sewer pipelines upgrades; construction of a new DPR plant at the current location of the WDWWTP and associated pipelines; and construction of distributed DPR systems with pipe networks as needed, are compared and discussed. For convenience of comparison, a summary of the information presented in Chapter 4.3 and 4.4.3, Chapter 4.4.1, and Chapter 4.4.2, are listed in Columns 2, 5 and 6 of Table 4.3, respectively. In Table 4.3, all capital costs are listed as a net-present value in 2012 US dollars, and all unit water costs
reported for capital investments assume 30-year municipal bonds at 5% real annual interest. Capital costs of DPR include class AA biosolids improvement estimated based on Miami-Dade Water and Sewer Department (2013c). In addition, the sum of the capital cost projected by the county for construction of the proposed WDWWTP, together with the estimated cost of an assumed single new water plant of 386,112 m³/d (102 MGD) treatment capacity with treatment train consisting of feed tank, lime addition, clarifier, re-carbonation, rapid sand filtration, chlorination, ammoniation and fluoridation, and clearwell [Fig. 5 (a)], is also shown in Column 3 of Table 4.3, calculated based on unit process cost functions and information reported by US EPA (2003; 2006) and Guo et al. (2014).

Note that the difference in the assumed capacities of the DPR plants and the proposed WDWWTP is due principally to the assumption of typical average wastewater generation rates totaling 0.42 m³/d per capita (110 gpcd) for generalized conditions (Metcalf & Eddy Inc., 2013; Guo & Englehardt, 2014). For the projected service population (673,219) and an assumed 2.09 peaking factor, a design peak flow of 156 MGD was further estimated. In contrast, the County used Innovyse Infoworks® CS software to develop a much more detailed hydraulic model of the system, with consideration of flow in every major line and of actual pump curve data for each pump station (Fallon, 2014), resulting in their projected design peak flow (215 MGD), as described in Chapter 4.3. Note that the effective overall peaking factor of 2.10 resulting from County projections is quite similar to the factor 2.09 used in the model. However, the peak flow projected by the County is 38% higher than estimated for generalized conditions (Metcalf and Eddy Inc., 2013). Therefore, the capital cost of a centralized DPR plant that has the same average flow
(386,122 m$^3$/d, 102 MGD) and peak capacity (813,864 m$^3$/d, 215 MGD) of the proposed WDWWTP [Figure 4.5 (b)] is also offered Column 4 of Table 4.3.

Because costs for all DPR alternatives are estimated assuming branched water networks, while the pipeline upgrades of WDWWTP-related alternatives (Columns 2 and 3) assume current looped water networks, and because the current implementation of the model is unable to simulate looped sewer networks, costs of conventional central water/wastewater systems approximating the current infrastructure, though with branched sewer networks, and assuming reduced infiltration and separate treatment of excess wet-weather and commercial flows, are shown in Column 7 of Table 4.3 for comparison. The treatment train assumed is shown in Figure 4.5 (e). These costs are estimated by the model, with the exception of capital costs for aquifer recharge and deep injection wells, which are based on information in Miami-Dade Water and Sewer Department (2013b) and adjusted proportional to capacity. Note that, given the high cost of multiple injection wells and the reliance of current water plants on well-fields near to Everglades-derived source water, decentralized water/wastewater systems are not considered a realistic alternative and are not considered here. In addition to capital costs, O&M costs are listed in Table 4.3 for estimation of the total unit water cost associated with each alternative. The O&M costs for the WDWWTP were obtained from the County (Coro, 2014). All other O&M costs are projected with the model, calculated based on unit process cost functions and information reported previously (US EPA, 2003 and 2006; Guo et al., 2014).

As can be seen in Table 4.3, mineralizing DPR appears to be a more expensive alternative when compared with the current plan of Miami-Dade County for compliance
with recent ocean outfall legislation, which centers on deep-well injection of disinfected treated effluent. Also, the costs of both DPR and conventional water and wastewater systems are greatly reduced when reduced peak flow is assumed, economy of scale notwithstanding. However, it should be noted that 12% of treatment capital cost and 14% of the total cost of providing DPR water can potentially be saved through consideration of distributed systems. Furthermore, the total cost of providing DPR water is projected as low as $13.00/1000 gallons, approximately 51% higher than that of the current plan according to County’s projection (Miami-Dade Water and Sewer Department, 2013c; Coro, 2014), and lower than costs reported for several major US cities and Florida municipalities (Miami-Dade Water and Sewer Department, 2013a).
Table 4.3 Comparison of Different Alternatives of Water and Wastewater Solutions (capital costs contain AA biosolids improvement, all costs are in constant 2012 US dollars)

<table>
<thead>
<tr>
<th>Plan</th>
<th>WDWWTP + current water plants upgrade (MDWASD)</th>
<th>WDWWTP + new water plant (MDWASD + model)</th>
<th>Centralized DPR plant (model)</th>
<th>Centralized DPR plant (model)</th>
<th>46 distributed DPR plants (model)</th>
<th>Central water and wastewater plants (model+MDWASD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corresponding treatment processes and flows</td>
<td>Figure 4.5 (a)</td>
<td>Figure 4.5 (a)</td>
<td>Figure 4.5 (b)</td>
<td>Figure 4.5 (c)</td>
<td>Figure 4.5 (d)</td>
<td>Figure 4.5 (e)</td>
</tr>
<tr>
<td>Design average flow (m³/d)</td>
<td>~386,112</td>
<td>386,112</td>
<td>386,112</td>
<td>282,752</td>
<td>282,752</td>
<td>282,752</td>
</tr>
<tr>
<td>Design peak capacity (m³/d)</td>
<td>Parallel system, up to 813,864 m³/d</td>
<td>Parallel system, up to 813,864 m³/d</td>
<td>813,864</td>
<td>590,952</td>
<td>590,952</td>
<td>590,952</td>
</tr>
<tr>
<td>Water reuse capacity (m³/d)</td>
<td>34,826 (recharge to Floridan Aquifer)</td>
<td>34,826 (recharge to Floridan Aquifer)</td>
<td>386,112</td>
<td>282,752</td>
<td>282,752</td>
<td>25,503 (recharge to Floridan Aquifer)</td>
</tr>
<tr>
<td>Effluent disposal capacity (m³/d)</td>
<td>Up to 779,038 (deep injection)</td>
<td>Up to 779,038 (deep injection)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Up to 570,494 (deep injection)</td>
</tr>
<tr>
<td>Wastewater treatment plant capital cost ($)</td>
<td>1,573,043,887</td>
<td>1,573,043,887</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>776,262,451</td>
</tr>
<tr>
<td>Water treatment plant capital (upgrade or construction; $)</td>
<td>30,157,480</td>
<td>66,275,595</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>49,312,705</td>
</tr>
<tr>
<td>Total treatment capital or DPR plant(s) capital cost</td>
<td>1,603,201,367</td>
<td>1,639,319,482</td>
<td>4,833,554,205</td>
<td>3,363,508,152</td>
<td>2,922,801,274</td>
<td>825,575,156</td>
</tr>
<tr>
<td>Pipeline upgrade or installation ($)</td>
<td>401,425,196</td>
<td>605,428,164</td>
<td>408,005,937</td>
<td>400,360,557</td>
<td>396,190,134</td>
<td>400,360,557</td>
</tr>
<tr>
<td>Total Capital ($)</td>
<td>2,004,626,563</td>
<td>2,244,747,646</td>
<td>5,241,560,142</td>
<td>3,763,868,709</td>
<td>3,318,991,408</td>
<td>1,225,935,713</td>
</tr>
<tr>
<td>Unit treatment capital cost ($/1000 gallons)</td>
<td>2.81</td>
<td>2.87</td>
<td>8.47</td>
<td>8.09</td>
<td>7.03</td>
<td>1.98</td>
</tr>
<tr>
<td>Unit pipeline capital cost ($/1000 gallons)</td>
<td>0.70</td>
<td>1.06</td>
<td>0.72</td>
<td>0.96</td>
<td>0.95</td>
<td>0.96</td>
</tr>
<tr>
<td>Total O&amp;M including hot water heat saving ($/1000 gallons)</td>
<td>5.10</td>
<td>5.10</td>
<td>6.67</td>
<td>6.11</td>
<td>5.02</td>
<td>4.67</td>
</tr>
<tr>
<td>Total unit cost ($/1000 gallons)</td>
<td>8.61</td>
<td>9.03</td>
<td>15.86</td>
<td>15.16</td>
<td>13.00</td>
<td>7.62</td>
</tr>
</tbody>
</table>

a Cost estimated based on literature review.
b Cost estimated with the model assuming DPR options with flows shown in Fig. 5.
c Cost estimated assuming upgraded sewer pipelines according to the current WDWWTP plan, and new water pipelines.
4.5 Summary

The DPR technology assumed and evaluated in this study has sufficient advanced oxidation capacity to mineralize organic constituents to below detection in terms of COD, as one strategy towards elimination of disinfection byproducts and to address the accumulation of endocrine disrupting compounds in the environment. The additional capacity, particularly in terms of ozonation equipment, entails additional cost. In addition, effectively 100% recycle is assumed. Hence, the process may be expected to achieve higher levels of water conservation and environmental protection, at higher cost relative to existing DPR implementations. Ultimately decisions will be made as to whether such investment in equipment and pipeline networks is worth the benefits. The analysis presented is intended to provide further information relating to the implementation of DPR, rather than to support design of the current plant expansion, as site-specific design decisions will depend heavily on additional factors such as utilization of existing pipe networks and right-of-ways. In that light, this study provides an initial evaluation of economic feasibility through a case study of a large-scale hypothetical implementation using a model recently proposed.

In general, the following conclusions can be drawn from this study:

- Cost functions presented previously as a part of this work are able to project the cost of DPR unit processes within 20% of the County’s projection for the study area;
- Mineralizing DPR technology is projected to be feasible for this case study at a capital cost approximately double that of the current plan of the Miami Water and Sewer Department;
• If DPR were to be considered for this infrastructure rehabilitation project, results indicate that, subject to property acquisition considerations, a network of approximately 46 DPR systems may be most cost-effective; and

• The total unit cost of providing DPR water to the service area of the Miami-Dade County WDWWTP is projected as low as $13.00/1000 gallons, assuming 46 distributed treatment plants in the service area. This cost is approximately 51% higher than that of the current plan according to County’s projection, and less than that of several major US cities and Florida municipalities.

Current limitations of the study include:

• Cost projections for DPR alternatives assume branched networks that optimize water and wastewater conveyance without consideration of rights-of-way. Although corresponding projections for water and wastewater systems with branched system are offered, cost estimation for systems with looped water mains is needed.

• The DPR technology assumed in this study is under development as a means to provide nearly 100% closed-loop municipal water reuse. Work is needed to confirm that organic constituents such as pharmaceuticals in the effluent are effectively mineralized. Further demonstration of such non-reverse osmosis-centered systems is also recommended to confirm consistent production of water meeting all drinking water standards and aesthetic considerations.
CHAPTER 5 CONCLUSIONS

Regarding questions of economic feasibility of direct potable water reuse (DPR) and optimal scale of DPR systems, results presented in this dissertation suggest that DPR cost is competitive with current water and sewer technology. Results also suggest that distributed DPR systems can be gradually introduced in the expansion of municipal water and wastewater system as a practical water management alternative to address the increasing water demand in the future.

The work presented here included the identification of a new and general law relating cost to process capacity over orders of magnitude. Specifically, a logarithmic variant of Williams Law is introduced, and used to provide general cost information for the estimation of unit water cost of various treatment capacities of DPR systems. This law is presented and demonstrated in Chapters 2 & 3. Available costs related to the capacities or flow rates in all components of DPR systems are reviewed from various literatures and current companies’ quotes, updated to constant US dollars and then synthesized into cost functions if applicable. The information includes 1) a review of cost versus scales for seven major water reuse technologies; 2) a review of capital cost of water/sewer pipeline installation in terms of pipeline diameter; 3) other cost estimation needed, e.g. water and sewer conveyance energy, capital cost of pump stations, operation and maintenance (O&M) cost of DPR treatment plants, cost saving in heating residential hot water. Such cost information may be useful to researchers in the design of DPR treatment processes, and provides a basis for further studies of evaluation of DPR systems.

The work presented also resulted in the development of the first model to simulate cost as a function of treatment system capacity, for scaling of DPR systems. The network model is constructed based on the concept of fractal topography, modified preferential
population growth, and graph theoretical agglomerative hierarchical clustering and minimal spanning tree (MST) methods as described in Chapter 3. Compared with previous optimization methods such as simulated annealing and genetic algorithms, the combination of these approaches does not require an initial solution to the problem, and would provide a much faster convergence in the estimation while guaranteeing that the solutions are globally optimal. In addition, the modification to the preferential growth model has made it useful for other similar researches in city planning, e.g. traffic management, warehouse siting, and subdivision layout.

The reliability of the model is further confirmed in a case study in Miami-Dade County, FL, USA in Chapter 4. Overall, the model is used to analyze DPR treatment alternatives for a 244.7 km² (94.52 mile²) area in an irregular shape, with a water demand totaling 386,112 m³/d (102 MGD) for a population of more than 670,000 persons. While previous conclusions regarding the cost of DPR systems in terms of scale are generally confirmed, Chapter 4 results also reveal that such large-scale DPR plants with complete mineralization of COD in wastewater would be at the expense of approximately twice the capital cost of the currently-proposed ocean outfall legislation compliance plan. Nevertheless, the total unit cost of providing DPR water to the service area of the Miami-Dade County WDWWTP (O&M cost not included) is projected as low as $13.00/1000 gallons, assuming 46 distributed treatment plants in the service area and 300 GPD per home. This cost might be ~51% higher than those of the current plan, and still lower than current costs in many cities and municipalities. Also of note, such a Miami-Dade County DPR implementation would represent the largest DPR installation ever, if DPR technology were fully applied (nearly three times the size of the Orange County project,
the largest one in Table 2.2, considering peak rainfall flow in Miami-Dade County). For a complete cost comparison of these DPR cases, much more cost information for a complete water/wastewater cycle would be needed.

Principal limitations of the current work are three. First, although the MST method used in the model implies an optimal water/sewer network with least total length of pipes and least distance of water conveyance, the simulated network is a branch system in terms of both water distribution and sewage collection, whereas current water networks (not sewer) are typically looped to provide higher reliability. In fact, water and sewer lines for DPR systems of the future may likely share rights-of-way, so that reliability may be increased by new means including trenchless water and sewer line technology. However, comparisons with current costs that include looped sewer networks should be interpreted with care. Second, while the objective function of Ward’s minimum variance method employed in the hierarchical clustering process of this study uses the classical square of Euclidean distance for conservative estimation, the actual power may not be an integer, and may depend upon the general relationship between the cost of a DPR system and the size of its service area. Thus, there is a limitation to the application of the Ward’s classical method. Székely & Rizzo (2005) extends this classical approach, by defining a cluster distance and an objective function defined in terms of any power of the Euclidean distance between zero and two. This technique would make it possible for a more optimized clustering method, once there is enough research to suggest a general function relating the cost of DPR systems to the Euclidean distance from buildings to treatment. Third, the DPR technology assumed in this dissertation is under development as a means to provide nearly 100% closed-loop municipal water reuse, while work is still needed to
confirm that organic constituents such as pharmaceuticals in the effluent effectively mineralized. Further demonstration of such non-RO-centered systems is recommended to confirm consistent production of water meeting all drinking water standards and aesthetic considerations.

Conclusions regarding cost in current DPR system projections include:

• The unit cost of water, including capital and OM, assuming new, COD-mineralizing DPR plants in urban areas (3860 persons/km²), is projected at ~$13.20/1000 gallons assuming a 30-year equipment design-life and planning period, or ~$15.10/1000 gallons assuming a 20-year equipment lifetime, at a scale of one plant per ca. 10000 homes. This cost is competitive with current combined water and sewer bills in US major cities, and in fact major facilities may last longer than 30 years, reducing cost further;

• Large scale of mineralization DPR plant (~378,500 m³/d, or 100 MGD) in urban and suburban cases is feasible at approximate 51% higher than the total costs of current water and wastewater treatment plants, while distributed DPR plants can bring down at least 10% of that cost depending on the population density and landscape in the study area; and

• Unit cost of mineralizing DPR can be brought down further with the development of a mass market for advanced oxidation treatment technologies. Thus DPR systems are recommended to be gradual introduced to address the increasing water demand in the future, particularly in new construction zones.

General principles of design in DPR systems resulting from this dissertation include:
• The cost of DPR systems depends principally on population density. While costs in urban and suburban are balanced by capital cost in pipelines and treatment plants, pipeline installation cost dominates cost in rural cases;

• In urban and suburban cases, distributed DPR systems are competitive with centralized DPR system, and more economical than onsite systems, as a general result of savings in energy for residential hot water due to thermal energy retention, reductions in infiltration occurring in large wastewater collection networks, and reduction in upgradient distribution of treated reuse water. The average optimal capacities of DPR systems in urban (3860 persons/km$^2$) and suburban cases (772 persons/km$^2$) are at ca. 13000 and 6000 homes per treatment plant, respectively; and

• In rural cases, costs of complete COD-mineralizing DPR systems are high. Optimal system capacity varies according to landscape; onsite DPR systems are generally preferred except for flat landscapes (< 150 m maximum elevation difference). Regarding the high DPR cost in strictly rural cases (96 persons/km$^2$), onsite septic systems and systems designed to return nutrients to local agricultural sectors may be a more economical alternative to DPR in such areas.
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APPENDIX: HEAT LOSS DURING TRANSPORTATION OF WATER

The objective is to find the distance \( x \) (m), when the temperature of the water, \( T(x) \) (K), drops from the initial temperature \( T_1 \) (K) to the final constant temperature \( T_0 \) (K). Note that \( T_0 \) would be higher than the room temperature, and in this case the distance is independent with the initial temperature difference. Considering the Fourier’s Law of heat transfer in cylindrical coordinates for 1-D conduction in the radial direction, heat loss over unit distance \( \frac{dQ}{dx} \) (W/m) can be expressed as

\[
\frac{dQ}{dx} = \frac{2\pi k(T - T_0)}{r_1 h_1 + \ln(r_2 / r_1) + \frac{k}{r_2 h_2} + \frac{k}{dh_3}}
\]

in which \( k \) is thermal conductivity of the material \([W/(m\cdot K)]\), \( T, T_0 \) are temperatures of water and air \((K)\), \( h_1, h_2, h_3 \) are heat transfer coefficients on the surface of water and pipe, surface of pipe and soil, and surface of soil and air \([W/(m^2\cdot K)]\), and \( r_1, r_2 \) are inside and outside pipe radii \((m)\), \( d \) is the distance from the center of pipe to the surface of soil \((m)\).

Define an overall heat transfer coefficient, \( h_0 \) \([W/(m^2\cdot K)]\),

\[
dQ = 2\pi r_2 h_0 (T_0 - T) dx
\]

Thus from (1) and (2), \( h_0 \) can be expressed as

\[
\frac{1}{h_0} = \frac{r_2}{r_1 h_1} + \frac{r_2}{k} \ln\left(\frac{r_2}{r_1}\right) + \frac{1}{h_2} + \frac{r_2}{dh_3}
\]

Also, the change of enthalpy in the water is

\[
dQ = \dot{m} c_p dT = \rho v c_p \pi r_1^2 dT
\]

in which \( \dot{m} \) is the mass flow \((kg/s)\), \( c_p \) is the heat capacity \((J/kg)\), \( \rho \) is the density \((kg/m^3)\), \( v \) is the flow speed \((m/s)\).

Set Equation (2) and (4) equal and integrate from the start of the pipe,

\[
\int_{T_1}^{T} \frac{dT}{T_0 - T} = \int_{0}^{x} \frac{2r_2 h_0}{\rho v c_p r_1^2} dx
\]

i.e.

\[
\ln\left(\frac{T_0 - T}{T_0 - T_1}\right) = - \frac{2r_2 h_0 x}{\rho v c_p r_1^2}
\]

or
\[ T = T_0 + (T_1 - T_0) \cdot e^{\frac{-2 \pi h x}{\rho c_p r_1}} \]  

Equation (7)

Assuming initial water temperature \( T_1 = 300 \text{K} \), final temperature \( T_0 = 293 \text{ K} \), inside pipe heat transfer coefficient of surface of water and pipe \( h_1 = 3000 \text{ W/}(\text{m}^2*\text{K}) \), coefficient of surface of pipe and soil \( h_2 = 1.30 \text{ W/}(\text{m}^2*\text{K}) \), coefficient of surface of soil and air \( h_3 = 10 \text{ W/}(\text{m}^2*\text{K}) \) (Whitelaw, 2011; ASHRAE, 2012), density of water \( \rho = 1000 \text{ kg/m}^3 \), average water velocity \( v = 0.2 \text{ m/s} \), heat capacity of water \( c_p = 4.2 \times 10^3 \text{ J/kg} \), distance from surface to the center of pipe \( d=0.3 \text{ m} \) (12 inch) (City of Carlsbad, 2009), Equation (7) can be calculated with different assumptions of the pipes.

Scenario 1. No insulation, Cast iron pipe.

![Figure 1](image-url)

Figure 1. Water temperature along the pipe with assumptions that inside radii \( r_1 = 0.025 \text{ m} \) (0.98 inch), outside pipe radii \( r_2 = 0.030 \text{ m} \) (1.18 inch) (Peterson Product, 2014), thermal conductivity of cast iron pipes \( k = 60 \text{ W/m*K} \) (Engineering toolbox, 2014).
Scenario 2. No insulation, Copper pipe. The simulated profiles are almost the same with Scenario 1.

Figure 2. Water temperature along the pipe with assumptions that inside radii $r_1 = 0.025$ m (0.98 inch), outside pipe radii $r_2 = 0.030$ m (1.18 inch) (Peterson Product, 2014), thermal conductivity of Copper pipes $k = 386$ W/m*K (Engineering toolbox, 2014).

Scenario 3. Copper pipe with insulation.

Figure 3. Water temperature along the pipe with assumptions that inside radii $r_1 = 0.025$ m (1 inch), outside pipe radii $r_2 = 0.051$ m (2 inch), thermal conductivity $k = 0.023$ W/m*K (Insapipe Industries Ltd., 2014),
Thus, for insulated pipe, the saved water price that expressed as the temperature difference, i.e., $T - T_0$, can also be calculated from Equation (7):

$$Q = mc_p \Delta T = mc_p (T - T_0) = mc_p (T_1 - T_0) \cdot e^{\frac{2r_h h_c}{mc_p \rho c_p r_1^2}}$$

(8)

References


